DESIGN GUIDE FOR

SEMI-RIGID COMPOSITE JOINTS AND BEAMS

2026 Edition

(3rd Edition)

J Y Richard Liew
Yuichi Nishida
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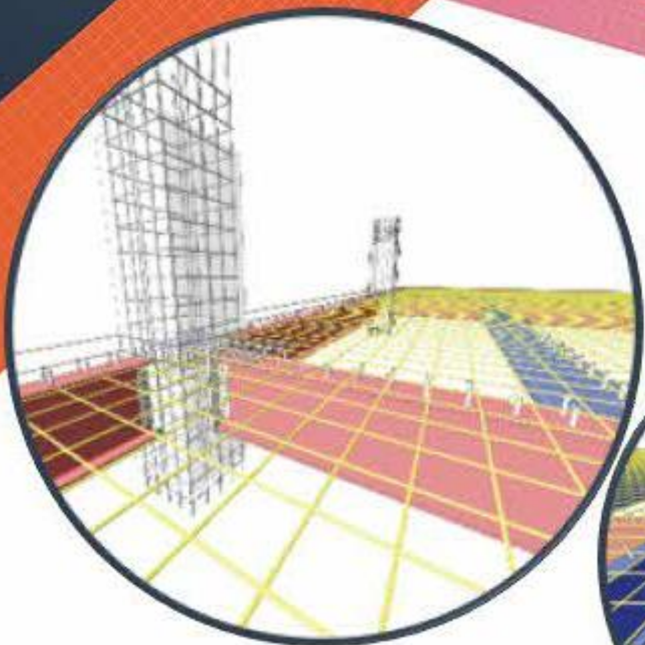

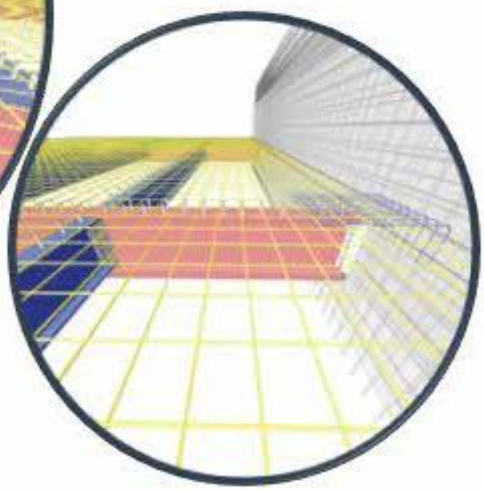

SINGAPORE STRUCTURAL STEEL SOCIETY

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SSSS & BCA Design Guide Series

Design Guide for Semi-rigid Composite Joints and Beams

(2026 Edition)

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Contents

Foreword

Multi-storey steel–concrete composite buildings that are braced against sidesway but not subjected to significant wind or seismic actions often adopt simplified beam-to-column and beam-to-beam joints, such as fin-plate bolted connections, to accelerate construction. Although these joints are straightforward to install and are widely preferred for their simplicity, modern commercial buildings with long-span floor systems and open-plan layouts may benefit from structural frames with semi-rigid connections. Such frames can achieve greater overall economy without resorting to complex rigid joint detailing.

According to [EN 1993-1-8][1], steel joints are classified as pinned, semi-rigid, or rigid on the basis of their initial rotational stiffness and moment resistance, depending on the analysis method adopted in design. Composite joints, as defined in [EN 1994-1-1][2], are those in which the slab reinforcement is taken into account in evaluating rotational stiffness and moment resistance, provided that the reinforcement is continuous or properly anchored at the joint. Consequently, some simple steel joints defined in EN 1993-1-8 may be treated as semi-rigid composite joints in accordance with EN 1994-1-1 if the floor slab reinforcement is continuous or anchored at the joints and provides a measurable degree of rotational restraint.

The previous edition of this design guide was limited to beam-to-beam composite joints in which both primary and secondary beams act compositely with the floor slab. It introduced a contact-type semi-rigid composite joint capable of developing enhanced rotational stiffness and moment resistance. In the associated mechanical model, the tensile force is transmitted through the reinforcing bars, while the compressive force is transferred through contact plates inserted at the bottom flange of the steel beam, thereby improving both stiffness and resistance.

This new edition extends the scope of application to a wider range of joints, including:

- beam-to-beam joints,
- steel beams connected to reinforced concrete primary beams,
- beam-to-column joints, and
- beam-to–reinforced concrete column or wall joints,

all incorporating contact plates to enhance semi-rigidity. In addition, a beam-to-corner column joint is now covered, in which a reinforced concrete column is biaxially connected to steel beams. The design procedures presented can therefore be applied to almost all connections within a typical floor beam layout. The proposed methods for contact-type semi-rigid joints have been validated by finite element analyses and full-scale tests carried out at the Steel Structures Research Laboratory of Nippon Steel Corporation in Japan.

This design guide is intended to give structural engineers the confidence to adopt semi-rigid composite joints safely and economically in the design and construction of steel–concrete composite buildings. With this updated guidance, designers can make wider and more efficient use of semi-rigid joints, potentially reduce construction time and cost while maintaining the required levels of structural performance.

References:

[1] EN 1993-1-8: Eurocode 3 – Design of steel structures – Part 1-8: Design of joints
[2] EN 1994-1-1: Eurocode 4 – Design of composite steel and concrete structures – Part 1-1: General rules and rules for buildings

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March, 2026

Acknowledgement

The Singapore Structural Steel Society ("SSSS") and Building and Construction Authority of Singapore ("BCA") would like to thank the authors for their hard work and dedication in developing this Guidebook. We would also like to extend our gratitude to the members of the expert committee for their valuable comments and contributions. Your efforts have helped make this Guidebook a comprehensive and valuable resource for the industry.

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List of Symbols

The following symbols are used in this design guide.

A_a	Cross-sectional area of steel beam
A_b	Tensile stress area of bolt
A_{bea}	Bearing area between bottom flange of steel beam and contact plate
A_{bf}	Cross-sectional area of bottom flange of steel beam
$A_{bwV,g}$	Shear area of web of steel beam for gross section
$A_{bwV,n}$	Shear area of web of steel beam for net section
A_c	Area per unit length of concrete slab
$A_{c,c}$	Area per unit length of concrete slab in compression
A_{cp}	Cross-sectional area of contact plate
A_{cs}	Cross-sectional area of composite slab within b_{effh} above profiled steel sheeting
a_{fp}	Effective throat thickness of fillet weld of fin plate
$A_{fp,nt}$	Net area of fin plate subjected to tension
$A_{fp,nV}$	Net area of fin plate subjected to shear
$a_{fp,req}$	Required minimum throat thickness of fillet weld of fin plate
$A_{fpV,n}$	Shear area of fin plate for net section
A_{pse}	Effective cross-sectional area of profiled steel sheeting per unit length
A_{sl}	Cross-sectional area of longitudinal reinforcing bars within b_{effh}
$A_{sl,j}$	Cross-sectional area of longitudinal reinforcing bars within $b_{eff,j}$; Cross-sectional area of bent reinforcing bars effective for joint design
$A_{sl,r}$	Cross-sectional area of longitudinal reinforcing bars within $b_{eff,j}$ for a row r
$A_{sl,req}$	Required minimum cross-sectional area of longitudinal reinforcing bars within b_{effh}
$A_{sl,1}$	Cross-sectional area of longitudinal reinforcing bars within $b_{eff,j}$ for row 1
$A_{sl,2}$	Cross-sectional area of longitudinal reinforcing bars within $b_{eff,j}$ for row 2
A_{st}	Cross-sectional area of transverse reinforcing bars per unit length
$A_{st,req}$	Required minimum cross-sectional area of transverse reinforcing bars per unit length
$A_{st,1}$	Cross-sectional area of transverse reinforcing bars per unit length for row 1
A_V	Shear area of steel beam
A_w	Total cross-sectional area of crossties or hoops within raking-out region
B_a	Width of steel beam
B_b	Beam spacing
B_{cc}	Width of reinforced concrete column
B_{cw}	Effective width of reinforced concrete wall
b_e	Effective width of raking-out region
b_{eff}	Effective width of equivalent T-stub flange in compression
$b_{eff,b}$	Effective width of composite beam assuming simply supported condition
b_{effh}	Effective width of composite beam in hogging moment region
$b_{eff,j}$	Effective width of composite joint

b_{effs}	Effective width of composite beam in sagging moment region
b_{eih}	Value of effective width of composite beam on each side of web of steel beam in hogging moment region
b_{eis}	Value of effective width of composite beam on each side of web of steel beam in sagging moment region
B_{ep}	Width of end plate
b_{ih}	Distance from outstand headed stud to a point mid-way between adjacent webs of steel beams in hogging moment region
b_{is}	Distance from outstand headed stud to a point mid-way between adjacent webs of steel beams in sagging moment region
b_{jp}	Effective width of joint panel
B_{pb}	Width of primary reinforced concrete beam
b_{sl}	Arrangement width of additional longitudinal reinforcing bars
$b_{sl,2}$	Arrangement width of additional longitudinal reinforcing bars (row 2)
b_{0h}	Distance between centres of outstand headed studs in hogging moment region
$b_{0,max}$	Maximum width for re-entrant of profiled steel sheeting
$b_{0,min}$	Minimum width for re-entrant of profiled steel sheeting
b_{0s}	Distance between centres of outstand headed studs in sagging moment region
c	Additional bearing width of equivalent T-stub flange in compression
c_{min}	Required minimum cover for reinforcing bars in concrete slab
c_{nom}	Nominal cover for reinforcing bars in concrete slab
C_0	Horizontal covering depth of bent reinforcing bars
C_1	Correction factor for non-uniform bending moment
C_4	Property of distribution of moment
D_a	Depth of steel beam
d_b	Size of bolt
D_{cc}	Depth of reinforced concrete column
D_{cs}	Overall depth of composite slab
d_e	Effective depth of raking-out region
d_{eff}	Effective depth of equivalent T-stub flange in compression
D_{ep}	Depth of end plate
D_{fp}	Depth of fin plate
d_{hs}	Diameter of shank of headed stud
D_{pb}	Depth of primary reinforced concrete beam
D_{ps}	Overall depth of profiled steel sheeting
d_0	Hole diameter of bolt
e	e value
E_a	Modulus of elasticity of steel beam
$e_{b\text{-}bw,h}$	Edge distance for web of steel beam on horizontal line
$e_{b\text{-}bw,v}$	Edge distance for web of steel beam on vertical line
$e_{b\text{-}fp,h}$	Edge distance for fin plate on horizontal line
$e_{b\text{-}fp,v}$	Edge distance for fin plate on vertical line
E_{cm}	Secant modulus of elasticity of concrete

$E_{cm,cc}$	Secant modulus of elasticity of concrete for reinforced concrete column
$E_{cm,cs}$	Secant modulus of elasticity of concrete for concrete slab
$E_{cm,cw}$	Secant modulus of elasticity of concrete for reinforced concrete wall
$E_{cm,pb}$	Secant modulus of elasticity of concrete for primary reinforced concrete beam
E_{fp}	Modulus of elasticity of fin plate
$(EI)_h$	Hogging flexural rigidity of composite beam
$(EI)_{h,l}$	Hogging flexural rigidity of composite beam at left side
$(EI)_{h,r}$	Hogging flexural rigidity of composite beam at right side
$(EI/L)_b$	Flexural stiffness of beam member
$(EI)_s$	Sagging flexural rigidity of composite beam
E_{lcm}	Secant modulus of elasticity of lightweight concrete
E_s	Modulus of elasticity of reinforcing bars
f_{au}	Ultimate tensile resistance of steel beam
f_{ay}	Nominal value of yield strength of steel beam
f_{ayd}	Design yield strength of steel beam
f_{bu}	Ultimate tensile strength of bolt
$F_{bV,Rd}$	Shear resistance of a single bolt
f_{by}	Nominal value of yield strength of bolt
f_{cd}	Design strength of concrete
$f_{cd,cc}$	Design strength of concrete for reinforced concrete column
$f_{cd,cs}$	Design strength of concrete for concrete slab
$f_{cd,cw}$	Design strength of concrete for reinforced concrete wall
$f_{cd,pb}$	Design strength of concrete for primary reinforced concrete beam
f_{ck}	Characteristic cylinder strength of concrete
$f_{ck,cc}$	Characteristic cylinder strength of concrete for reinforced concrete column
$f_{ck,cs}$	Characteristic cylinder strength of concrete for concrete slab
$f_{ck,cw}$	Characteristic cylinder strength of concrete for reinforced concrete wall
$f_{ck,pb}$	Characteristic cylinder strength of concrete for primary reinforced concrete beam
f_{cpy}	Nominal value of yield strength of contact plate
f_{cpyd}	Design yield strength of contact plate
$F_{C,Rd}$	Compression resistance of equivalent T-stub flange
f_{ctm}	Mean value of tensile strength of concrete
f_{epy}	Nominal value of yield strength of end plate
f_{epyd}	Design yield strength of end plate
f_{fpu}	Ultimate tensile strength of fin plate
f_{fpy}	Nominal value of yield strength of fin plate
$F_{hbb,Rd}$	Horizontal bearing resistance of a single bolt
f_{hsu}	Ultimate strength of headed stud
f_{jd}	Design bearing strength of concrete
$f_{jd,cc}$	Design bearing strength of concrete for reinforced concrete column
$f_{jd,cw}$	Design bearing strength of concrete for reinforced concrete wall
$f_{jd,pb}$	Design bearing strength of concrete for primary reinforced concrete beam

F_{jp}	Nominal value of shear resistance of joint panel
f_{lck}	Characteristic cylinder strength of lightweight concrete
f_{lctm}	Mean value of tensile strength of lightweight concrete
f_{psd}	Design yield strength of profiled steel sheeting
f_{psk}	Characteristic yield strength of profiled steel sheeting
$f_{P+0.1V}$	Natural frequency due to "dead loads, superimposed dead loads, and 10% of live loads"
f_{req}	Required minimum natural frequency
f_{sd}	Design yield strength of reinforcing bars
f_{sk}	Characteristic yield strength of reinforcing bars
$F_{vbb,Rd}$	Vertical bearing resistance of a single bolt
f_{wk}	Characteristic yield strength of crossties or hoops
f_{wu}	Ultimate tensile strength of web of steel beam
f_{wy}	Nominal value of yield strength of web of steel beam
G_a	Shear modulus of elasticity of steel beam
g_a	Mass per metre of steel beam
$g_{k,1}$	Dead load per unit area in construction stage
$g_{k,2}$	Dead load per unit area in composite stage
$g_{k,3}$	Superimposed dead load per unit area in composite stage
g_{ps}	Mass per metre of profiled steel sheeting
$h_{cj\text{-}fhs}$	Distance between centre of joint and first headed stud
h_{cs}	Thickness of composite slab above profiled steel sheeting
H_{cw}	Effective height of reinforced concrete wall
h_{fhs}	Distance between first headed studs on opposite composite beams
h_{hs}	Overall height of headed stud
$h_{pb\text{-}fhs}$	Distance between surface of primary beam and first headed stud
$h_{sm\text{-}fhs}$	Distance between surface of supporting member and first headed stud
i_{ax}	Polar radius of gyration of area of steel beam
I_{ay}	Second moment of area of steel beam about major axis (y-y axis)
I_{az}	Second moment of area of steel beam about minor axis (z-z axis)
I_b	Second moment of area of composite beam
I_{bfz}	Second moment of area of bottom flange of steel beam about minor axis (z-z axis)
I_{cs2}	Second moment of area of cracked composite slab in direction transverse to steel beam
I_h	Second moment of area of composite beam in hogging moment region
i_{fpz}	Radius of gyration of area of fin plate about minor axis (z-z axis)
$I_{T,a}$	Torsion constant of steel beam
$I_{w,a}$	Warping constant of steel beam
k_c	Coefficient taking into account stress distribution within section immediately prior to cracking; k_c factor; Coefficient for effect of horizontal cover
k_{con}	Stiffness of elastic spring for the contact part
k_d	Coefficient for effect of projected anchorage length
k_j	Coefficient for effect of bent position

k_s	Transverse (rotational) stiffness per unit length of composite beam; Coefficient for effect of crossties or hoops
k_{sc}	Stiffness of one headed stud
K_{sc}	Stiffness related to headed studs
k_{slip}	Stiffness reduction factor due to deformation of headed studs
$k_{sl,eq}$	Equivalent stiffness coefficient of longitudinal reinforcing bars
k_{th}	Reduction factor for shear resistance of a headed stud in hogging moment region
$k_{th,max}$	Maximum reduction factor for shear resistance of a headed stud in hogging moment region
$k_{t,max}$	Maximum reduction factor for shear resistance of a headed stud
k_{ts}	Reduction factor for shear resistance of a headed stud in sagging moment region
$k_{ts,max}$	Maximum reduction factor for shear resistance of a headed stud in sagging moment region
$k_{\tau,min}$	Minimum shear buckling coefficient
k_1	Flexural stiffness of cracked composite slab in direction transverse to steel beam
$k_{1,hbb}$	k_1 factor for horizontal bolt bearing resistance
$k_{1,vbb}$	k_1 factor for vertical bolt bearing resistance
$k_{13,cc}$	Stiffness coefficient of concrete for reinforced concrete column
$k_{13,cw}$	Stiffness coefficient of concrete for reinforced concrete wall
$k_{13,pb}$	Stiffness coefficient of concrete for primary reinforced concrete beam
k_2	Flexural stiffness of web of steel beam
l	Length of composite beam in hogging moment region adjacent to joint
L_b	Beam length; Beam span
$L_{b,A}$	Beam length of composite beam (A)
l_b	Basic anchorage length of longitudinal reinforcing bars
L_{cr}	Length of composite beam between points at which bottom flange is laterally restrained
$L_{cr,a}$	Length of steel beam between points at which top flange of steel beam is laterally restrained
l_{dh}	Projected anchorage length of bent reinforcing bars
$l_{dh,r}$	Projected anchorage length of bent or hooked reinforcing bars for a row r
l_{dv}	Projected extension length of bent reinforcing bars
L_e	Distance between inflection points
l_{eff}	Effective length of reinforcement
$l_{eff,r}$	Effective length of reinforcement for a row r
$l_{eff,1}$	Effective length of reinforcement for row 1
$l_{eff,2}$	Effective length of reinforcement for row 2
L_{eh}	Distance between inflection points in hogging moment region
L_{es}	Distance between inflection points in sagging moment region
l_{sl}	Arrangement length of additional longitudinal reinforcing bars
l_v	Straight extension length of bent reinforcing bars

l_0	Design lap length of longitudinal reinforcing bars
$l_{0,min}$	Minimum lap length of longitudinal reinforcing bars
M_{cr}	Elastic critical moment for lateral-torsional buckling of composite beam
$M_{cr,a}$	Elastic critical moment for lateral-torsional buckling of steel beam
M_{Edh}	Design hogging moment
M_{Eds}	Design sagging moment
$M_{Eds,A}$	Design sagging moment of composite beam (A)
$M_{el,fp,Rd}$	Elastic moment resistance of fin plate
$M_{el,Rdh}$	Elastic hogging moment resistance of composite beam
$M_{el,vbw,Rd}$	Elastic moment resistance of web of steel beam on vertical line of bolts
$M_{h,A}$	Actual end moment of composite beam (A)
$M_{h,(wA,max)}$	End moment of composite beam (A) due to $w_{A,max}$; Released moment
$M_{h,(wB,min)}$	End moment of composite beam (B) due to $w_{B,min}$; Released moment
M_j	Joint moment
$M_{j,l}$	Joint moment at left side
$M_{j,r}$	Joint moment at right side
$M_{j,Rd}$	Moment resistance of composite joint
$M_{j,Rd,A}$	Moment resistance of beam-to-beam composite joint applied to composite beam (A)
$M_{LT,a,Rd}$	Buckling moment resistance of laterally unrestrained steel beam
$M_{LT,fp,Rd}$	Lateral torsional buckling moment resistance of fin plate
$M_{LT,Rd}$	Buckling moment resistance of laterally unrestrained composite beam
$M_{pl,a,Rd}$	Plastic moment resistance of steel beam
$M_{pl,f,Rd}$	Plastic moment resistance of composite beam after deducting shear area
$M_{plf,Rdh}$	Plastic hogging moment resistance of composite beam with full shear connection
$M_{plf,Rds}$	Plastic sagging moment resistance of composite beam with full shear connection
$M_{plp,Rds}$	Plastic sagging moment resistance of composite beam with partial shear connection
$M_{pl,Rd}$	Plastic moment resistance of composite beam
$M_{pl,Rkh}$	Characteristic value of plastic hogging moment resistance of composite beam
$M(x)$	Moment of composite beam along x-axis
$M_{y,v,Rdh}$	Reduced moment resistance of composite beam making allowance for presence of shear force
$M_{y,v,vbw,Rd}$	Reduced moment resistance of web of steel beam on vertical line of bolts making allowance for presence of shear force
M_0	Mid-length moment of simply supported beam
N	Number of headed studs distributed over length l
$n_{b,h}$	Number of bolts on horizontal line
$n_{b,v}$	Number of bolts on vertical line
n_{hs}	Number of headed studs per sheeting rib
n_{hsh}	Number of headed studs per sheeting rib in hogging moment region
N_{hsh}	Number of headed studs arranged within half of L_{eh}

n_{hss}	Number of headed studs per sheeting rib in sagging moment region
N_{hss}	Number of headed studs arranged within half of L_{es}
N_L	Longitudinal force in composite slab
n_0	Modular ratio for short-term loading
$p_{b,h}$	Pitch of bolts on horizontal line
$p_{b,v}$	Pitch of bolts on vertical line
p_{ps}	Pitch of ribs of profiled steel sheeting
P_{Rd}	Shear resistance of a headed stud
p_{sl}	Pitch (spacing) of longitudinal reinforcing bars
$p_{sl,lim}$	Limit of spacing of longitudinal reinforcing bars
$p_{sl,1}$	Pitch of anti-crack longitudinal reinforcing bars (row 1)
$p_{sl,2}$	Pitch of additional longitudinal reinforcing bars (row 2)
$p_{st,1}$	Pitch of anti-crack transverse reinforcing bars (row 1)
$p_{st,2}$	Pitch of additional transverse reinforcing bars (row 2)
$q_{k,1}$	Construction load per unit area in construction stage
$q_{k,2}$	Imposed floor load per unit area in composite stage
r	Root radius of steel beam
R_a	Tension (compression) resistance of steel beam
R_{con}	Compression resistance of contact part
R_{cs}	Compression resistance of composite slab within b_{effs}
$R_{eff,a}$	Tension (compression) resistance of effective steel beam
$R_{eff,v}$	Tension (compression) resistance of effective clear web of steel beam
R_f	Tension (compression) resistance of flange of steel beam
R_{qh}	Longitudinal shear force transfer within half of L_{eh}
R_{qs}	Longitudinal shear force transfer within half of L_{es}
$R_{st}+R_{pse}$	Tension resistance of transverse reinforcement per unit length
R_{sl}	Tension resistance of longitudinal reinforcing bars within b_{effh}
$R_{sl,j}$	Tension resistance of longitudinal reinforcing bars within $b_{eff,j}$
$R_{tr,\ req}$	Required tension resistance of transverse reinforcement per unit length
R_v	Tension (compression) resistance of clear web of steel beam
R_w	Tension (compression) resistance of overall web of steel beam
s_{fp}	Length of fillet weld of fin plate
S_j	Rotational stiffness of composite joint
$S_{j,A}$	Rotational stiffness of beam-to-beam composite joint applied to composite beam (A)
$S_{j,B}$	Rotational stiffness of beam-to-beam composite joint applied to composite beam (B)
$S_{j,ini}$	Initial rotational stiffness of composite joint
$S_{j,ini,eq}$	Equivalent initial rotational stiffness of composite joint
$S_{j,l}$	Rotational stiffness of composite joint at left side
$S_{j,r}$	Rotational stiffness of composite joint at right side
T_c	Contribution of concrete for T_{ro}
t_{cw}	Thickness of reinforced concrete wall
t_{ep}	Thickness of end plate

t_f	Flange thickness of steel beam
t_{fp}	Thickness of fin plate
T_{lc}	Ultimate tensile strength of reinforcing bars due to local compression failure
t_{ps}	Thickness of profiled steel sheeting
T_{ro}	Ultimate tensile strength of reinforcing bars due to raking-out failure
t_w	Web thickness of steel beam
T_w	Contribution of spreader bars for T_{ro}
$V_{b,a,Rd}$	Shear buckling resistance of steel beam
$V_{bb,Rd}$	Bolt bearing resistance
$V_{b,Rd}$	Shear buckling resistance of composite beam; Bolt shear resistance
$V_{bw,Rd,g}$	Shear resistance of web of steel beam for gross section
$V_{bw,Rd,n}$	Shear resistance of web of steel beam for net section
V_{Ed}	Design shear force
$V_{fp,Rd,b}$	Fin plate block shear resistance
$V_{fp,Rd,g}$	Fin plate shear resistance for gross section
$V_{fp,Rd,n}$	Fin plate shear resistance for net section
V_{jp}	Shear resistance of joint panel
$v_{L,Ed}$	Design longitudinal shear stress in composite slab
$V_{pl,a,Rd}$	Plastic shear resistance of steel beam
$V_{pl,hbw,Rd}$	Plastic shear resistance of web of steel beam on top and bottom horizontal line of bolts
$V_{pl,Rd}$	Plastic shear resistance of composite beam
$V_{pl,vbw,Rd}$	Plastic shear resistance of web of steel beam on vertical line of bolts
v_{Rd}	Crushing shear stress of concrete slab
$V_{vbw,Ed}$	Design shear force of web of steel beam on vertical line of bolts
w	Uniformly distributed load
$w_{A,max}$	Maximum uniformly distributed load of composite beam (A)
$w_{B,min}$	Minimum uniformly distributed load of composite beam (B)
$w_{com,max}$	Maximum design distributed load in composite stage
$w_{com,min}$	Minimum design distributed load in composite stage
$w_{com,P}$	Design distributed load due to "superimposed dead loads" in composite stage
$w_{com,P+0.1V}$	Design distributed load due to "dead loads, superimposed dead loads, and 10% of live loads"
$w_{com,V,max}$	Maximum design distributed load due to "live loads" in composite stage
$w_{com,V,min}$	Minimum design distributed load due to "live loads" in composite stage
$w_{con,max}$	Maximum design distributed load in construction stage
$w_{con,P}$	Design distributed load due to "dead loads" in construction stage
$w_{con,P+V}$	Design distributed load due to "dead loads and live loads" in construction stage
$w_{con,V}$	Design distributed load due to "live loads" in construction stage
$W_{eff,pl,a}$	Effective plastic section modulus of steel beam
w_k	Design crack width
$W_{pl,a}$	Plastic section modulus of steel beam
x_δ	x-coordinate where deflection is maximized

x_0	x-coordinate at inflection point
x_0'	x-coordinate at inflection point
$z_{ccs\text{-}ca}$	Vertical distance between centre of composite slab and centre of steel beam
$z_{csl\text{-}tf}$	Vertical distance between centre of longitudinal reinforcing bars and top of flange of steel beam
$z_{csl\text{-}cc}$	Vertical distance between bent reinforcing bars and centre of contact part
$z_{csl,1\text{-}cc}$	Vertical distance between centre of longitudinal reinforcing bars and centre of contact part for row 1
$z_{csl,2\text{-}cc}$	Vertical distance between centre of longitudinal reinforcing bars and centre of contact part for row 2
$z_{cst\text{-}na}$	Vertical distance between centre of transverse reinforcing bars and neutral axis of composite beam
$z_{ctf\text{-}cbf}$	Vertical distance between centres of top and bottom flanges of steel beam
$z_{fs\text{-}b}$	Distance between face of support and assumed line of shear transfer
$z_{na\text{-}ccs,c}$	Vertical distance between neutral axis of composite slab and centre of concrete slab in compression
$z_{sl,eq\text{-}ca}$	Equivalent vertical distance between longitudinal reinforcing bars and centre of steel beam
$z_{sl,eq\text{-}cc}$	Equivalent vertical distance between longitudinal reinforcing bars and centre of contact part
$z_{sl,eq\text{-}na}$	Equivalent vertical distance between longitudinal reinforcing bars and neutral axis of composite beam
$z_{sl,eq\text{-}tf}$	Equivalent vertical distance between longitudinal reinforcing bars and top of flange of steel beam
$z_{st,eq\text{-}ccs,c}$	Equivalent vertical distance between transverse reinforcing bars and centre of concrete slab in compression
$z_{st,eq\text{-}na}$	Equivalent vertical distance between transverse reinforcing bars and neutral axis of composite slab
$z_{tcs\text{-}csl}$	Covering depth of longitudinal reinforcing bars
$z_{tcs\text{-}csl,1}$	Covering depth of longitudinal reinforcing bars for row 1
$z_{tcs\text{-}csl,2}$	Covering depth of longitudinal reinforcing bars for row 2
$z_{tcs\text{-}cst,1}$	Covering depth of transverse reinforcing bars for row 1
$z_{tcs\text{-}cst,2}$	Covering depth of transverse reinforcing bars for row 2
z_0	Vertical distance between centre of un-cracked concrete flange and un-cracked composite section
α	Portion of part of cross-section in compression; α factor; Amplification factor from loaded area to maximum design distribution area
α_{bV}	Correction factor for bolt shear resistance
α_{LT}	Imperfection factor corresponding to appropriate lateral-torsional buckling curve
α_{hbb}	Correction factor for horizontal bolt bearing resistance
α_{vbb}	Correction factor for vertical bolt bearing resistance
α_1	Coefficient considering shape of reinforcing bars

α_2	Coefficient considering concrete cover
α_3	Coefficient considering confinement by transverse reinforcing bars
α_5	Coefficient considering confinement by transverse pressure
α_6	Coefficient considering percentage of lapped reinforcing bars
β	β factor
β_j	Joint material coefficient of precast reinforced concrete beam
γ_a	Partial factor of resistance of members and cross-sections of steel beam
$\gamma_{a,2}$	Partial factor of resistance of steel beam in bearing
γ_b	Partial factor of bolt
γ_c	Partial factor of concrete
$\gamma_{c,cc}$	Partial factor of concrete for reinforced concrete column
$\gamma_{c,cs}$	Partial factor of concrete for concrete slab
$\gamma_{c,cw}$	Partial factor of concrete for reinforced concrete wall
γ_{cp}	Partial factor of resistance of members and cross-sections of contact plate
$\gamma_{c,pb}$	Partial factor of concrete for primary reinforced concrete beam
$\gamma_{cp,2}$	Partial factor of resistance of contact plate in bearing
γ_{ep}	Partial factor of resistance of members and cross-sections of end plate
γ_{fp}	Partial factor of resistance of members and cross-sections of fin plate
$\gamma_{fp,2}$	Partial factor of resistance of fin plate in bearing
$\gamma_{G,sup}$	Partial factor for permanent actions (unfavourable)
$\gamma_{G,inf}$	Partial factor for permanent actions (favourable)
γ_{ps}	Partial factor of profiled steel sheeting
γ_Q	Partial factor for variable actions (unfavourable)
γ_{Qi}	Partial factor for variable actions (favourable)
γ_s	Partial factor of reinforcing bars
γ_V	Partial factor of headed stud
γ_w	Partial factor of crossties or hoops
δ_A	Deflection of composite beam (A)
δ_{max}	Maximum deflection of composite beam
δ_{P+V}	Deflection due to “dead loads and live loads”
$\delta_{P+V,lim}$	Limit of deflection due to “dead loads and live loads”
$\delta_{P+0.1V}$	Deflection due to “dead loads, superimposed dead loads, and 10% of live loads”
δ_{tP}	Deflection due to “dead loads and superimposed dead loads”
δ_{tP+V}	Deflection due to “dead loads, superimposed dead loads, and live loads”
δ_V	Deflection due to “live loads”
$\delta_{V,lim}$	Limit of deflection due to “live loads”
η	Stiffness modification coefficient
η_h	Degree of shear connection in hogging moment region
$\eta_{h,req}$	Required minimum degree of shear connection in hogging moment region
η_s	Degree of shear connection in sagging moment region

$\eta_{s,req}$	Required minimum degree of shear connection in sagging moment region
θ	Angle between diagonal strut and axis of beam
θ_{min}	Minimum angle to minimize cross-sectional area of transverse reinforcing bars
θ_{pin}	Rotation of pin joint
κ	Coefficient for effect of joint type
λ_{LT}	Non-dimensional slenderness for lateral-torsional buckling of composite beam
$\lambda_{LT,a}$	Non-dimensional slenderness for lateral-torsional buckling of steel beam
$\lambda_{LT,fp}$	Non-dimensional slenderness for lateral torsional buckling of fin plate
λ_w	Modified slenderness of web of steel beam
μ_A	Distribution factor for composite beam (A)
ν	Parameter related to deformation of headed studs
ξ	Parameter related to deformation of headed studs
ρ_c	Dry density of concrete
ρ_{lc}	Dry density of lightweight concrete
$\rho_{sl,req}$	Required minimum reinforcement ratio
σ_{sl}	Tensile stress in longitudinal reinforcing bars due to direct loading
$\sigma_{sl,lim}$	Limit of stress permitted in longitudinal reinforcing bars immediately after cracking
$\sigma_{sl,0}$	Stress in longitudinal reinforcing bars caused by M_{Edh}
ϕ	Correction coefficient due to presence of transverse beam
ϕ_j	Joint rotation
ϕ_m	Mandrel diameter of bent reinforcing bars
$\phi_{m,min}$	Minimum mandrel diameter of bent reinforcing bars
$\phi_{m,r}$	Mandrel diameter of bent reinforcing bars for a row r
Φ_{LT}	Value to determine reduction factor for lateral-torsional buckling of composite beam
$\Phi_{LT,a}$	Value to determine reduction factor for lateral-torsional buckling of steel beam
$\Phi_{LT,fp}$	Value to determine reduction factor for lateral-torsional buckling of fin plate
ϕ_{sl}	Diameter of longitudinal reinforcing bars; Diameter of bent reinforcing bars
$\phi_{sl,r}$	Diameter of longitudinal reinforcing bars for a row r
$\phi_{sl,1}$	Diameter of anti-crack longitudinal reinforcing bars (row 1)
$\phi_{sl,2}$	Diameter of additional longitudinal reinforcing bars (row 2)
ϕ_{sl}^*	Diameter of longitudinal reinforcing bars; Maximum diameter of longitudinal reinforcing bars
$\phi_{st,1}$	Diameter of anti-crack transverse reinforcing bars (row 1)
$\phi_{st,2}$	Diameter of additional transverse reinforcing bars (row 2)
ϕ_w	Diameter of crossties or hoops
χ_{LT}	Reduction factor for lateral-torsional buckling of composite beam
$\chi_{LT,a}$	Reduction factor for lateral-torsional buckling of steel beam

$\chi_{LT,fp}$	Reduction factor for lateral-torsional buckling of fin plate
χ_w	Factor for contribution of web of steel beam to shear buckling resistance
ψ	Ratio of the design hogging moment to M_0

Chapter 1 General

(1) Application

This design guide is applicable for the design of composite joints and composite beams with semi-rigid ends. The term “composite joint” in this design guide shall refer to a composite joint in which at least two essential components, reinforcing bars in tension and contact parts in compression, are considered in design for the rotational stiffness and moment resistance of the joint. The term “composite beam” shall refer to a structural member with a steel beam and reinforced concrete slab or composite slab interconnected by shear connectors.

(2) Design standards

This design guide is based on EN 1993-1-8 and EN 1994-1-1 for the design of composite joints with detailed methods developed for practical use. The other European Standards can be referred for the matters not covered in this design guide.

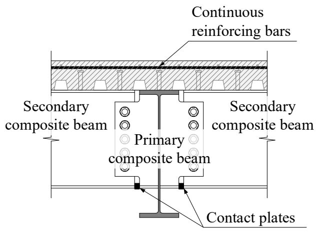
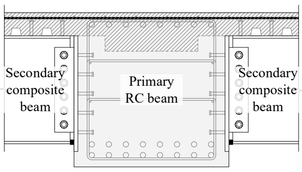
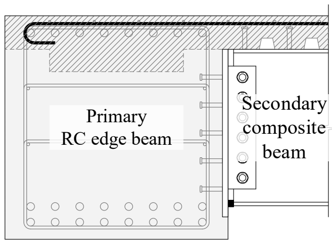
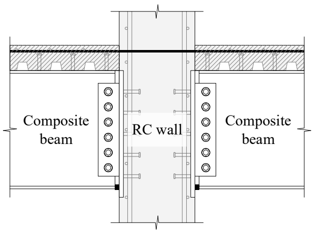
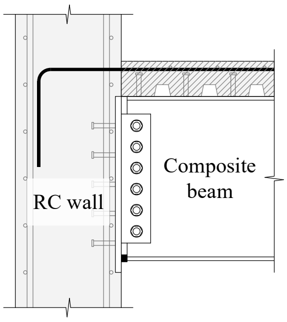
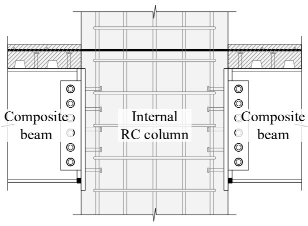
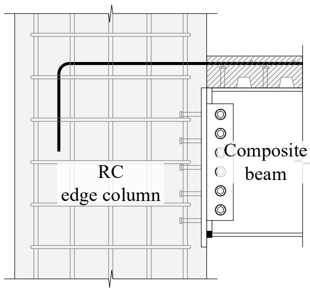
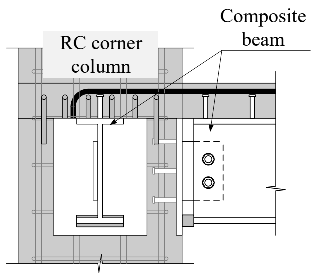
(3) Types of composite joints

Composite beams can be designed with semi-rigid ends by having continuous reinforcing bars over the supporting members or by having reinforcing bars anchored to them. The supporting members correspond to beams, walls, and columns in building frames. Table 1.1 shows the typical composite joint details covered in this design guide. Contact plates may be attached at the bottom flange of the steel beams to ensure proper contact with the supporting members.

(4) Composite joint without contact plates

Joint details without contact plates may be conservatively designed as pinned joint by ignoring the contribution of the slab reinforcement. Alternatively, the rotational stiffness and moment resistance of a joint should be determined by analogy to provisions for steel joints given in EN 1993-1-8, taking into account of reinforcement in the slab. Rotational capacity of composite joint may be demonstrated by experimental evidence, unless such details, when used in practice, have proven adequate properties.

Table 1.1: Composite joint details with contact plates

Supporting members	Composite beams with		
	Continuous reinforcing bars	Anchored reinforcing bars	
		Uniaxial	Biaxial
Primary Composite beam			
Primary RC beam			
RC wall			
RC column			

Chapter 2 Materials

2.1 Structural Steel

(1) Yield strength

The nominal yield strength of steel beams f_{ay} should be less than or equal to 355 [N/mm^2]. The nominal yield strength of other structural steel materials such as fin plates, stiffeners, and end plates should be at least a matching grade as the beam but cannot be higher than 460 [N/mm^2].

(2) Steel grade

The typical steel grades of structural steel are given in Table 2.1.

Table 2.1: Steel grades of structural steel

Steel grade	Nominal values of yield strength / ultimate tensile strength [N/mm^2] with thickness [mm] less than or equal to					
	16	40	63	80	100	150
S235JR,J0,J2	235 / 360	225 / 360	215 / 360	215 / 360	215 / 360	195 / 350
S275JR,J0,J2	275 / 410	265 / 410	255 / 410	245 / 410	235 / 410	225 / 400
S355JR,J0,J2	355 / 470	345 / 470	335 / 470	325 / 470	315 / 470	295 / 450

(3) Alternative steel grade

Alternative steel grades not listed in Table 2.1 such as American standard (API, ASTM and AWS) and Japanese standard (JIS) should be in compliance with BC1:2012[4].

(4) Modulus of elasticity

The modulus of elasticity of structural steel should be taken as 210,000 [N/mm^2].

(5) Partial factor

The partial factor of the resistance of members and cross-sections should be taken as 1.00. The partial factor of the resistance of plates in bearing should be taken as 1.25.

2.2 Concrete

(1) Strength classes

The concrete strength class of floor slab should be in the range of C20/25 (LC20/22) to C60/75 (LC60/66). Other reinforced concrete members including beams, walls, and columns can follow EN 1992-1-1[5] in which C90/105 is recommended as the maximum strength class. The typical strength classes of concrete are given in Table 2.2 and Table 2.3.

Table 2.2: Typical strength classes of concrete

Strength class	C 20/25	C 25/30	C 30/37	C 35/45	C 40/50	C 45/55	C 50/60	C 55/67	C 60/75
Characteristic cylinder strength f_{ck} [N/mm^2]	20	25	30	35	40	45	50	55	60
Mean value of tensile strength f_{ctm} [N/mm^2]	2.2	2.6	2.9	3.2	3.5	3.8	4.1	4.2	4.4
Secant modulus of elasticity E_{cm} [GPa]	30	31	33	34	35	36	37	38	39

Table 2.3: Typical strength classes of lightweight concrete

Strength class	LC 20/22	LC 25/28	LC 30/33	LC 35/38	LC 40/44	LC 45/50	LC 50/55	LC 55/60	LC 60/66
Characteristic cylinder strength f_{lck} [N/mm^2]	20	25	30	35	40	45	50	55	60
Mean value of tensile strength f_{lctm} [N/mm^2]	$f_{ctm}\left(0.40+\frac{0.60\rho_{lc}}{2200}\right)$								
Secant modulus of elasticity E_{lcm} [GPa]	$E_{cm}\left(\frac{\rho_{lc}}{2200}\right)^2$								

Note: ρ_{lc} is the dry density of lightweight concrete in accordance with EN 206-1[6]

(2) Other properties

Unless otherwise given by this design guide, other concrete properties can be referred to EN 1992-1-1 for both normal weight concrete and lightweight concrete.

(3) Partial factor

The partial factor of concrete γ_c should be taken as 1.50.

2.3 Reinforcing Steel

(1) Yield strength

The characteristic yield strength of reinforcing steel f_{sk} should be limited to the range of 400 [N/mm^2] to 600 [N/mm^2] as conforming to EN 1992-1-1.

(2) Strength classes

The strength classes of reinforcing steel given in Table 2.4 can be used for the design of composite joints and composite beams with semi-rigid ends.

Table 2.4: Strength classes of reinforcing steel

Class	Characteristic yield strength f_{sk} [N/mm^2]	Ultimate/yield strength ratio	Ultimate elongation
B500B	500	≥ 1.08	5.0%
B500C	500	$\geq 1.15, < 1.35$	7.5%

(3) Modulus of elasticity

The modulus of elasticity of reinforcing steel E_s should be taken as 210,000 [N/mm^2].

(4) Partial factor

The partial factor of reinforcing steel γ_s should be taken as 1.15.

2.4 Shear Studs

(1) Mechanical characteristics and nominal dimensions

The mechanical characteristics and normal dimensions of shear studs may be referred to BS EN ISO 13918[7] and BS EN ISO 898-1[8].

(2) Weldability and welding examination

Weldability and welding examination of shear studs should be checked in accordance with BS EN ISO 14555[9].

(3) Shear resistance

The shear resistance of a headed stud P_{Rd} can be determined from:

$$P_{Rd} = \min\left(\frac{0.8 f_{hsu} \pi d_{hs}^{2}}{4\gamma_V} \; ; \; \frac{0.29 \alpha_{hs} d_{hs}^{2} \sqrt{f_{ck} E_{cm}}}{\gamma_V}\right) \tag{2.1}$$

α_{hs} is given by:

$$\alpha_{hs} = 0.2\left(\frac{h_{hs}}{d_{hs}} + 1\right) \qquad \text{for } 3 \le \frac{h_{hs}}{d_{hs}} \le 4 \tag{2.2}$$

$$\alpha_{hs} = 1 \qquad \text{for } \frac{h_{hs}}{d_{hs}} > 4 \tag{2.3}$$

where

f_{hsu}	is the ultimate strength of headed stud
d_{hs}	is the diameter of the shank of headed stud, 16 [mm] $\le d_{hs} \le$ 25 [mm]
γ_V	is the partial factor of headed stud taken as 1.25
f_{ck}	is the characteristic cylinder strength of concrete
E_{cm}	is the secant modulus of elasticity of concrete
h_{hs}	is the overall height of headed stud, 16 [mm]

(4) Alternative shear studs

Alternative shear studs not covered in this design guide can be allowed if they are in compliance with BC1:2012.

2.5 Profiled Steel Sheeting

(1) Material properties

The material properties of profiled steel sheeting may be referred to EN 1993-1-3[10].

(2) Alternative profiled steel sheeting

Alternative profiled steel sheeting not covered in this design guide can be allowed provided that they are in compliance with BC1:2012.

(3) Partial factor

The partial factor of profiled steel sheeting, $\gamma_{ps,}$ should be taken as 1.00.

2.6 Bolts

(1) Strength classes

The strength classes of bolts given in Table 2.5 can be used for the design of composite joints and composite beams with semi-rigid ends.

(2) Tensile strength

The nominal values of yield strength f_{by} and ultimate tensile strength f_{bu} for bolt classes 4.6, 4.8, 5.6, 8.8 and 10.9 are given in Table 2.5. Bolts from strength class 4.6 up to and including strength class 10.9 can be used as non-preloaded connections, whereas for preloaded connections, strength class 8.8 and 10.9 should be used.

Table 2.5: Strength classes of bolts

Strength class	4.6	4.8	5.6	8.8	10.9
Nominal value of yield strength f_{by} [N/mm^2]	240	320	300	640	900
Ultimate tensile strength f_{bu} [N/mm^2]	400	400	500	800	1000

(3) Alternative strength classes

Alternative strength classes of bolts not covered in this design guide can be allowed if they are in compliance with BC1:2012.

(4) Partial factor

The partial factor of bolts, γ_b, should be taken as 1.25.

Chapter 3 Scope of Application

3.1 General

(1) Structural type

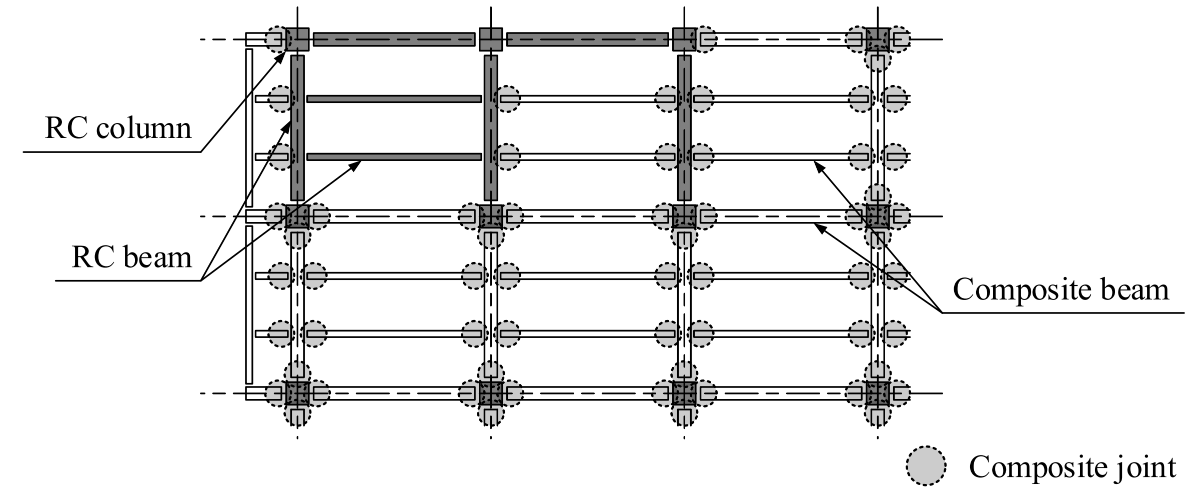
The structural type should be limited to flooring systems consisting of primary beams, secondary beams, and floor slab.

(2) Beam members

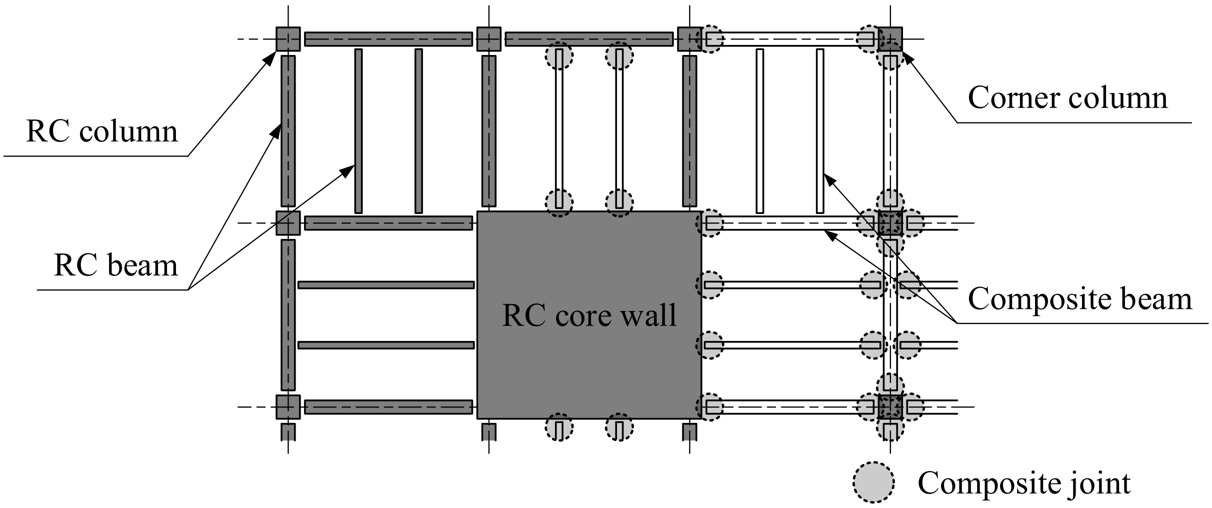
When a composite joint is applied to a double-sided beam-to-beam joint, the secondary beams should be composite beams and the primary beam should be either a composite or reinforced concrete beam. When a composite joint is applied to a beam-to-wall or beam-to-column joint, the beams should be composite beams. Except for the above cases schematized in Figure 3.1, composite joints may not be applicable unless otherwise verified by experiments or numerical analysis.

(3) Column members

Structural type of column members is not limited except when composite joints are applied to beam-to-column joints. When a composite joint with anchored reinforcing bars is applied to a beam-to-column joint as shown in Figure 1.1 (g), the column should be a reinforced concrete column.



(a) Floor plan without RC core wall



(b) Floor plan with RC core wall

Figure 3.1: Floor plan showing composite joints

Commentary:

(2) Beam members

The beam members supported by composite joints should not be subject to excessive axial force that may affect the moment and rotational behaviour of the connections. Also, they should not cause the sagging moment at their beam-ends to prevent the contact plates from separating and falling.

(3) Column members

Composite joints can be applied to a beam-to-column joint where the column member is reinforced concrete. Composite joints should not be applied for other structural types of columns such as steel columns, steel encased reinforced concrete columns, concrete filled steel tubular columns unless otherwise their structural behaviour is verified by experiments or numerical analysis.

3.2 Steel Beams

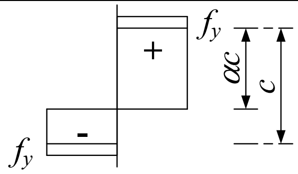
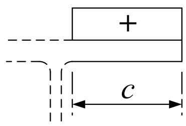
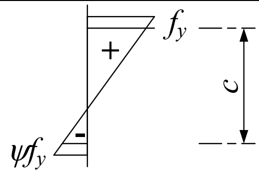
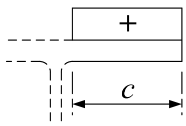
(1) Steel section

The steel section of beams with semi-rigid ends should be a beam of uniform and doubly symmetrical section.

(2) Classification of cross-section

Class 1 or 2 cross-sections where both the web and the compression flange are Class 1 or 2 based on the limiting proportions for compression parts given in Table 3.1 should be used for beams with semi-rigid ends. Effective Class 2 cross-sections where the web is Class 3 and the compression flange is Class 1 or 2 may also be used.

Table 3.1: Maximum width-to-thickness ratios for compression parts

Class	Web	Flange
Stress distribution in parts (compression positive)		
1	$\dfrac{c}{t_w} \le \dfrac{396\varepsilon}{13\alpha-1}$ for $\alpha > 0.5$ $\dfrac{c}{t_w} \le \dfrac{36\varepsilon}{\alpha}$ for $\alpha \le 0.5$	$\dfrac{c}{t_f} \le 9\varepsilon$
2	$\dfrac{c}{t_w} \le \dfrac{456\varepsilon}{13\alpha-1}$ for $\alpha > 0.5$ $\dfrac{c}{t_w} \le \dfrac{41.5\varepsilon}{\alpha}$ for $\alpha \le 0.5$	$\dfrac{c}{t_f} \le 10\varepsilon$
Stress distribution in parts (compression positive)		
3	$\dfrac{c}{t_w} \le \dfrac{42\varepsilon}{0.67+0.33\psi}$ for $\psi > -1$ $\dfrac{c}{t_w} \le 62\varepsilon(1-\psi)\sqrt{(-\psi)}$ for $\psi \le -1$ *)	$\dfrac{c}{t_f} \le 14\varepsilon$

$\varepsilon = \sqrt{\dfrac{235}{f_{ay}}}$	f_{ay}	235	275	355
	ε	1.00	0.92	0.81

*) $\psi \le -1$ applies where either the compression stress $\sigma \le f_{ay}$ or the tensile strain $\varepsilon_y > f_{ay}/E_a$

Commentary:

(2) Classification of cross-section

As described in **Chapter 4**, the plastic moment resistances of beams with semi-rigid ends are checked at ultimate limit state. Therefore, Class 1 or 2 cross-sections which can develop their plastic moment resistance without local buckling of web and flange are required.

3.3 Floor Slab

(1) Slab arrangement

Floor slab should be arranged on top of steel beams and connected to the top flange of steel beams through shear studs.

(2) Slab type

Floor slab should be reinforced concrete slab or composite slab with profiled steel sheeting.

(3) Reinforcing bars

Longitudinal reinforcing bars in concrete slab should be continuous or anchored at beam-to-beam, beam-to-wall, and beam-to-column joints. Transverse reinforcing bars are also required to provide appropriate stress distribution on the longitudinal reinforcements within the effective width. The diameter of reinforcing bars utilized as a structural component of composite joints should be less than or equal to 16 [mm] unless the structural resistance and serviceability of the joints have been verified by experimental or numerical evidence.

(4) Slab span

The maximum slab span should be in accordance with the allowable span of profiled steel sheeting during construction considering ultimate and serviceability limit states.

Commentary:

(2) Slab type

Beam-end joints should be designed as nominally pinned joint when floor slab other than reinforced concrete slab or composite slab is used, in which case rotational restraint may not be adequately developed at the joints.

(3) Reinforcing bars

Beam-end joints should be designed as nominally pinned joint when the longitudinal reinforcing bars in the slab are not continuous or not properly anchored at beam-to-beam, beam-to-wall, and beam-to-column joints, in which case sufficient rotational restraint may not be developed at the joints.

3.4 Composite Joints

(1) Joint types

Composite joints should be beam-to-beam, beam-to-wall, or beam-to-column joints, and the joints should be the extended fin plate bolted type. In addition, the following should be noted for each joint detail as well.

a) Beam-to-beam joint with primary composite beam

This joint type refers to a primary composite beam which is double-side connected by secondary beams. When the depth of at least one secondary beam is same as that of primary beam as shown in Figure 3.2 (a), the bottom flange of primary beam should be designed against bi-axial force due to the additional compression from the contact plates. When the secondary beams have different depths as shown in Figure 3.2 (b), continuous fin plates should be provided between the top and bottom flanges to prevent out-of-plane deformation of the primary beam web. The shear resistance of the panel zone formed by the fin plates, stiffeners, and bottom flanges should be checked against actual shear force caused by the eccentricity of the bottom flanges of secondary beams.

b) Beam-to-beam joint with primary reinforced concrete beam

This joint type refers to a primary reinforced concrete beam which is double-side or single-side connected by secondary steel beam. The torsional moment from the secondary beam should also be taken into consideration for the primary beam design. As shown in Figure 3.3, fin plates should be welded to the end plates fixed to the primary beam by appropriate anchorages to transfer the shear force from the secondary beams.

c) Beam-to-wall joint with reinforced concrete wall

The joints may be either double-side or single-side connected. For single-side connected joint, the moment due to eccentricity from the beam reaction force should be taken into consideration for the wall design. As shown in Figure 3.4, fin plates should be welded or bolted to the end plates fixed to the reinforced concrete wall by appropriate anchorages to transfer the shear force from the beams.

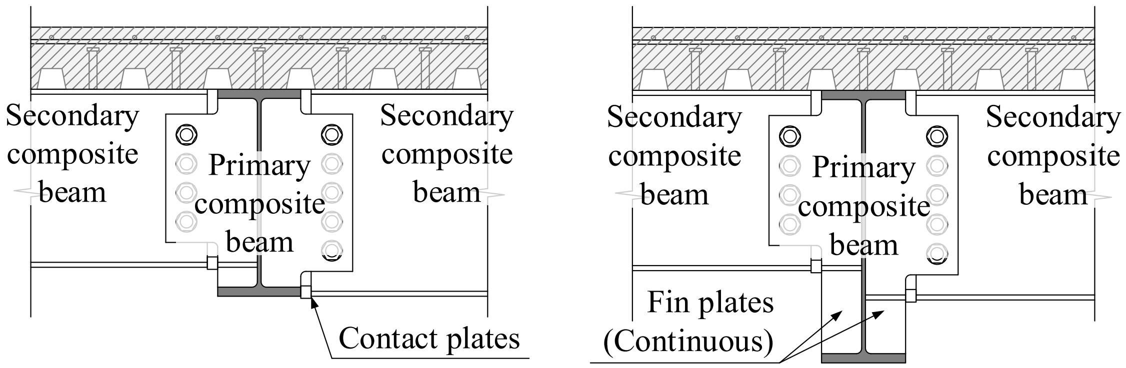
d) Beam-to-column joint with reinforced concrete column

For single-sided joints, the moment arising from the eccentricity of the beam reaction relative to the column centroidal axis should be taken into account in the column design.

As shown in Figure 3.5, fin plates should be welded to end plates, which are fixed to the reinforced concrete column by suitable anchorages, in order to transfer the beam shear forces into the column.

(2) Contact plates and stiffeners

As shown in Figures 3.2 to 3.5, contact plates and stiffeners may be attached at the bottom flange level of steel beams to enhance the rotational stiffness and moment resistance of the joints.



(a) Beams of same depth (b) Beams of different depth

Figure 3.2: Beam-to-beam joint with primary composite beam

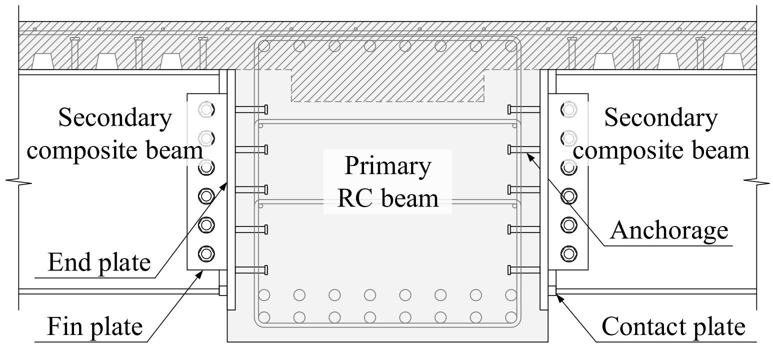


Figure 3.3: Beam-to-beam joint with primary reinforced concrete beam

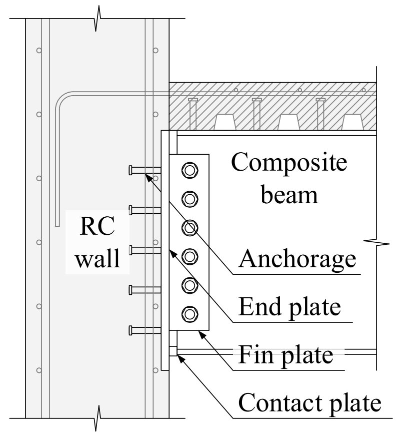
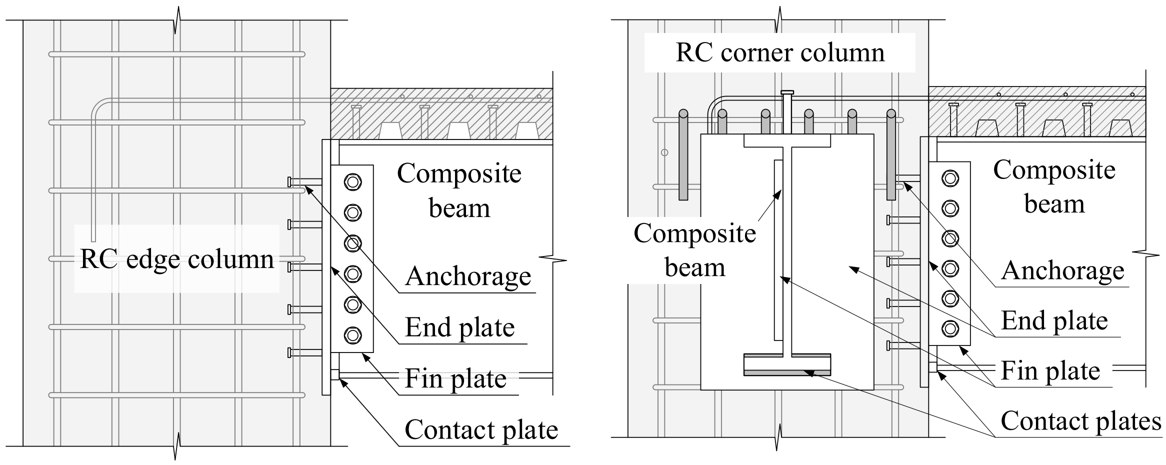


Figure 3.4: Beam-to-wall joint with reinforced concrete wall



(a) Uniaxial connected (b) Biaxial connected

Figure 3.5: Beam-to-column joint with reinforced concrete column

Commentary:

(1) Joint type

The single-sided beam-to-beam joint where a secondary composite beam is attached from one side of a primary composite beam is encountered in the outer periphery and around the voids as shown in Figure 3.6. They should be designed as nominally pinned joints unless special measures are taken to ensure proper anchorage of the reinforcing bars and the torsional actions on primary beams are properly considered in structural design.

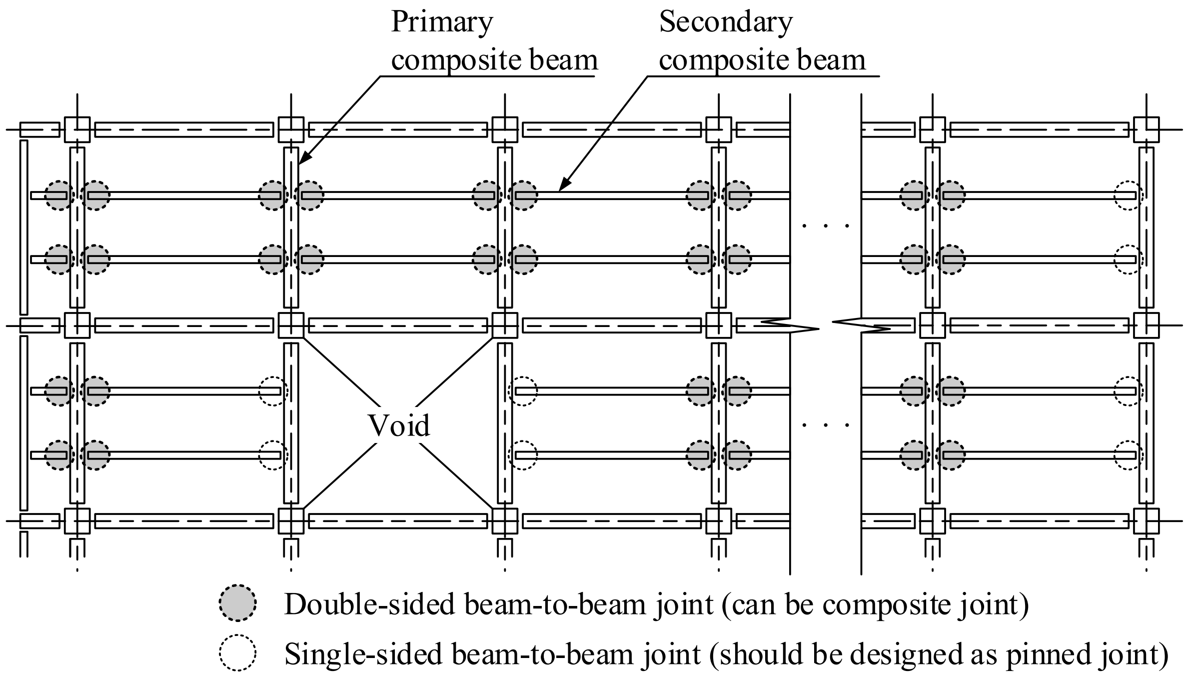


Figure 3.6: Floor beam layout with composite beams

Chapter 4 Design of Composite Joint and Beam

4.1 General

(1) Basis of design

The design of composite joints and composite beams with semi-rigid ends should be in accordance with the basis of design given in EN1994-1-1. This chapter provides supplementary provisions such as design criteria, and structural properties of composite joints and beams, which should also be applied in structural design.

(2) Design procedure

The design of composite joints and composite beams with semi-rigid ends should follow the design flow chart shown in Figure 4.1. Refer to each clause for the details.

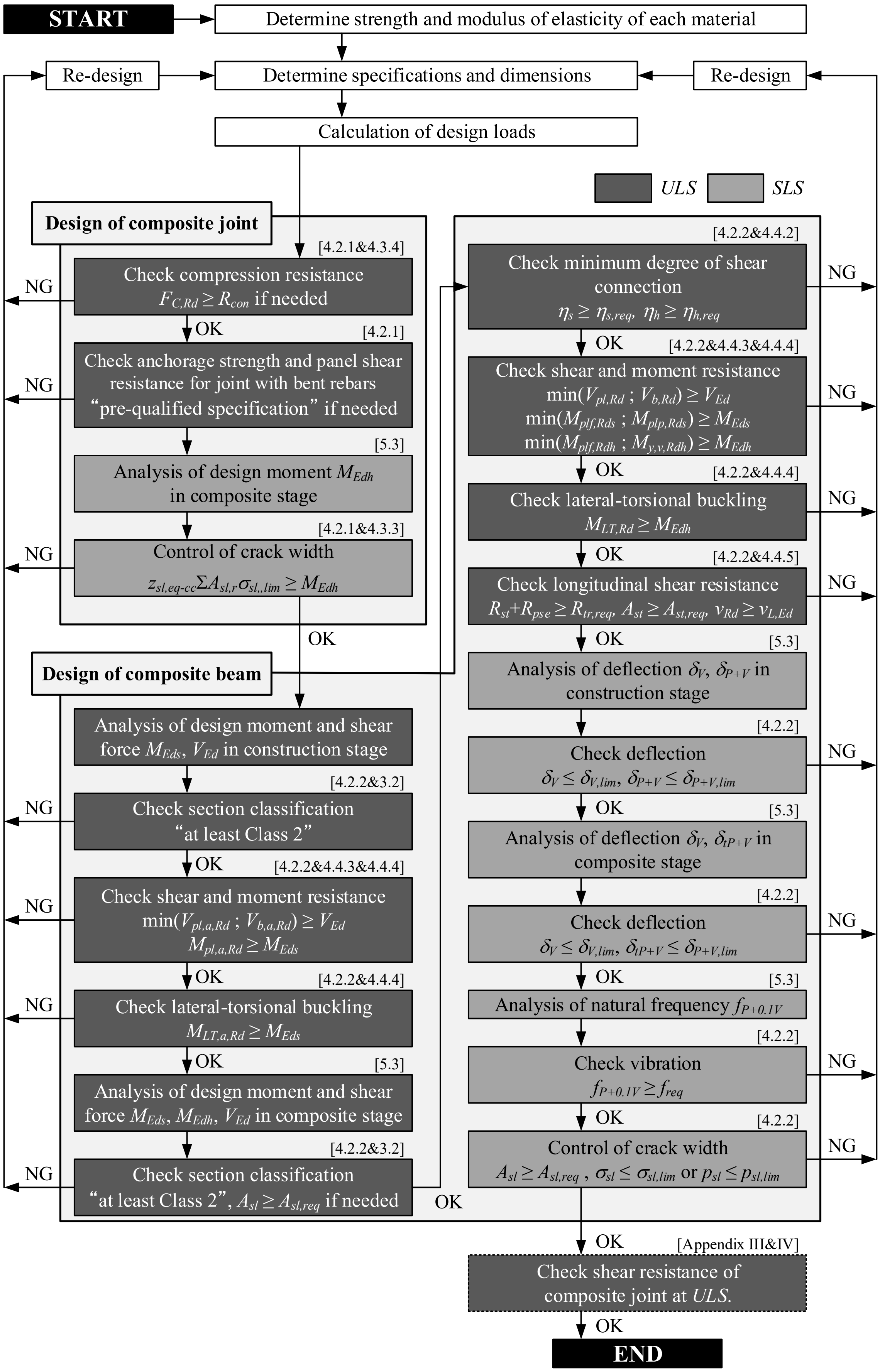


Figure 4.1: Design flowchart for composite joint and composite beam

Commentary:

(2) Design procedure

The following design steps should be performed in accordance with the flow chart shown in Figure 4.1.

a) Design of composite joint

Based on the design criteria in subsection 4.2.1, joint classification should be conducted to check if the composite joints are classified as semi-rigid. The initial rotational stiffness and moment resistance can be determined according to subsection 4.3.2 and 4.3.3, respectively. Structural resistances of concrete in composite stage should be checked in accordance with EN 1992-1-1. In the case of beam-to-wall and beam-to-column composite joints with bent reinforcing bars (anchored reinforcing bars), the joint details should comply with the pre-qualified specifications described in subsection 4.2.1. In addition, serviceability check should also be carried out. The design hogging moments at the beam ends can be obtained by carrying out the analysis of composite beam with end restraints in accordance with Section 5.3.

b) Design of composite beam with semi-rigid ends

Based on the design criteria in subsection 4.2.2, structural resistances at ultimate limit state should be checked. The degree of shear connection, shear resistance, moment resistance, and the longitudinal shear resistance of composite beams can be determined according to subsection 4.4.2, 4.4.3, 4.4.4, and 4.4.5, respectively. In addition, serviceability checks should also be carried out. The design moment and deflection of the beams can be obtained by the structural analysis described in Section 5.3.

4.2 Design Criteria

4.2.1 Composite Joint

(1) Joint classification

Composite joints with any level of stiffness and resistance may be treated as semi-rigid joints. For simplicity in design, however, composite joints may be idealised as pinned or rigid if the following conditions on initial rotational stiffness and moment resistance are satisfied.

a) Initial rotational stiffness

for pin joints

$$S_{j,ini} \leq \frac{0.5E_a I_b}{L_b} \tag{4.1a}$$

for rigid joints

$$S_{j,ini} \geq \frac{25E_a I_b}{L_b} \tag{4.1b}$$

I_b is given by:

$$I_b = \frac{A_a\left(h_{cs}+2D_{ps}+D_a\right)^2}{4\left(1+\frac{2E_a}{E_{cm,cs}}\frac{A_a}{b_{eff,b}h_{cs}}\right)}+\frac{b_{eff,b}h_{cs}{}^3}{12\left(\frac{2E_a}{E_{cm,cs}}\right)}+I_{ay} \tag{4.2}$$

where

$S_{j,ini}$	is the initial rotational stiffness of composite joint, see 4.3.2
E_a	is the modulus of elasticity of steel beam
L_b	is the beam length
I_b	is the second moment of area of composite beam
A_a	is the cross-sectional area of steel beam
h_{cs}	is the thickness of composite slab above profiled steel sheeting
D_{ps}	is the overall depth of profiled steel sheeting
D_a	is the depth of steel beam
$E_{cm,cs}$	is the secant modulus of elasticity of concrete for concrete slab
$b_{eff,b}$	is the effective width of composite beam assuming simply supported condition
I_{ay}	is the second moment of area of steel beam about major axis (y-y axis)

b) Moment resistance

for pin joints

$$M_{j,Rd} \leq 0.25M_{pl,Rd} \tag{4.3}$$

for rigid joints

$$M_{j,Rd} \geq M_{pl,Rd} \tag{4.3}$$

where

$M_{j,Rd}$ is the moment resistance of composite joint, see 4.3.3

$M_{pl,Rd}$ is the plastic moment resistance of composite beam

(2) Structural resistance check of concrete in composite stage

a) Compression resistance

When the compression force from the bottom flange of steel beam is transferred to concrete through the end plate, the following condition for compression resistance of equivalent T-stub flange should be satisfied to prevent the concrete bearing failure.

$$F_{C,Rd} \geq R_{con} \tag{4.4}$$

where

$F_{C,Rd}$ is the compression resistance of equivalent T-stub flange, see 4.3.3

R_{con} is the compression resistance of contact part, see 4.3.3

In the case of composite joints with thin wall, the punching shear resistance of concrete wall, referred to EN 1992-1-1 for slab design, should be checked as well.

b) Anchorage strength and panel shear resistance

In the case of composite joints with bent reinforcing bars (anchored reinforcing bars) utilized as a component of the joints, the joint details should comply with the pre-qualified specifications to prevent the anchorage failure and panel shear failure of supporting members. The pre-qualified specifications are shown in Table 4.1 and Table 4.2 in which the symbols correspond to those in Figure 4.2. Note that they are applicable only when the following conditions are satisfied.

- The arrangement width of bent reinforcing bars does not exceed 1200 [mm].
- The depth of the adjacent beam does not exceed 700 [mm].
- The end plate width is more than or equal to 300 [mm].
- Dowel bars with a diameter of 13 [mm] and a length of at least the arrangement width are placed with appropriate anchorage (Links or shear reinforcement may be utilized).
- For beam-to-wall joints, successive crossties with a diameter of 13 [mm] and a pitch of 300 [mm] in vertical and horizontal are arranged throughout the wall including joint panel.
- Table 4.2 is applicable for either uniaxially or biaxially connected.

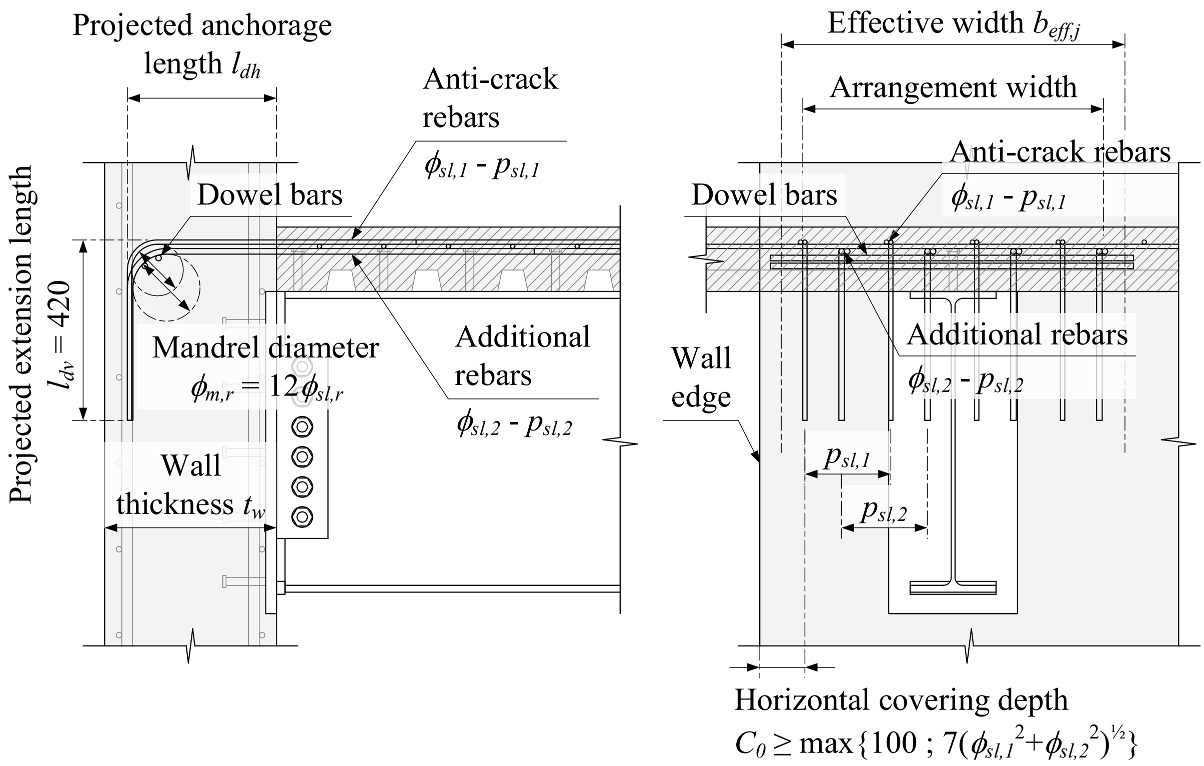
(3) Serviceability check in composite stage

The following condition for joint moment at serviceability limit state should be satisfied to control the crack width of floor slab.

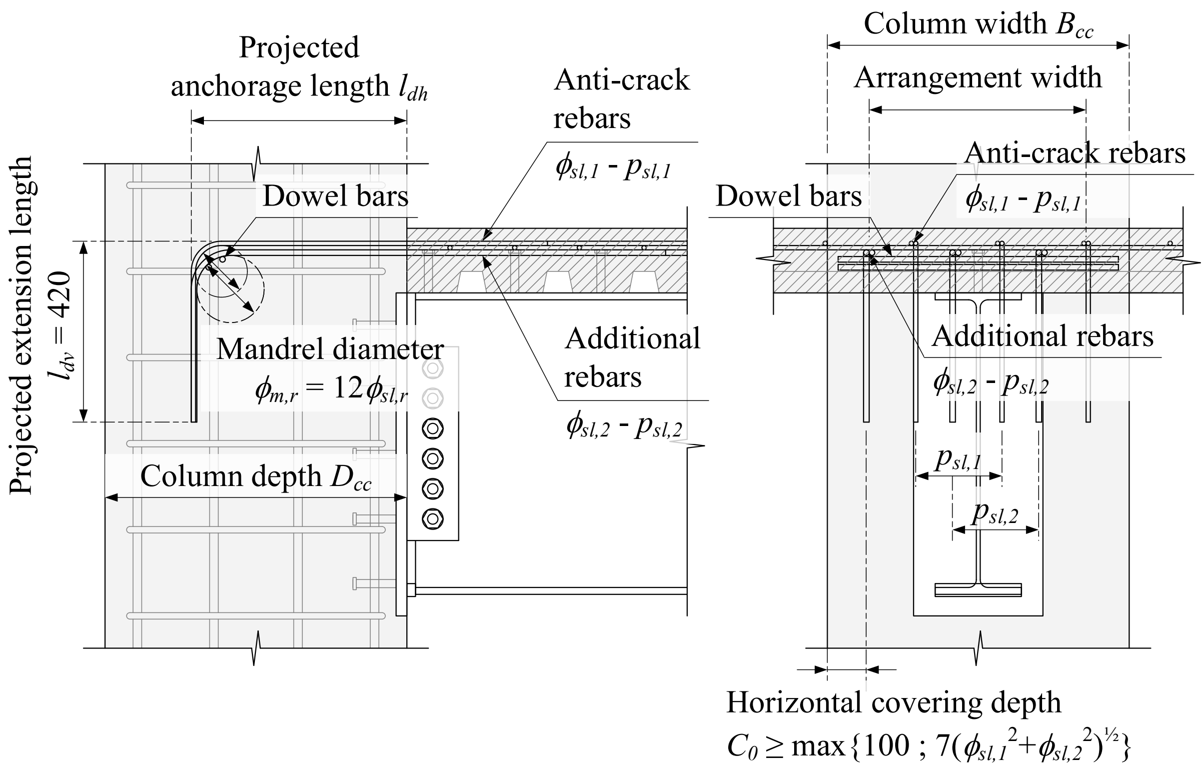
$$z_{sl,eq\text{-}cc} \sum A_{sl,r} \sigma_{sl,lim} \geq M_{Edh} \tag{4.5}$$

where

$z_{sl,eq\text{-}cc}$	is the equivalent vertical distance between longitudinal reinforcing bars and centre of contact part
$A_{sl,r}$	is the cross-sectional area of longitudinal reinforcing bars within $b_{eff,j}$ for a row r
$\sigma_{sl,lim}$	is the limit of stress permitted in longitudinal reinforcing bars immediately after cracking, taken as the larger value given in Table 4.3 and 4.4
M_{Edh}	is the design hogging moment



(a) Beam-to-wall composite joint



(b) Beam-to-column composite joint

Figure 4.2: Dimensions specified in pre-qualified specifications

Table 4.1: Pre-qualified specifications for **beam-to-wall composite joints** with bent reinforcing bars

<table>
<tr><th colspan="4">Characteristic cylinder strength of reinforced concrete wall
40 [N/mm^2] $\leq f_{ck,cw}$ < 45 [N/mm^2]</th></tr>
<tr><th rowspan="2">Diameter and pitch of anti-crack reinforcing bars
$\phi_{sl,1}$ - $p_{sl,1}$ [mm]</th><th colspan="3">Diameter and pitch of additional reinforcing bars $\phi_{sl,2}$ - $p_{sl,2}$ [mm]
for wall thickness t_{cw} [mm]</th></tr>
<tr><th>$250 \leq t_{cw}$
(l_{dh} = 200 [mm])</th><th>$300 \leq t_{cw}$
(l_{dh} = 250 [mm])</th><th>$400 \leq t_{cw}$
(l_{dh} = 350 [mm])</th></tr>
<tr><td>Additional rebars only</td><td></td><td></td><td>10 - 100 or more
13 - 150 or more
16 - 200 or more</td></tr>
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</table>

Table 4.1: Pre-qualified specifications for **beam-to-wall composite joints** with bent reinforcing bars (cont'd)

<table>
<tr><th colspan="4">Characteristic cylinder strength of reinforced concrete wall
45 [N/mm^2] $\leq f_{ck,cw} <$ 50 [N/mm^2]</th></tr>
<tr><th rowspan="2">Diameter and pitch of anti-crack reinforcing bars
$\phi_{sl,1}$ - $p_{sl,1}$ [mm]</th><th colspan="3">Diameter and pitch of additional reinforcing bars $\phi_{sl,2}$ - $p_{sl,2}$ [mm]
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</table>

Table 4.1: Pre-qualified specifications for **beam-to-wall composite joints** with bent reinforcing bars (cont'd)

<table>
<tr><th colspan="4">Characteristic cylinder strength of reinforced concrete wall
50 [N/mm^2] $\leq f_{ck,cw} <$ 55 [N/mm^2]</th></tr>
<tr><th rowspan="2">Diameter and pitch of anti-crack reinforcing bars
$\phi_{sl,1}$ - $p_{sl,1}$ [mm]</th><th colspan="3">Diameter and pitch of additional reinforcing bars $\phi_{sl,2}$ - $p_{sl,2}$ [mm]
for wall thickness t_{cw} [mm]</th></tr>
<tr><th>250 $\leq t_{cw}$
(l_{dh} = 200 [mm])</th><th>300 $\leq t_{cw}$
(l_{dh} = 250 [mm])</th><th>400 $\leq t_{cw}$
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Table 4.1: Pre-qualified specifications for **beam-to-wall composite joints** with bent reinforcing bars (cont'd)

<table>
<tr><th colspan="4">Characteristic cylinder strength of reinforced concrete wall
55 [N/mm^2] $\leq f_{ck,cw} <$ 60 [N/mm^2]</th></tr>
<tr><th rowspan="2">Diameter and pitch of anti-crack reinforcing bars
$\phi_{sl,1}$ - $p_{sl,1}$ [mm]</th><th colspan="3">Diameter and pitch of additional reinforcing bars $\phi_{sl,2}$ - $p_{sl,2}$ [mm]
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Table 4.1: Pre-qualified specifications for **beam-to-wall composite joints** with bent reinforcing bars (cont'd)

<table>
<tr><th colspan="4">Characteristic cylinder strength of reinforced concrete wall
60 [N/mm^2] $\leq f_{ck,cw}$</th></tr>
<tr><th rowspan="2">Diameter and pitch of anti-crack reinforcing bars
$\phi_{sl,1}$ - $p_{sl,1}$ [mm]</th><th colspan="3">Diameter and pitch of additional reinforcing bars $\phi_{sl,2}$ - $p_{sl,2}$ [mm]
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Table 4.2: Pre-qualified specifications for **beam-to-column composite joints** with bent reinforcing bars

<table>
<tr><td colspan="4">Characteristic cylinder strength of reinforced concrete column
40 [N/mm^2] $\leq f_{ck,cc} <$ 45 [N/mm^2]</td></tr>
<tr><td rowspan="2">Diameter and pitch of anti-crack reinforcing bars
$\phi_{sl,1}$ - $p_{sl,1}$ [mm]</td><td colspan="3">Diameter and pitch of additional reinforcing bars $\phi_{sl,2}$ - $p_{sl,2}$ [mm]
for column depth D_{cc} [mm]</td></tr>
<tr><td>$300 \leq D_{cc}$
($l_{dh} = 2/3D_{cc}$)</td><td>$375 \leq D_{cc}$
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</table>

Table 4.2: Pre-qualified specifications for **beam-to-column composite joints** with bent reinforcing bars (cont'd)

Characteristic cylinder strength of reinforced concrete column 45 [N/mm^2] $\le f_{ck,cc} <$ 50 [N/mm^2]			
Diameter and pitch of anti-crack reinforcing bars $\phi_{sl,1}$ - $p_{sl,1}$ [mm]	Diameter and pitch of additional reinforcing bars $\phi_{sl,2}$ - $p_{sl,2}$ [mm] for column depth D_{cc} [mm]		
	300 $\le D_{cc}$ (l_{dh} = 2/3D_{cc})	375 $\le D_{cc}$ (l_{dh} = 2/3D_{cc})	525 $\le D_{cc}$ (l_{dh} = 2/3D_{cc})
Additional rebars only		10 - 100 or more 13 - 150 or more	10 - 100 or more 13 - 100 or more 16 - 150 or more
10 - 200 or more		Anti-crack rebars only 10 - 150 or more 13 - 200 or more	Anti-crack rebars only 10 - 100 or more 13 - 100 or more 16 - 200 or more
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Table 4.2: Pre-qualified specifications for **beam-to-column composite joints** with bent reinforcing bars (cont'd)

<table>
<tr><th colspan="4">Characteristic cylinder strength of reinforced concrete column
50 [N/mm^2] $\leq f_{ck,cc} <$ 55 [N/mm^2]</th></tr>
<tr><th rowspan="2">Diameter and pitch of anti-crack reinforcing bars
$\phi_{sl,1}$ - $p_{sl,1}$ [mm]</th><th colspan="3">Diameter and pitch of additional reinforcing bars $\phi_{sl,2}$ - $p_{sl,2}$ [mm]
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Table 4.2: Pre-qualified specifications for **beam-to-column composite joints** with bent reinforcing bars (cont'd)

<table>
<tr><th colspan="4">Characteristic cylinder strength of reinforced concrete column
55 [N/mm^2] $\leq f_{ck,cc}$ < 60 [N/mm^2]</th></tr>
<tr><th rowspan="2">Diameter and pitch of anti-crack reinforcing bars
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<tr><td>13 - 100 or more</td><td></td><td></td><td>Anti-crack rebars only
10 - 150 or more</td></tr>
<tr><td>16 - 200 or more</td><td></td><td>Anti-crack rebars only</td><td>Anti-crack rebars only
10 - 100 or more
13 - 150 or more
16 - 200 or more</td></tr>
<tr><td>16 - 150 or more</td><td></td><td></td><td>Anti-crack rebars only
10 - 150 or more</td></tr>
<tr><td>16 - 100 or more</td><td></td><td></td><td></td></tr>
</table>

Table 4.2: Pre-qualified specifications for **beam-to-column composite joints** with bent reinforcing bars (cont'd)

<table>
<tr><th colspan="4">Characteristic cylinder strength of reinforced concrete column
60 [N/mm^2] $\leq f_{ck,cc}$</th></tr>
<tr><th rowspan="2">Diameter and pitch of anti-crack reinforcing bars
$\phi_{sl,1}$ - $p_{sl,1}$ [mm]</th><th colspan="3">Diameter and pitch of additional reinforcing bars $\phi_{sl,2}$ - $p_{sl,2}$ [mm]
for column depth D_{cc} [mm]</th></tr>
<tr><th>$300 \leq D_{cc}$
($l_{dh} = 2/3D_{cc}$)</th><th>$375 \leq D_{cc}$
($l_{dh} = 2/3D_{cc}$)</th><th>$525 \leq D_{cc}$
($l_{dh} = 2/3D_{cc}$)</th></tr>
<tr><td>Additional rebars only</td><td>10 - 100 or more
13 - 150 or more</td><td>10 - 100 or more
13 - 150 or more
16 - 200 or more</td><td>10 - 100 or more
13 - 100 or more
16 - 150 or more</td></tr>
<tr><td>10 - 200 or more</td><td>Anti-crack rebars only
10 - 150 or more
13 - 200 or more</td><td>Anti-crack rebars only
10 - 100 or more
13 - 150 or more
16 - 200 or more</td><td rowspan="2">Anti-crack rebars only
10 - 100 or more
13 - 100 or more
16 - 150 or more</td></tr>
<tr><td>10 - 150 or more</td><td>Anti-crack rebars only
10 - 200 or more</td><td>Anti-crack rebars only
10 - 150 or more
13 - 200 or more</td></tr>
<tr><td>10 - 100 or more</td><td>Anti-crack rebars only</td><td>Anti-crack rebars only
10 - 200 or more</td><td>Anti-crack rebars only
10 - 100 or more
13 - 150 or more
16 - 200 or more</td></tr>
<tr><td>13 - 200 or more</td><td>Anti-crack rebars only
10 - 200 or more</td><td>Anti-crack rebars only
10 - 150 or more
13 - 200 or more</td><td>Anti-crack rebars only
10 - 100 or more
13 - 100 or more
16 - 150 or more</td></tr>
<tr><td>13 - 150 or more</td><td>Anti-crack rebars only</td><td>Anti-crack rebars only
10 - 200 or more</td><td>Anti-crack rebars only
10 - 100 or more
13 - 150 or more
16 - 200 or more</td></tr>
<tr><td>13 - 100 or more</td><td></td><td></td><td>Anti-crack rebars only
10 - 150 or more
13 - 200 or more</td></tr>
<tr><td>16 - 200 or more</td><td></td><td>Anti-crack rebars only
10 - 200 or more</td><td>Anti-crack rebars only
10 - 100 or more
13 - 150 or more
16 - 200 or more</td></tr>
<tr><td>16 - 150 or more</td><td></td><td></td><td>Anti-crack rebars only
10 - 150 or more
13 - 200 or more</td></tr>
<tr><td>16 - 100 or more</td><td></td><td></td><td></td></tr>
</table>

Table 4.3: Limit of stress permitted in longitudinal reinforcing bars based on diameters

Limit of stress $\sigma_{sl,lim}$ [N/mm^2]	Diameter of longitudinal reinforcing bars ϕ_{sl}^* [mm] for design crack width w_k		
	w_k = 0.4 [mm]	w_k = 0.3 [mm]	w_k = 0.2 [mm]
160	40	32	25
200	32	25	16
240	20	16	12
280	16	12	8
320	12	10	6
360	10	8	5
400	8	6	4
450	6	5	-

Table 4.4: Limit of stress permitted in longitudinal reinforcing bars based on spacing

Limit of stress $\sigma_{sl,lim}$ [N/mm^2]	Spacing of longitudinal reinforcing bars p_{sl} [mm] for design crack width w_k		
	w_k = 0.4 [mm]	w_k = 0.3 [mm]	w_k = 0.2 [mm]
160	300	300	200
200	300	250	150
240	250	200	100
280	200	150	50
320	150	100	-
360	100	50	-

Commentary:

(1) Joint classification

Various definitions of semi-rigid joints are used in structural analysis. One of the principal classification schemes is given in [EN 1993-1-8]. According to EN 1993-1-8, joints are classified as rigid, nominally pinned, or semi-rigid by comparing the initial rotational stiffness of the joint, $S_{j,ini}$, with the classification limits shown in Figure 4.3. Here, $S_{j,ini}$ is defined as the bending moment per unit rotation, and the stiffness boundaries are related to the flexural stiffness of the adjacent beam member, $(EI/L)_b$.

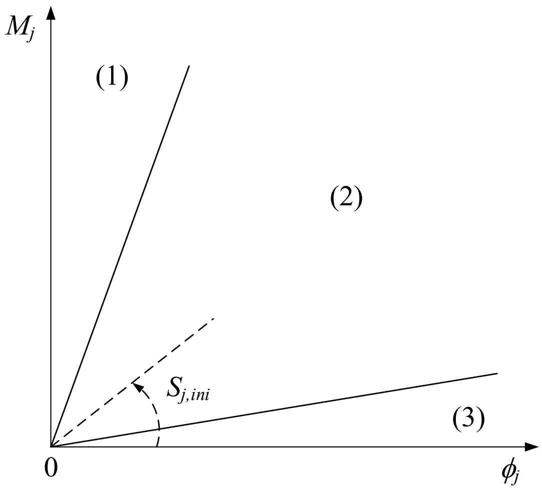
In Figure 4.3, joints that are neither rigid nor nominally pinned—i.e. those with initial rotational stiffness lower than the rigid boundary but higher than the nominally pinned boundary—are defined as semi-rigid. All joints in zone (2) should be classified as semi-rigid. Joints in zones (1) and (3) may, if desired, also be treated as semi-rigid.

Joints are also classified as full-strength, partial-strength, or nominally pinned by comparing the joint moment resistance, $M_{j,Rd}$, with the plastic moment resistance of the adjacent beam member, $M_{pl,Rd}$. If $M_{j,Rd} \geq M_{pl,Rd}$, the joint is classified as full-strength; if $M_{j,Rd} < 0.25\, M_{pl,Rd}$, it is classified as nominally pinned; otherwise, it is classified as partial-strength.

In this design guide, composite beams are to be designed with semi-rigid end conditions. Accordingly, composite joints are required to satisfy appropriate conditions on initial rotational stiffness and moment resistance, based on the following joint classification scheme.

$$S_{j,ini} \geqslant \frac{0.5 E_a I_b}{L_b} \tag{4.6}$$

$$M_{j,Rd} \geq 0.25 M_{pl,Rd} \tag{4.7}$$



(1) rigid, if $S_{j,ini} \geq k_b(EI/L)_b$

(2) semi-rigid

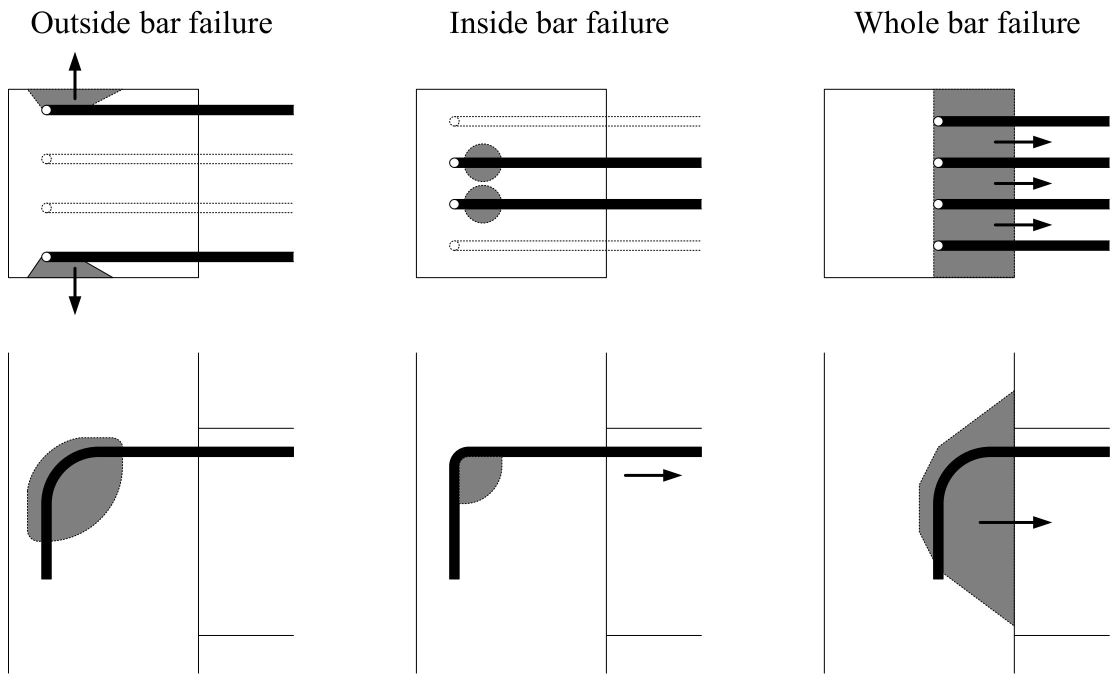
(3) nominally pinned, if $S_{j,ini} \leq 0.5(EI/L)_b$

Figure 4.3: Joint classification scheme in EN 1993-1-8
(k_b = 8 for braced frame and k_b = 25 for unbraced frame)

(2) Structural resistance check of concrete in composite stage

Elastic-plastic global analysis in which the joints are allowed to behave as plastic hinges is recommended in this design guide, hence the moment resistance of composite joints may not be checked at ultimate limit state. However, in the case of composite joints with bent reinforcing bars (anchored reinforcing bars), it should be checked that the anchorage failure and panel shear failure of supporting members do not occur until the joints exhibit sufficient rotational capacity due to the tensile elongation of the reinforcing bars.

According to design guidelines for earthquake resistant reinforced concrete building based on inelastic displacement concept[11], anchorage failure using bent reinforcing bars can be classified into three failure modes, side split failure, local compression failure, and raking-out failure as shown in Figure 4.4. Although the anchorage strength equations for each failure mode are proposed based on previous studies[12-15], many of them are meant for beam-to-column joints in reinforced concrete structures. Therefore, their applicability to composite joints in which bending moment can be transferred by slab reinforcement and contact parts has not been fully verified. In this design guide, therefore, in the case of beam-to-wall or beam-to-column composite joints with bent reinforcing bars, the joint details should comply with the pre-qualified specifications which have been experimentally proven that the anchorage failure and panel shear failure of supporting members do not occur until the joints exhibit sufficient rotational capacity[16,17]. As reference information, the calculation procedures for the anchorage strength and panel shear resistance of supporting members are provided in **Appendix I**. However, it is preferable to obtain additional experimental or numerical evidence to use the joint details other than the pre-qualified specifications as the accuracy of those equations have not been confirmed under arbitrary design conditions.



(a) Side split failure (b) Local compression failure (c) Raking-out failure

Figure 4.4: Anchorage failure mode using bent reinforcing bars

In structural design, anti-crack reinforcing bars arranged over the entire surface of floor slab can be utilized as a structural component of composite joints if they are appropriately anchored to supporting members. Also, additional reinforcing bars can be arranged with the anti-crack reinforcing bars to improve the structural performance of the composite joints. The applicable combinations of these reinforcing bars to prevent the anchorage failure and panel shear failure at composite joints mainly depend on wall thickness, column depth, and concrete strength. In this design guide, the conservative combinations of the bent reinforcing bars based on the practical design conditions are summarized as the pre-qualified specifications in Table 4.1 and Table 4.2. Since they are based on experimental evidence, they are not applicable under more severe conditions than those of the specimens used in the experimental works. Therefore, some conditions on arrangement width of bent reinforcing bars, depth of the adjacent beam, end plate width, dowel bars, and successive crossties shall be satisfied to use the pre-qualified specifications.

(3) Serviceability check in composite stage

When composite joints are classified as semi-rigid joints, cracking on floor slab is a concern since the joints are subjected to the hogging moment. As for the cracking of the concrete, the design requirements may depend on the appearance of structures, but the limiting value of crack width of 0.3 [mm] is recommended for reinforced members in accordance with EN 1992-1-1 in terms of the functioning and durability. Crack width is related to the tensile stress in reinforcing bars, therefore design criteria on the longitudinal reinforcing bars to control the crack width on the floor slab needs to be imposed on the design of composite joints. According to EN 1994-1-1 Clause 7.4.1 (3), the design crack width may be achieved by ensuring bar diameters or spacing not exceeding the limits defined in Table 4.3 and 4.4, which has also been verified by the full-scale joint component tests[18]. In this method, the limit of stress permitted in longitudinal reinforcing bars is determined only based on the bar diameters and bar spacing. Therefore, composite joints should satisfy the following condition for the joint moment.

$$z_{sl,eq\text{-}cc} \sum A_{sl,r}\, \sigma_{sl,lim} \geq M_{Edh} \tag{4.8}$$

4.2.2 Composite Beam with Semi-rigid Ends

(1) Structural resistance check in construction stage

Unless floor slab and beams are propped during constructions, the following conditions for classification of cross-section, shear resistance, and moment resistance at ultimate limit state should be satisfied as steel beams with simply supported ends as composite action cannot be developed until the concrete of floor slab has hardened.

a) Classification of cross-section

The classification of cross-section of steel beam should be at least Class 2.

b) Shear resistance

$$\min\left(V_{pl,a,Rd}\ ;\ V_{b,a,Rd}\right) \geq V_{Ed} \tag{4.9}$$

where

$V_{pl,a,Rd}$	is the plastic shear resistance of steel beam, see 4.4.3
$V_{b,a,Rd}$	is the shear buckling resistance of steel beam, see 4.4.3
V_{Ed}	is the design shear force

c) Moment resistance

$$M_{pl,a,Rd} \geq M_{Eds} \tag{4.10}$$

where

$M_{pl,a,Rd}$	is the plastic moment resistance of steel beam, see 4.4.4
M_{Eds}	is the design sagging moment

d) Lateral-torsional buckling moment resistance

$$M_{LT,a,Rd} \geq M_{Eds} \tag{4.11}$$

where

$M_{LT,a,Rd}$	is the buckling moment resistance of laterally unrestrained steel beam, see 4.4.4

(2) Structural resistance check in composite stage

Regardless of propped or un-propped constructions, the following conditions for classification of cross-section, degree of shear connection, shear resistance, moment resistance, lateral torsional buckling, and longitudinal shear resistance at ultimate limit state should be satisfied as composite beams with semi-rigid ends as composite action can be considered after the concrete of floor slab has hardened.

a) Classification of cross-section

The classification of cross-section of composite beam should be at least Class 2. If the longitudinal reinforcing bars in concrete slab are in tension and the elastic hogging moment resistance of composite beam $M_{el,Rdh}$ is less than the moment resistance of adjacent composite joints $M_{j,Rd}$, the following additional condition should be satisfied.

$$A_{sl} \geq A_{sl,req} \tag{4.12}$$

$A_{sl,req}$ is given by:

$$A_{sl,req} = \rho_{sl,req} A_{cs} \tag{4.13}$$

where

$$\rho_{sl,req} = \delta \frac{f_{ay}}{235} \frac{f_{ctm}}{f_{sk}} \sqrt{k_c} \tag{4.14}$$

$$k_c = \min \left\{ \frac{1}{1+\left(\frac{h_{cs}}{2z_0}\right)} + 0.3 \; ; \; 1.0 \right\} \tag{4.15}$$

$$z_0 = \frac{(A_a + A_{sl})\left(0.5D_a + D_{ps} + 0.5h_{cs}\right)}{A_a + A_{sl} + \left(\frac{h_{cs} b_{effh}}{n_0}\right)} \tag{4.16}$$

$$n_0 = \frac{E_a}{E_{cm,cs}} \tag{4.17}$$

A_{sl}	is the cross-sectional area of longitudinal reinforcing bars within b_{effh}
b_{effh}	is the effective width of composite beam in hogging moment region, see 4.4.1
$A_{sl,req}$	is the required minimum cross-sectional area of longitudinal reinforcing bars within b_{effh}
A_{cs}	is the cross-sectional area of composite slab within b_{effh} above profiled steel sheeting
$\rho_{sl,req}$	is the required minimum reinforcement ratio
δ	is equal to 1.0 for Class 2 cross-sections, and equal to 1.1 for Class 1 cross-sections at which plastic hinge rotation is required
f_{ay}	is the nominal value of yield strength of steel beam
f_{ctm}	is the mean value of tensile strength of concrete
f_{sk}	is the characteristic yield strength of reinforcing bars
k_c	is the coefficient taking into account of stress distribution within section immediately prior to cracking
h_{cs}	is the thickness of composite slab above profiled steel sheeting
z_0	is the vertical distance between centre of un-cracked concrete flange and un-cracked composite section
A_a	is the cross-sectional area of steel beam
D_a	is the depth of steel beam
D_{ps}	is the overall depth of profiled steel sheeting
n_0	is the modular ratio for short-term loading
E_a	is the modulus of elasticity of steel beam
$E_{cm,cs}$	is the secant modulus of elasticity of concrete for concrete slab

b) Degree of shear connection

$$\eta_s \geq \eta_{s,req} \tag{4.18}$$

$$\eta_h \geq \eta_{h,req} \tag{4.19}$$

$\eta_{s,req}$ and $\eta_{h,req}$ are given by:

$$\eta_{s,req} = \max\left\{1\text{-}\left(\frac{355}{f_{ayd}}\right)(0.75\text{-}0.03L_{es});\ 0.4\right\} \quad \text{for } L_{es} \leq 25 \tag{4.20}$$

$$\eta_{s,req} = 1 \quad \text{for } L_{es} > 25 \tag{4.21}$$

$$\eta_{h,req} = 1 \tag{4.22}$$

where

$$f_{ayd} = \frac{f_{ay}}{\gamma_a} \tag{4.23}$$

η_s	is the degree of shear connection in sagging moment region, see 4.4.2
η_h	is the degree of shear connection in hogging moment region, see 4.4.2
$\eta_{s,req}$	is the required minimum degree of shear connection in sagging moment region
$\eta_{h,req}$	is the required minimum degree of shear connection in hogging moment region
L_{es}	is the distance between inflection points in sagging moment region
f_{ayd}	is the design yield strength of steel beam
γ_a	is the partial factor of resistance of members and cross-sections of steel beam

c) Shear resistance

$$\min\left(V_{pl,Rd}\ ;\ V_{b,Rd}\right) \geq V_{Ed} \tag{4.24}$$

where

$V_{pl,Rd}$	is the plastic shear resistance of composite beam, see 4.4.3
$V_{b,Rd}$	is the shear buckling resistance of composite beam, see 4.4.3
V_{Ed}	is the design shear force

d) Moment resistance

$$\min\left(M_{plf,Rds}\ ;\ M_{plp,Rds}\right) \geq M_{Eds} \tag{4.25}$$

$$\min\left(M_{plf,Rdh}\ ;\ M_{y,v,Rdh}\right) \geq M_{Edh} \tag{4.26}$$

where

$M_{plf,Rds}$	is the plastic sagging moment resistance of composite beam with full shear connection, see 4.4.4
$M_{plp,Rds}$	is the plastic sagging moment resistance of composite beam with partial shear connection, see 4.4.4
M_{Eds}	is the design sagging moment
$M_{plf,Rdh}$	is the plastic hogging moment resistance of composite beam with full shear connection, see 4.4.4
$M_{y,v,Rdh}$	is the reduced moment resistance of composite beam making allowance for presence of shear force, see 4.4.4
M_{Edh}	is the design hogging moment

e) Lateral-torsional buckling moment resistance

$$M_{LT,Rd} \geq M_{Edh} \tag{4.27}$$

where

$M_{LT,Rd}$ is the buckling moment resistance of laterally unrestrained composite beam, see 4.4.4

f) Longitudinal shear resistance

$$R_{st}+R_{pse} \geq R_{tr,req} \tag{4.28}$$

$$A_{st} \geq A_{st,req} \tag{4.29}$$

$$v_{Rd} \geq v_{L,Ed} \tag{4.30}$$

$R_{tr,req}$ and $A_{st,req}$ are given by:

$$R_{tr,req} = h_{cs} \frac{v_{L,Ed}}{\cot\theta} \tag{4.31}$$

$$A_{st,req} = h_{cs} \frac{0.08\sqrt{f_{ck}}}{f_{sk}} \tag{4.32}$$

where

$R_{st}+R_{pse}$ is the tension resistance of transverse reinforcement per unit length, see 4.4.5

A_{st} is the cross-sectional area of transverse reinforcing bars per unit length

v_{Rd} is the crushing shear stress of concrete slab, see 4.4.5

$v_{L,Ed}$ is the design longitudinal shear stress in composite slab

$R_{tr,req}$ is the required tension resistance of transverse reinforcement per unit length

$A_{st,req}$ is the required minimum cross-sectional area of transverse reinforcing bars per unit length

θ is the angle between diagonal strut and axis of beam, $26.5° \leq \theta \leq 45°$ for concrete flange in compression, $38.6° \leq \theta \leq 45°$ for concrete flange in tension

f_{ck} is the characteristic cylinder strength of concrete for concrete slab

(3) Serviceability check in construction stage

As with (1), the following conditions for deflection at serviceability limit state should be satisfied as steel beams with simply supported ends as composite action cannot be developed until the concrete of floor slab has hardened.

$$\delta_V \leq \delta_{V,lim} \tag{4.33}$$

$$\delta_{P+V} \leq \delta_{P+V,lim} \tag{4.34}$$

where

δ_V is the deflection due to “live loads”

$\delta_{V,lim}$ is the limit of deflection due to “live loads”

δ_{P+V} is the deflection due to “dead loads and live loads”

$\delta_{P+V,lim}$ is the limit of deflection due to "dead loads and live loads"

(4) Serviceability check in composite stage

As with (2), the following conditions for deflection and vibration at serviceability limit state should be satisfied as composite beams with semi-rigid ends as composite action can be considered after the concrete of floor slab has hardened. In addition, the criteria of crack width control should be satisfied.

a) Deflection

$$\delta_V \leq \delta_{V,lim} \tag{4.35}$$

$$\delta_{tP+V} \leq \delta_{P+V,lim} \tag{4.36}$$

where

δ_V is the deflection due to "live loads"

$\delta_{V,lim}$ is the limit of deflection due to "live loads"

δ_{tP+V} is the deflection due to "dead loads, superimposed dead loads, and live loads"

$\delta_{P+V,lim}$ is the limit of deflection due to "dead loads, superimposed dead loads, and live loads"

b) Vibration

$$f_{P+0.1V} \geq f_{req} \tag{4.37}$$

where

$f_{P+0.1V}$ is the natural frequency due to "dead loads, superimposed dead loads, and 10% of live loads"

f_{req} is the required minimum natural frequency

c) Control of crack width

$$A_{sl} \geq A_{sl,req} \tag{4.38}$$

$$\sigma_{sl} \leq \sigma_{sl,lim} \quad \text{or} \quad p_{sl} \leq p_{sl,lim} \tag{4.39}$$

$A_{sl,req}$ is given by:

$$A_{sl,req} = \frac{0.72 k_c f_{ctm} A_{cs}}{\sigma_{s,lim}} \tag{4.40}$$

where

$$k_c = \min\left\{\frac{1}{1+\left(\dfrac{h_{cs}}{2z_0}\right)}+0.3\ ;\ 1.0\right\} \tag{4.41}$$

$$z_0 = \frac{(A_a+A_{sl})\left(0.5D_a+D_{ps}+0.5h_{cs}\right)}{A_a+A_{sl}+\left(\dfrac{h_{cs}b_{effh}}{n_0}\right)} \tag{4.42}$$

$$n_0 = \frac{E_a}{E_{cm,cs}} \tag{4.43}$$

A_{sl} is the cross-sectional area of longitudinal reinforcing bars within b_{effh}

b_{effh} is the effective width of composite beam in hogging moment region, see 4.4.1

σ_{sl} is the tensile stress in longitudinal reinforcing bars due to direct loading

$\sigma_{sl,lim}$ is the limit of stress permitted in longitudinal reinforcing bars immediately after cracking, given in Table 4.5

p_{sl} is the pitch of longitudinal reinforcing bars

$p_{sl,lim}$ is the limit of spacing of longitudinal reinforcing bars, given in Table 4.6

$A_{sl,req}$ is the required minimum cross-sectional area of longitudinal reinforcing bars within b_{effh}

A_{cs} is the cross-sectional area of composite slab within b_{effh} above profiled steel sheeting

k_c is the coefficient taking into account of stress distribution within section immediately prior to cracking

z_0 is the vertical distance between centre of un-cracked concrete flange and un-cracked composite section

Table 4.5: Limit of stress permitted in longitudinal reinforcing bars

Limit of stress $\sigma_{sl,lim}$ [N/mm^2]	Maximum diameter of longitudinal reinforcing bars ϕ_{sl}^* [mm] for design crack width w_k		
	w_k = 0.4 [mm]	w_k = 0.3 [mm]	w_k = 0.2 [mm]
160	40	32	25
200	32	25	16
240	20	16	12
280	16	12	8
320	12	10	6
360	10	8	5
400	8	6	4
450	6	5	-

Table 4.6: Limit of spacing of longitudinal reinforcing bars

Limit of stress $\sigma_{sl,lim}$ [N/mm^2]	Limit of spacing of longitudinal reinforcing bars $p_{sl,lim}$ [mm] for design crack width w_k		
	w_k = 0.4 [mm]	w_k = 0.3 [mm]	w_k = 0.2 [mm]
160	300	300	200
200	300	250	150
240	250	200	100
280	200	150	50
320	150	100	-
360	100	50	-

Commentary:

(1) Structural resistance check in construction stage

Unless floor slab and beams are propped during constructions, dead loads and live loads in construction stage are supported by steel beams because composite action cannot be developed until concrete of floor slab has hardened. Therefore, structural resistance as the steel beams with simply supported ends at ultimate limit state should be checked.

With respect to the classification of cross-section, at least Class 2 should be used for steel beams to prevent the local buckling of web and flange as mentioned in Section 3.2.

With respect to the shear resistance, the following condition should be satisfied to make the plastic shear resistance and shear buckling resistance of steel beam larger than the design shear force at ultimate limit state.

$$\min\left(V_{pl,a,Rd}\ ;\ V_{b,a,Rd}\right) \geq V_{Ed} \tag{4.44}$$

With respect to the moment resistance, the following condition should be satisfied to make the plastic moment resistance of steel beam larger than the design sagging moment at ultimate limit state. According to EN 1993-1-1[19], when a steel beam is subjected to bending and shear, the plastic moment resistance of the steel beam should be reduced considering the effect of the shear force. However, steel beams are simply supported in construction stage and the design sagging moment is equal to the moment at beam centre where no shear force occurs. Thus, the reduction of the plastic moment resistance need not be considered.

$$M_{pl,a,Rd} \geq M_{Eds} \tag{4.45}$$

With respect to the lateral-torsional buckling, the following condition should be satisfied to make the buckling moment resistance of laterally unrestrained steel beam larger than the design moment at ultimate limit state. However, when the profiled steel sheeting spans perpendicularly to a steel beam and is attached to its top flange, the beam can be considered as restrained along its length. For this case, lateral torsional buckling check is not required.

$$M_{LT,a,Rd} \geq M_{Eds} \tag{4.46}$$

(2) Structural resistance check in composite stage

Regardless of propped or un-propped constructions, total dead loads and live loads in composite stage are supported by composite beams because composite action can be considered after the concrete of floor slab has hardened. Therefore, structural resistance as the composite beams with semi-rigid ends at ultimate limit state should be checked. This is based on the concept of ultimate limit state that composite beams can develop the moment resistance as the composite sections when they reach the ultimate strength.

With respect to the classification of cross-section, at least Class 2 should be used for composite beams to prevent the local buckling of web and flange. It should be noted that the classification of cross-section of composite beams may be different from that of steel beams even if the same steel beams are used. This is because the neutral axis of composite beams is different from that of steel beams due to the composite effect with floor slab. Additionally, if the longitudinal reinforcing bars in concrete slab are in tension and the elastic hogging moment resistance of composite beam $M_{el,Rdh}$ is less than the moment resistance of the adjacent composite joints $M_{j,Rd}$, the following condition should be satisfied to make the cross-sectional area of longitudinal reinforcing bars within b_{effh} at ultimate limit state larger than the required minimum value.

$$A_{sl} \geq A_{sl,req} \tag{4.47}$$

With respect to the degree of shear connection, the following conditions should be satisfied to make the degree of shear connection in sagging and hogging moment region at ultimate limit state larger than the respective required minimum values. Basically, composite beams should be designed with full shear connection, but many shear studs may be required when the composite beams subjected to only the gravity load are designed with full shear connection. However, as far as ductile shear studs are used and the degree of shear connection is not extremely small, the shear studs can be greatly deformed at ultimate limit state, and the composite beams can exhibit relatively large rotational capacity in sagging moment region. Therefore, the use of the composite beams with partial shear connection in sagging moment region is permitted in EN 1994-1-1. On the other hand, composite beams should be designed with full shear connection in hogging moment region because the structural behaviour of the composite beams with partial shear connection has not been clearly elucidated.

$$\eta_s \geq \eta_{s,req} \tag{4.48}$$

$$\eta_h \geq \eta_{h,req} \tag{4.49}$$

With respect to the shear resistance, the following condition should be satisfied to make the plastic shear resistance and shear buckling resistance of composite beam larger than the design shear force at ultimate limit state. Here, the contribution of floor slab can be ignored, so that the plastic shear resistance and shear buckling resistance of composite beams can be equal to those of steel beams.

$$\min\left(V_{pl,Rd}\ ;\ V_{b,Rd}\right) \geq V_{Ed} \tag{4.50}$$

With respect to the moment resistance, the following conditions should be satisfied to make the plastic moment resistance of composite beam larger than the design moment at ultimate limit state. When composite beams are designed with partial shear connection in sagging moment region, the reduction of the plastic moment resistance with full shear connection should be considered depending on the reduced number of shear studs. In addition, beam members are subjected to bending and shear in hogging moment region, thus the reduction of plastic moment resistance should be considered also in hogging moment region.

$$\min\left(M_{plf,Rds}\ ;\ M_{plp,Rds}\right) \geq M_{Eds} \tag{4.51}$$

$$\min\left(M_{plf,Rdh}\ ;\ M_{y,v,Rdh}\right) \geq M_{Edh} \tag{4.52}$$

With respect to the lateral-torsional buckling, the following condition should be satisfied to make the buckling moment resistance of laterally unrestrained composite beam larger than the design moment at ultimate limit state.

$$M_{LT,Rd} \geq M_{Edh} \tag{4.53}$$

With respect to the longitudinal shear resistance, the following conditions should be satisfied to make the tension resistance of transverse reinforcement per unit length and the cross-sectional area of transverse reinforcing bars per unit length larger than the respective required minimum values.

$$R_{st}+R_{pse} \geq R_{tr,req} \tag{4.54}$$

$$A_{st} \geq A_{st,req} \tag{4.55}$$

The following condition should also be satisfied to make the shear stress of concrete slab larger than the design longitudinal shear stress in composite slab at ultimate limit state.

$$v_{Rd} \geq v_{L,Ed} \tag{4.56}$$

(3) Serviceability check in construction stage

As with (1), unless floor slab and beams are propped during constructions, dead loads and live loads in construction stage are supported by steel beams because composite action cannot be developed until concrete of floor slab has hardened. Therefore, serviceability deflection of the steel beam with simply supported ends should be checked.

With respect to the deflection, the following conditions should be satisfied to make the deflection due to “live loads only” and “dead loads and live loads” smaller than the respective limits.

$$\delta_V \leq \delta_{V,lim} \tag{4.57}$$

$$\delta_{P+V} \leq \delta_{P+V,lim} \tag{4.58}$$

Based on EN 1990[20], the limits of deflection should be specified in each project. However, for typical office buildings, $L_b/360$ and $L_b/200$ are considered as the limit of deflection due to “live loads only” and “dead loads and live loads”, respectively. Here, L_b is the beam span.

(4) Serviceability check in composite stage

As with (2), regardless of propped or un-propped constructions, serviceability deflection and vibration of the composite beams with semi-rigid ends should be checked.

With respect to the deflection, the following conditions should be satisfied to make the deflection due to “superimposed dead loads and live loads” and “dead loads, superimposed dead loads, and live loads” smaller than the respective limits.

$$\delta_V \leq \delta_{V,lim} \tag{4.59}$$

$$\delta_{tP+V} \leq \delta_{P+V,lim} \tag{4.60}$$

Unless floor slab and beams are propped during constructions, the total beam deflection should be the sum of the deflection of steel beam due to “dead loads” and the deflection of composite beam due to “superimposed dead loads and live loads”. For typical office buildings, $L_b/360$ and $L_b/200$ are considered as the limit of deflection due to “superimposed dead loads and live loads” and “total load”, respectively.

With respect to the vibration, the following condition should be satisfied to make the natural frequency of composite beam at serviceability limit state larger than the required minimum value. Note that 10% of the live loads can be taken into account in calculating the natural frequency.

$$f_{P+0.1V} \geq f_{req} \tag{4.61}$$

To control the crack width in the slab, the following condition should be satisfied to make the cross-sectional area of longitudinal reinforcing bars within b_{effh} at serviceability limit state larger than the required minimum value.

$$A_{sl} \geq A_{sl,req} \tag{4.62}$$

The following condition should also be satisfied to make the tensile stress in longitudinal reinforcing bars due to direct loading at serviceability limit state smaller than its limit or the pitch of longitudinal reinforcing bars smaller than its limit.

$$\sigma_{sl} \leq \sigma_{sl,lim} \quad \text{or} \quad p_{sl} \leq p_{sl,lim} \tag{4.63}$$

4.3 Structural Properties of Composite Joint

4.3.1 Effective Width and Effective Length

(1) Effective width of composite joint

The effective width of composite joints $b_{eff,j}$ which is an essential design parameter to evaluate the rotational stiffness and moment resistance of the joints can be referred to EN 1994-1-1 for effective width at the end of the beam. However, it can be modified based on the experimental evidence or advanced calculation method supported by structural testing.

(2) Effective length of reinforcement

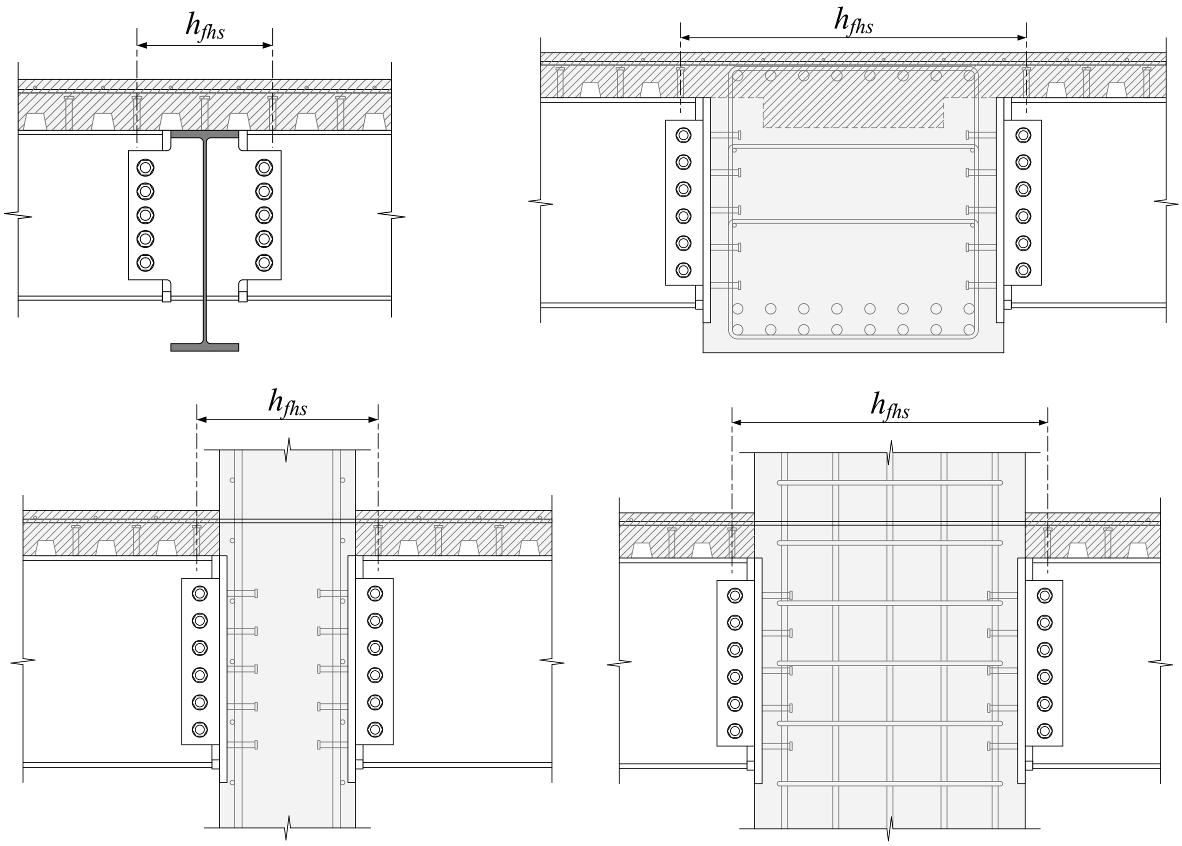
The effective length of reinforcement l_{eff} is another essential design parameter to evaluate the joint rotational stiffness. It can be determined by the following equations depending on the shape of longitudinal reinforcing bars at composite joints.

$$l_{eff,r} = \min\left(\frac{h_{fhs}}{2}\ ;\ 20\phi_{sl,r}\right) \quad \text{for straight reinforcing bars} \tag{4.64}$$

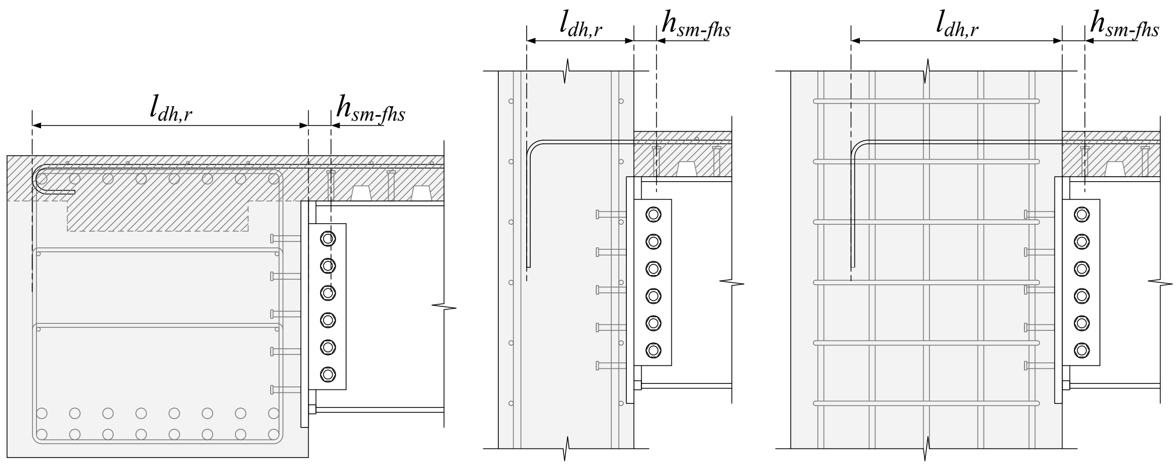
$$l_{eff,r} = \min\left(\frac{l_{dh,r}+h_{sm\text{-}fhs}}{2}\ ;\ 20\phi_{sl,r}\right) \quad \text{for bent or hooked reinforcing bars} \tag{4.65}$$

where

h_{fhs}	is the distance between first headed studs on opposite composite beams, see Figure 4.5
$\phi_{sl,r}$	is the diameter of longitudinal reinforcing bars for a row r
$l_{dh,r}$	is the projected anchorage length of bent or hooked reinforcing bars for a row r, see Figure 4.5
$h_{sm\text{-}fhs}$	is the distance between surface of supporting member and first headed stud, see Figure 4.5



(a) Composite joint with straight reinforcing bars



(b) Composite joint with bent or hooked reinforcing bars

Figure 4.5: Effective length of reinforcement

Commentary:

(1) Effective width of composite joint

The effective width of composite joint $b_{eff,j}$ is an essential design parameter to evaluate the rotational stiffness and the moment resistance of the joints. It can be the effective width of the slab at the end of the composite beam in accordance with EN 1994-1-1. However, $b_{eff,j}$ may depend on the joint detail and the arrangement of the various joint components. As such, the moment-rotation characteristic of composite joint may not be accurately predicted based on the current definition in EN 1994-1-1. The effective width of composite joint $b_{eff,j}$ may also be obtained based on the experimental evidence or advanced calculation method supported by structural testing.

(2) Effective length of reinforcement

In addition to $b_{eff,j}$, the effective length of reinforcement l_{eff} is another essential design parameter to evaluate the rotational stiffness of the joints. In EN 1994-1-1, the effective length of reinforcement is specified but limited to the beam-to-column composite joints with steel column section. Therefore, it may not be applicable to beam-to-beam and beam-to-wall composite joints as well as the beam-to-column composite joints with relatively deeper concrete column section.

In general, the effective length of reinforcement depends on the length where the longitudinal reinforcing bars are subjected to tension. According to the full-scale joint component tests[12], it can be determined by the distance between the first headed studs on the opposite composite beams h_{fhs}, provided that the joint is double-sided with straight reinforcing bars. However, the tensile stress in the longitudinal reinforcing bars may decrease toward the joint centre since the tension force of the reinforcing bars is transferred to the supporting member by bond with concrete. In other words, if h_{fhs} is relatively large due to a deep column section, the effective length cannot be determined only by h_{fhs} because the reinforcing bars near the joint centre may no longer be subjected to tension. In this design guide, the effective length of reinforcement l_{eff} can be determined considering h_{fhs} and bar diameters ϕ_{sl} for straight reinforcing bars and projected anchorage length l_{dh} and ϕ_{sl} for bent or hooked reinforcing bars.

4.3.2 Initial Rotational Stiffness

(1) Concept

The initial rotational stiffness of composite joints should be evaluated considering the longitudinal reinforcing bars in tension and the contact parts in compression as the basic joint components. Concrete in tension shall be neglected.

(2) Calculation of initial rotational stiffness

a) Beam-to-beam composite joint with primary composite beam

The initial rotational stiffness of beam-to-beam composite joints with primary composite beam $S_{j,ini}$ can be determined by the following equation assuming the assembly of the two elastic springs for each component, see Figure 4.6 (a). Here, the stiffness of the elastic spring for the contact part k_{con} can be taken as infinity as proposed in EN 1994-1-1. However, in the case for the secondary beams of different depths, the effect of shear deformation of the fin plates needs to be considered if necessary.

$$S_{j,ini} = E_s k_{slip} k_{sl,eq} z_{sl,eq\text{-}cc}^{\,2} \tag{4.66}$$

k_{slip} and $k_{sl,eq}$ are given by:

$$k_{slip} = \frac{1}{1+\left(\dfrac{E_s k_{sl,eq}}{K_{sc}}\right)} \tag{4.67}$$

$$k_{sl,eq} = \sum \frac{A_{sl,r}}{l_{eff,r}} \tag{4.68}$$

where

$$K_{sc} = \frac{N k_{sc}}{\nu\text{-}\left(\dfrac{\nu\text{-}1}{1+\xi}\right)\left(\dfrac{z_{sl,eq\text{-}cc}}{z_{sl,eq\text{-}ca}}\right)} \tag{4.69}$$

$$\nu = \sqrt{\frac{(1+\xi) N k_{sc} l z_{sl,eq\text{-}ca}^{\,2}}{E_a I_{ay}}} \tag{4.70}$$

$$\xi = \frac{E_a I_{ay}}{z_{sl,eq\text{-}ca}^{\,2} E_s A_{sl,r}} \tag{4.71}$$

E_s	is the modulus of elasticity of reinforcing bars
$z_{sl,eq\text{-}cc}$	is the equivalent vertical distance between longitudinal reinforcing bars and centre of contact part
k_{slip}	is the stiffness reduction factor due to deformation of headed studs
$k_{sl,eq}$	is the equivalent stiffness coefficient of longitudinal reinforcing bars
$A_{sl,r}$	is the cross-sectional area of longitudinal reinforcing bars within $b_{eff,j}$ for a row r
$b_{eff,j}$	is the effective width of composite joint, see 4.3.1
$l_{eff,r}$	is the effective length of reinforcement, see 4.3.1
K_{sc}	is the stiffness related to headed studs
N	is the number of headed studs distributed over length l

l	is the length of composite beam in hogging moment region adjacent to joint, which can be taken as 15% of beam span
k_{sc}	is the stiffness of one headed stud, which can be taken as 100 [kN/mm]
$z_{sl,eq\text{-}ca}$	is the equivalent vertical distance between longitudinal reinforcing bars and centre of steel beam
ν	is the parameter related to deformation of headed studs
E_a	is the modulus of elasticity of steel beam
I_{ay}	is the second moment of area of steel beam about major axis (y-y axis)
ξ	is the parameter related to deformation of headed studs

b) Beam-to-beam composite joint with primary reinforced concrete beam

As far as Eq.(4.72) and (4.73) are satisfied, the initial rotational stiffness of beam-to-beam composite joints with primary reinforced concrete beam $S_{j,ini}$ can be determined by Eq.(4.74) assuming the assembly of the two elastic springs for each component, see Figure 4.6 (b). In the case of a single-sided joint, the primary beam may be torsionally deformed by the additional moment from the secondary beam, resulting in the apparent decrease in $S_{j,ini}$. Therefore, the equivalent initial rotational stiffness $S_{j,ini,eq}$ incorporating the torsional rigidity of the primary beam should be used as the rotational stiffness S_j in structural analysis if necessary.

$$B_{pb} \geq \max\{(\alpha\text{-}1)d_{eff}\ ;\ (\alpha\text{-}1)b_{eff}\} \tag{4.72}$$

$$D_{pb}\text{-}D_{cs}\text{-}D_a \geq \frac{\alpha d_{eff}\text{-}t_f}{2} \tag{4.73}$$

$$S_{j,ini} = E_s \frac{k_{slip}k_{sl,eq}k_{13,pb}}{\left(k_{slip}k_{sl,eq}+k_{13,pb}\right)} z_{sl,eq\text{-}cc}{}^2 \tag{4.74}$$

d_{eff}, b_{eff}, k_{slip}, $k_{sl,eq}$, and $k_{13,pb}$ are given by:

$$d_{eff} = t_f+c+\min\left(D_{ep}\text{-}D_a\ ;\ c\right) \tag{4.75}$$

$$b_{eff} = \min\left(B_a+2c\ ;\ B_{ep}\right) \tag{4.76}$$

$$k_{slip} = \frac{1}{1+\left(\dfrac{E_s k_{sl,eq}}{K_{sc}}\right)} \tag{4.77}$$

$$k_{sl,eq} = \sum \frac{A_{sl,r}}{l_{eff,r}} \tag{4.78}$$

$$k_{13,pb} = \frac{E_{cm,pb}\sqrt{d_{eff}b_{eff}}}{1.275E_a} \tag{4.79}$$

α is given by Eq.(4.80) using convergence calculation, but it should be 3.0 if the calculated value is more than 3.0.

$$(\alpha\text{-}1)b_{eff}\text{-}B_{pb} = 0 \tag{4.80}$$

where

$$c = t_{ep}\sqrt{\frac{f_{epyd}}{3f_{jd,pb}}} \tag{4.81}$$

$$f_{epyd} = \frac{f_{epy}}{\gamma_{ep}} \tag{4.82}$$

$$f_{jd,pb} = \beta_j \alpha f_{cd,pb} \tag{4.83}$$

$$f_{cd,pb} = \frac{f_{ck,pb}}{\gamma_{c,pb}} \tag{4.84}$$

B_{pb} is the width of primary reinforced concrete beam
α is the amplification factor from loaded area to maximum design distribution area
D_{pb} is the depth of primary reinforced concrete beam
D_a is the depth of steel beam
t_f is the flange thickness of steel beam
d_{eff} is the effective depth of equivalent T-stub flange in compression
b_{eff} is the effective width of equivalent T-stub flange in compression
$k_{13,pb}$ is the stiffness coefficient of concrete for primary reinforced concrete beam
D_{ep} is the depth of end plate
B_a is the width of steel beam
$E_{cm,pb}$ is the secant modulus of elasticity of concrete for primary reinforced concrete beam
E_a is the modulus of elasticity of steel beam
c is the additional bearing width of equivalent T-stub flange in compression
t_{ep} is the thickness of end plate
f_{epyd} is the design yield strength of end plate
f_{epy} is the nominal value of yield strength of end plate
γ_{ep} is the partial factor of resistance of members and cross-sections of end plate
$f_{jd,pb}$ is the design bearing strength of concrete for primary reinforced concrete beam
β_j is the joint material coefficient of precast reinforced concrete beam, which can be taken as 2/3
$f_{cd,pb}$ is the design strength of concrete for primary reinforced concrete beam
$f_{ck,pb}$ is the characteristic cylinder strength of concrete for primary reinforced concrete beam
$\gamma_{c,pb}$ is the partial factor of concrete for primary reinforced concrete beam

c) Beam-to-wall composite joint with reinforced concrete wall

As far as Eq.(4.85) is satisfied, the initial rotational stiffness of beam-to-wall composite joints with reinforced concrete wall $S_{j,ini}$ can be determined by Eq.(4.86) assuming the assembly of the two elastic springs for each component, see Figure 4.6 (c). In the case of a single-sided joint, the wall may be locally deformed by the additional moment from the beam member, resulting in the apparent decrease in $S_{j,ini}$. Therefore, the equivalent initial rotational stiffness $S_{j,ini,eq}$ incorporating the flexural rigidity of the wall should be used as the rotational stiffness S_j in structural analysis if necessary.

$$t_{cw} \geq \max\{(\alpha\text{-}1)d_{eff}\,;\,(\alpha\text{-}1)b_{eff}\} \tag{4.85}$$

$$S_{j,ini} = E_s \frac{k_{slip}k_{sl,eq}k_{13,cw}}{\left(k_{slip}k_{sl,eq}+k_{13,cw}\right)} z_{sl,eq\text{-}cc}{}^2 \tag{4.86}$$

d_{eff}, b_{eff}, k_{slip}, $k_{sl,eq}$, and $k_{13,cw}$ are given by:

$$d_{eff} = t_f + c + \min\left(D_{ep}\text{-}D_a\,;\,c\right) \tag{4.87}$$

$$b_{eff} = \min\left(B_a+2c\,;\,B_{ep}\right) \tag{4.88}$$

$$k_{slip} = \frac{1}{1+\left(\dfrac{E_s k_{sl,eq}}{K_{sc}}\right)} \tag{4.89}$$

$$k_{sl,eq} = \sum \frac{A_{sl,r}}{l_{eff,r}} \tag{4.90}$$

$$k_{13,cw} = \frac{E_{cm,cw}\sqrt{d_{eff}b_{eff}}}{1.275E_a} \tag{4.91}$$

α is given by Eq.(4.95) using convergence calculation, but it should be 3.0 if the calculated value is more than 3.0.

$$(\alpha\text{-}1)b_{eff}\text{-}t_{cw} = 0 \tag{4.92}$$

where

$$c = t_{ep}\sqrt{\frac{f_{epyd}}{3f_{jd,cw}}} \tag{4.93}$$

$$f_{jd,cw} = \beta_j \alpha f_{cd,cw} \tag{4.94}$$

$$f_{cd,cw} = \frac{f_{ck,cw}}{\gamma_{c,cw}} \tag{4.95}$$

t_{cw} is the thickness of reinforced concrete wall
$k_{13,cw}$ is the stiffness coefficient of concrete for reinforced concrete wall
$E_{cm,cw}$ is the secant modulus of elasticity of concrete for reinforced concrete wall
$f_{jd,cw}$ is the design bearing strength of concrete for reinforced concrete wall
$f_{cd,cw}$ is the design strength of concrete for reinforced concrete wall
$f_{ck,cw}$ is the characteristic cylinder strength of concrete for reinforced concrete wall
$\gamma_{c,cw}$ is the partial factor of concrete for reinforced concrete wall

d) Beam-to-column composite joint with reinforced concrete column

As far as Eq.(4.96) is satisfied, the initial rotational stiffness of beam-to-column composite joints with reinforced concrete column $S_{j,ini}$ can be determined by Eq.(4.97) assuming the assembly of the two elastic springs for each component, see Figure 4.6 (d). In the case of a single-sided joint, the column may be locally deformed by the additional moment from the beam member especially for single-sided joints, resulting in the apparent decrease in $S_{j,ini}$. Therefore, the equivalent initial rotational stiffness $S_{j,ini,eq}$ incorporating the flexural rigidity of the column should be used as the rotational stiffness S_j in structural analysis if necessary.

$$D_{cc} \geq \max\left\{(\alpha\text{-}1)d_{eff}\,;\,(\alpha\text{-}1)b_{eff}\right\} \tag{4.96}$$

$$S_{j,ini} = E_s \frac{k_{slip}k_{sl,eq}k_{13,cc}}{\left(k_{slip}k_{sl,eq}+k_{13,cc}\right)} z_{sl,eq\text{-}cc}{}^2 \tag{4.97}$$

d_{eff}, b_{eff}, k_{slip}, $k_{sl,eq}$, and $k_{13,cc}$ are given by:

$$d_{eff} = t_f + c + \min\left(D_{ep}\text{-}D_a\,;\,c\right) \tag{4.98}$$

$$b_{eff} = \min\left(B_a+2c\,;\,B_{ep}\right) \tag{4.99}$$

$$k_{slip} = \frac{1}{1+\left(\dfrac{E_s k_{sl,eq}}{K_{sc}}\right)} \tag{4.100}$$

$$k_{sl,eq} = \sum \frac{A_{sl,r}}{l_{eff,r}} \tag{4.101}$$

$$k_{13,cc} = \frac{E_{cm,cc}\sqrt{d_{eff}b_{eff}}}{1.275E_a} \tag{4.102}$$

α is given by Eq.(4.106) using convergence calculation, but it should be 3.0 if the calculated value is more than 3.0.

$$(\alpha\text{-}1)b_{eff}\text{-}D_{cc} = 0 \tag{4.103}$$

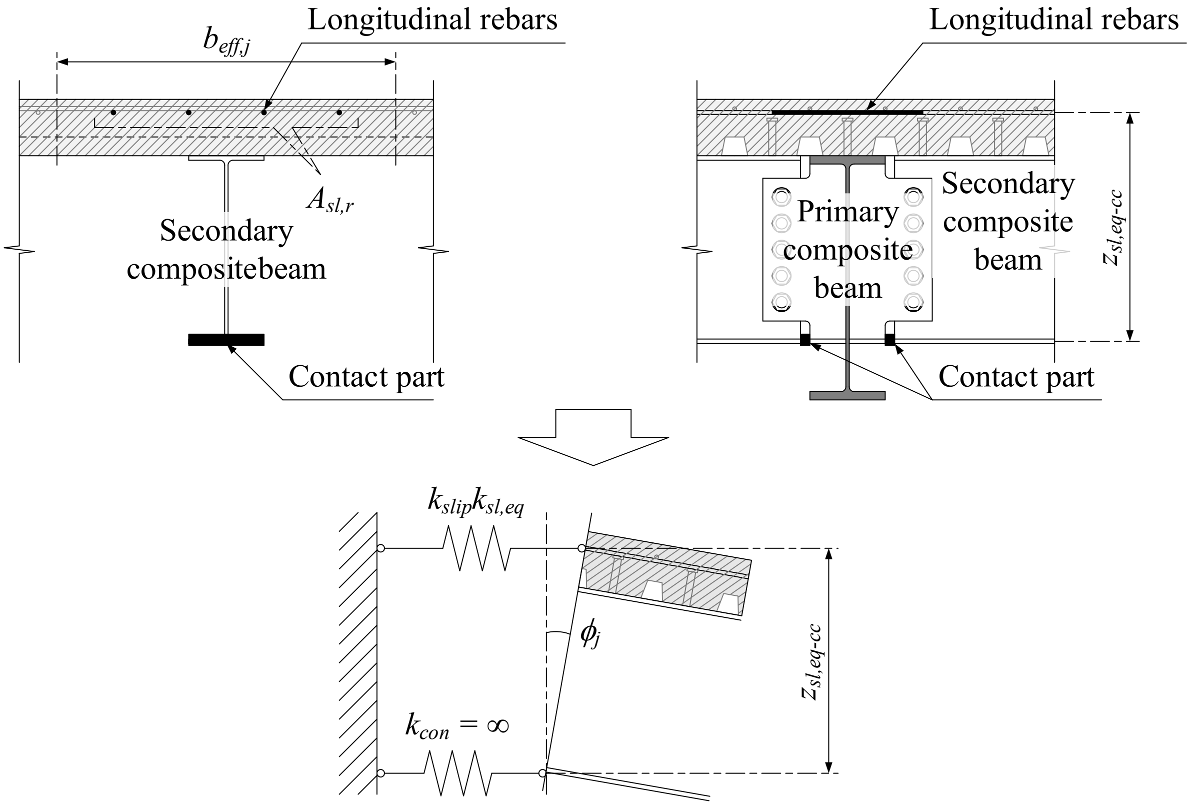
where

$$c = t_{ep}\sqrt{\frac{f_{epyd}}{3f_{jd,cc}}} \tag{4.104}$$

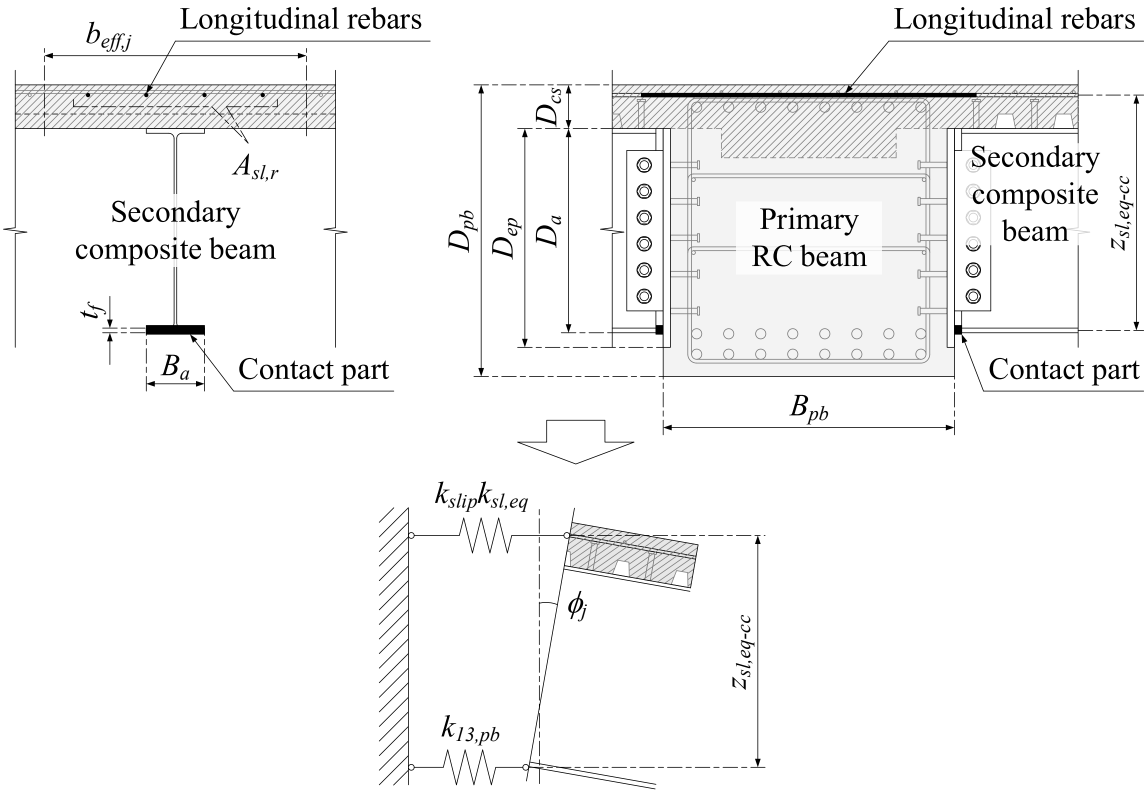
$$f_{jd,cc} = \beta_j \alpha f_{cd,cc} \tag{4.105}$$

$$f_{cd,cc} = \frac{f_{ck,cc}}{\gamma_{c,cc}} \tag{4.106}$$

D_{cc}	is the depth of reinforced concrete column
$k_{13,cc}$	is the stiffness coefficient of concrete for reinforced concrete column
$E_{cm,cc}$	is the secant modulus of elasticity of concrete for reinforced concrete column
$f_{jd,cc}$	is the design bearing strength of concrete for reinforced concrete column
$f_{cd,cc}$	is the design strength of concrete for reinforced concrete column
$f_{ck,cc}$	is the characteristic cylinder strength of concrete for reinforced concrete column
$\gamma_{c,cc}$	is the partial factor of concrete for reinforced concrete column

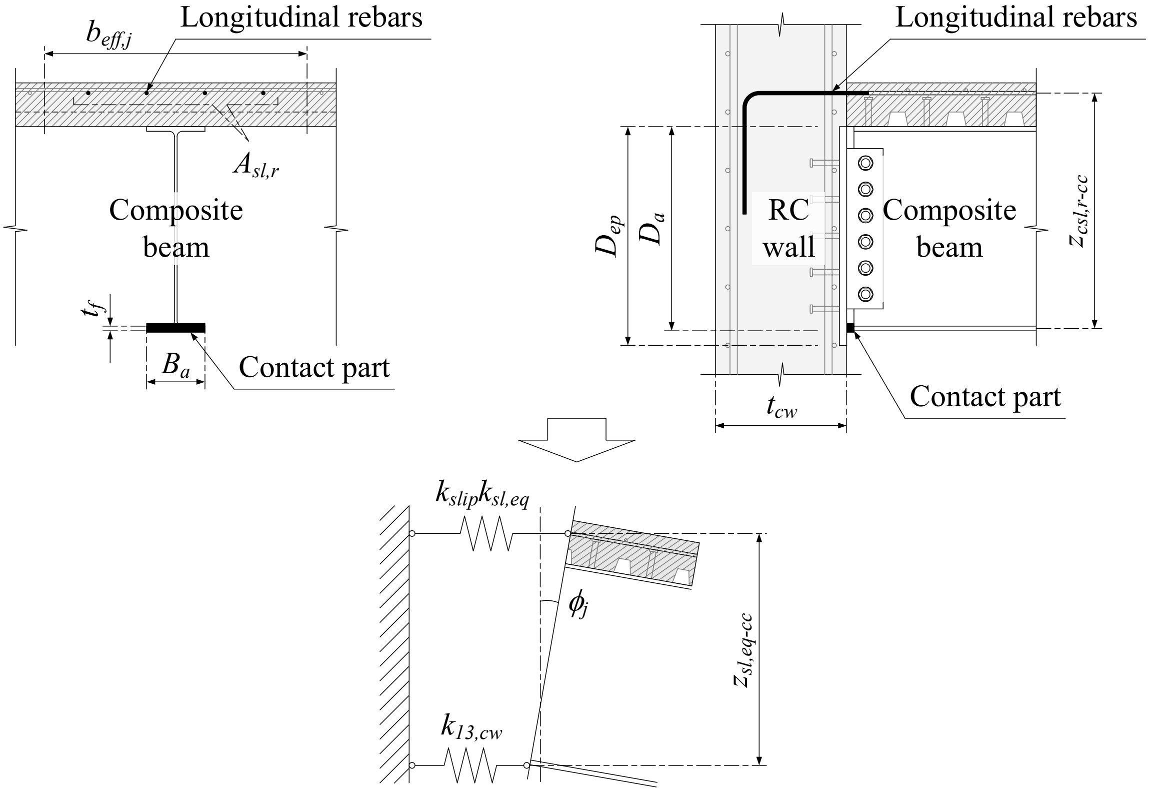


(a) Beam-to-beam composite joint with primary composite beam

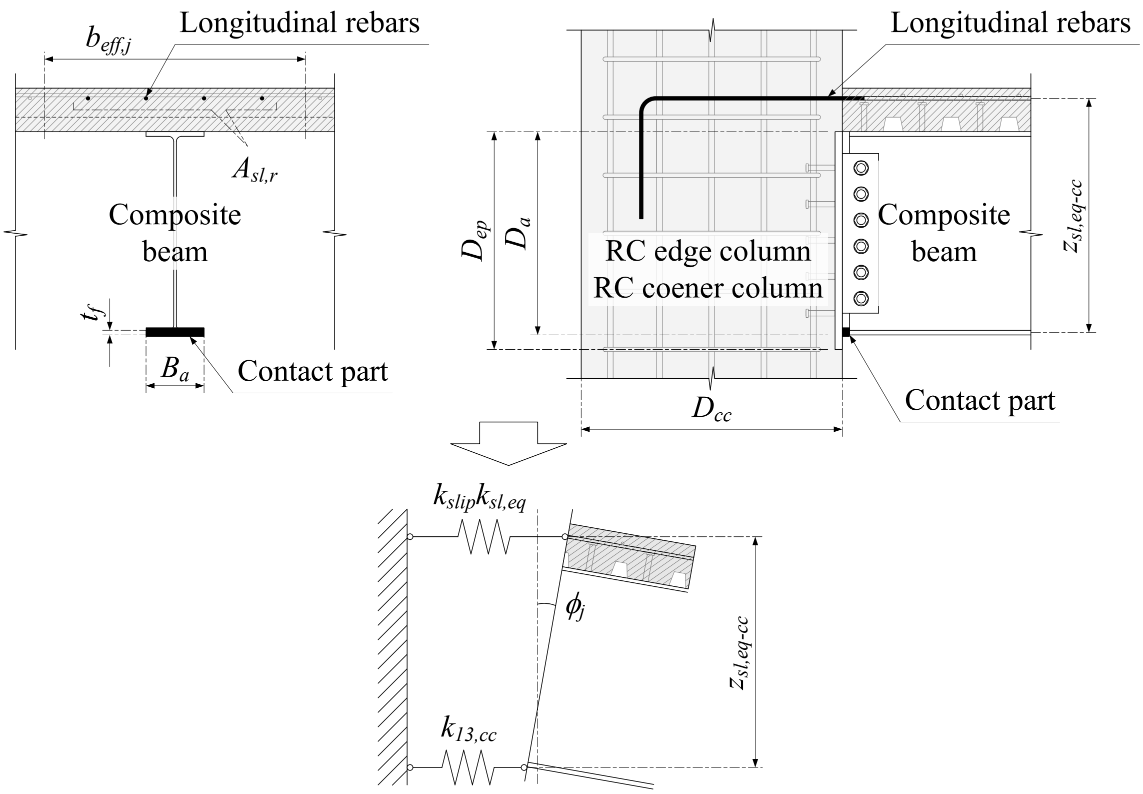


(b) Beam-to-beam composite joint with primary reinforced concrete beam

Figure 4.6: Modelling of composite joint for initial rotational stiffness



(c) Beam-to-wall composite joint with reinforced concrete wall



(d) Beam-to-column composite joint with reinforced concrete column

Figure 4.6: Modelling of composite joint for initial rotational stiffness (cont'd)

Commentary:

(1) Concept

In EN 1993-1-8, the component method in which a semi-rigid joint is modelled as an assembly of basic components is described as one of the analytical methods to predict its structural response. Based on this method, a composite joint with contact plates can be modelled as an assembly of two components represented by the elastic springs as shown in Figure 4.6, one is the longitudinal reinforcing bars in tension and the other is the contact parts in compression. Concrete in tension shall be neglected considering the effects of cracking.

4.3.3 Moment resistance

(1) Concept

The moment resistance of composite joints should be evaluated considering the longitudinal reinforcing bars in tension and the contact parts in compression as the basic joint components. Concrete in tension shall be neglected.

(2) Calculation of moment resistance

As shown in Figure 4.7, the moment resistance of composite joints $M_{j,Rd}$ can be determined by the following equation using the tension resistance of the longitudinal reinforcing bars $R_{sl,j}$ or the compression resistance of the contact part R_{con}, whichever is smaller. In the case for beam-to-beam composite joints with primary composite beam, the stiffeners should be considered in calculating R_{con} unless they are welded to the fin plates and the width, thickness, and the nominal value of yield strength of the stiffeners are more than or equal to those of bottom flange of steel beams.

$$M_{j,Rd} = z_{sl,eq\text{-}cc} \min\left(R_{sl,j}\ ;\ R_{con}\right) \tag{4.107}$$

$R_{sl,j}$ and R_{con} are given by:

$$R_{sl,j} = \sum A_{sl,r} f_{sd} \tag{4.108}$$

$$R_{con} = \min\left\{A_{bf} f_{ayd}\ ;\ A_{cp} f_{cpyd}\ ;\ 1.5 A_{bea} \min\left(\frac{f_{ay}}{\gamma_{a,2}}\ ;\ \frac{f_{cpy}}{\gamma_{cp,2}}\right)\right\} \tag{4.109}$$

where

$$f_{sd} = \frac{f_{sk}}{\gamma_s} \tag{4.110}$$

$$f_{cpyd} = \frac{f_{cpy}}{\gamma_{cp}} \tag{4.111}$$

$z_{sl,eq\text{-}cc}$	is the equivalent vertical distance between longitudinal reinforcing bars and centre of contact part
$R_{sl,j}$	is the tension resistance of longitudinal reinforcing bars within $b_{eff,j}$
R_{con}	is the compression resistance of contact part
$b_{eff,j}$	is the effective width of composite joint, see 4.3.1
$A_{sl,r}$	is the cross-sectional area of longitudinal reinforcing bars within $b_{eff,j}$ for a row r
A_{bf}	is the cross-sectional area of bottom flange of steel beam
f_{ayd}	is the design yield strength of steel beam, see 4.2.2
A_{cp}	is the cross-sectional area of contact plate
A_{bea}	is the bearing area between bottom flange of steel beam and contact plate
f_{ay}	is the nominal value of yield strength of steel beam
$\gamma_{a,2}$	is the partial factor of resistance of steel beam in bearing
f_{cpy}	is the nominal value of yield strength of contact plate
$\gamma_{cp,2}$	is the partial factor of resistance of contact plate in bearing

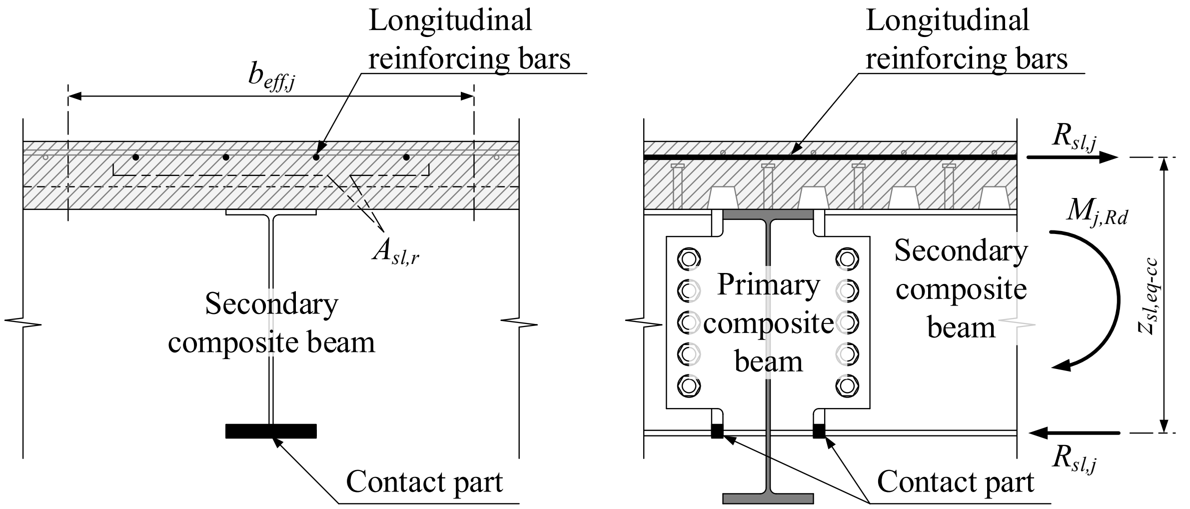
f_{sd} is the design yield strength of reinforcing bars

f_{sk} is the characteristic yield strength of reinforcing bars

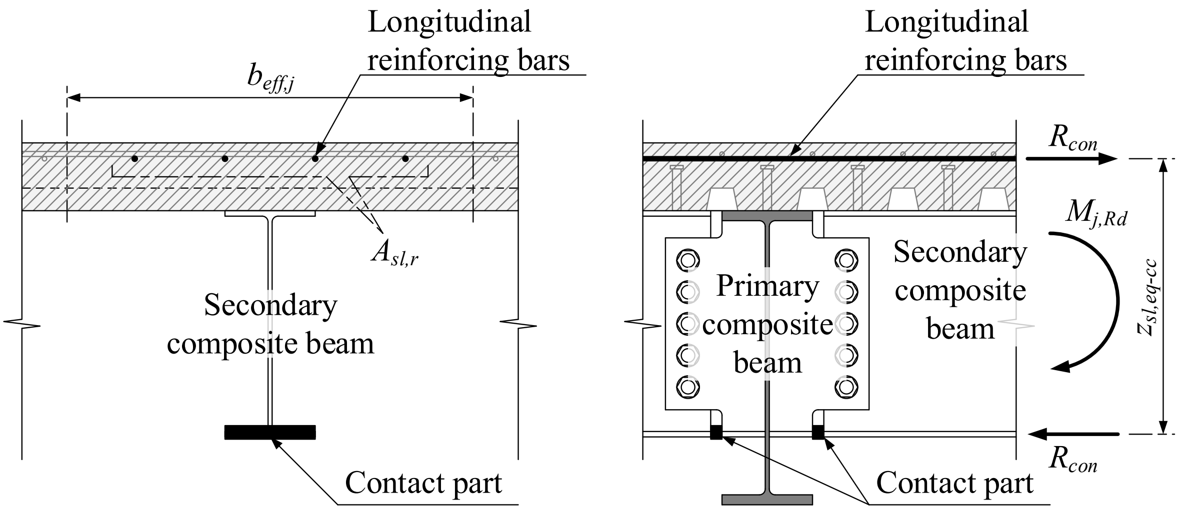
γ_s is the partial factor of reinforcing bars

f_{cpyd} is the design yield strength of contact plate

γ_{cp} is the partial factor of resistance of members and cross-sections of contact plate



(a) $R_{sl,j} \leq R_{con}$



(b) $R_{sl,j} > R_{con}$

Figure 4.7: Modelling of composite joint for moment resistance

4.3.4 Compression Resistance

(1) Compression resistance of equivalent T-stub flange

The compression resistance of equivalent T-stub flange $F_{C,Rd}$ to be checked for the composite joints with primary reinforced concrete beam, reinforced concrete wall, and reinforced concrete column can be determined by Eq.(4.112).

$$F_{C,Rd} = f_{jd} d_{eff} b_{eff} \tag{4.112}$$

where

f_{jd} is the design bearing strength of concrete, which can be taken as $f_{jd,pb}$ for primary reinforced concrete beam, $f_{jd,cw}$ for reinforced concrete wall, and $f_{jd,cc}$ for reinforced concrete column

4.4 Structural Properties of Composite Beam

4.4.1 Effective Width

(1) Effective width of composite beams in sagging moment region

The effective width of composite beams in sagging moment region b_{effs} which is an essential design parameter to evaluate the flexural rigidity and the moment resistance of the beams in the sagging moment region can be determined by the following equation.

$$b_{effs} = b_{0s} + \sum b_{eis} \tag{4.113}$$

b_{eis} is given by:

$$b_{eis} = \min\left(\frac{L_{es}}{8}\ ;\ b_{is}\right) \tag{4.114}$$

where

b_{0s}	is the distance between centres of outstand headed studs in sagging moment region
b_{eis}	is the value of effective width of composite beam on each side of web of steel beam in sagging moment region
L_{es}	is the distance between inflection points in sagging moment region, see Figure 4.8
b_{is}	is the distance from outstand headed stud to a point mid-way between adjacent webs of steel beams in sagging moment region

(2) Effective width of composite beams in hogging moment region

The effective width of composite beams in hogging moment region b_{effh} which is an essential design parameter to evaluate the flexural rigidity and the moment resistance of the beams in the hogging moment region can be determined by the following equation.

$$b_{effh} = b_{0h} + \sum b_{eih} \tag{4.115}$$

b_{eih} is given by:

$$b_{eih} = \min\left(\frac{L_{eh}}{8}\ ;\ b_{ih}\right) \tag{4.116}$$

where

b_{0h}	is the distance between centres of outstand headed studs in hogging moment region
b_{eih}	is the value of effective width of composite beam on each side of web of steel beam in hogging moment region
L_{eh}	is the distance between inflection points in hogging moment region, see Figure 4.8
b_{ih}	is the distance from outstand headed stud to a point mid-way between adjacent webs of steel beams in hogging moment region

(3) Distance between inflection points

Because the distance between inflection points L_e may be different at ultimate limit state and serviceability limit state, the effective widths of composite beams should be evaluated for both the limit states individually.

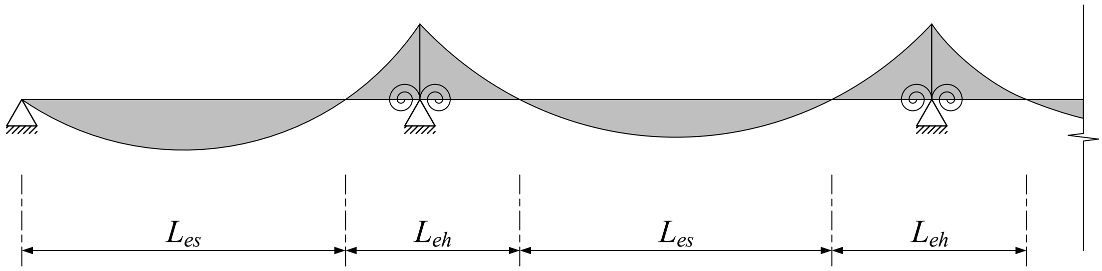


Figure 4.8: Distance between inflection points

Commentary:

(3) Distance between inflection points

According to the design criteria in subsection 4.2.1 (3), composite joints should be kept elastic at serviceability limit state in terms of the crack width. Therefore, structural analysis is carried out considering the rotational stiffness of the joints S_j as their initial rotational stiffness $S_{j,ini}$ as mentioned later in section 5.2. On the other hand, structural analysis should be performed assuming S_j as $S_{j,ini}/\eta$ in the case that the joints cannot be kept elastic at ultimate limit state. For that reason, the position of the inflection points may be different at ultimate limit state and serviceability limit state, so that the distance between the inflection points may be also different accordingly. That is, the effective widths of the composite beams should be evaluated for both the limit states individually.

4.4.2 Degree of Shear Connection

(1) Degree of shear connection in sagging moment region

The degree of shear connection in sagging moment region η_s can be determined by the following equation.

$$\eta_s = \frac{R_{qs}}{\min(R_a\ ;\ R_{cs})} \tag{4.117}$$

R_{qs}, R_a, and R_{cs} are given by:

$$R_{qs} = N_{hss} k_{ts} P_{Rd} \tag{4.118}$$

$$R_a = A_a f_{ayd} \tag{4.119}$$

$$R_{cs} = b_{effs} h_{cs} \left(0.85 f_{cd}\right) \tag{4.120}$$

where

$$k_{ts} = \min\left\{\frac{0.7}{\sqrt{\min(n_{hss}\ ;\ 2)}} \frac{b_{0,min}}{D_{ps}} \left(\frac{h_{hs}}{D_{ps}} -1\right)\ ;\ k_{ts,max}\right\} \tag{4.121}$$

$$f_{cd} = \frac{f_{ck}}{\gamma_c} \tag{4.122}$$

R_{qs}	is the longitudinal shear force transfer within half of L_{es}
R_a	is the tension (compression) resistance of steel beam
R_{cs}	is the compression resistance of composite slab within b_{effs}
L_{es}	is the distance between inflection points in sagging moment region, see Figure 4.8
b_{effs}	is the effective width of composite beams in sagging moment region, see 4.5.1
N_{hss}	is the number of headed studs arranged within half of L_{es}
P_{Rd}	is the shear resistance of a headed stud, see 2.4
A_a	is the cross-sectional area of steel beam
f_{ayd}	is the design yield strength of steel beam, see 4.2.2
h_{cs}	is the thickness of composite slab above profiled steel sheeting
k_{ts}	is the reduction factor for shear resistance of a headed stud in sagging moment region
n_{hss}	is the number of headed studs per sheeting rib in sagging moment region
$b_{0,min}$	is the minimum width for re-entrant of profiled steel sheeting
D_{ps}	is the overall depth of profiled steel sheeting
h_{hs}	is the overall hight of headed stud
$k_{ts,max}$	is the maximum reduction factor for shear resistance of a headed stud in sagging moment region, given in Table 4.7
f_{cd}	is the design strength of concrete
f_{ck}	is the characteristic cylinder strength of concrete
$\gamma_{c,cs}$	is the partial factor of concrete

Table 4.7: Maximum reduction factor for shear resistance of a headed stud $k_{t,max}$

Number of headed studs per sheeting rib n_{hs}	Thickness of profiled steel sheeting t_{ps} [mm]	Headed studs not exceeding 20[mm] in diameter of shank d_{hs} and welded through profiled steel sheeting	Profiled steel sheeting with holes and headed studs 19[mm] or 22[mm] in diameter of shank d_{hs}
1	≤ 1.0	0.85	0.75
	> 1.0	1.00	0.75
2	≤ 1.0	0.70	0.60
	> 1.0	0.80	0.60

(2) Degree of shear connection in hogging moment region

The degree of shear connection in hogging moment region η_h can be determined by the following equation.

$$\eta_h = \frac{R_{qh}}{\min(R_a \; ; R_{sl})} \tag{4.123}$$

R_{qh} and R_{sl} are given by:

$$R_{qh} = N_{hsh} k_{th} P_{Rd} \tag{4.124}$$

$$R_{sl} = A_{sl} f_{sd} \tag{4.125}$$

where

$$k_{th} = \min\left\{ \frac{0.7}{\sqrt{\min(n_{hsh} \; ; 2)}} \frac{b_{0,min}}{D_{ps}} \left(\frac{h_{hs}}{D_{ps}} - 1 \right) \; ; k_{th,max} \right\} \tag{4.126}$$

R_{qh} is the longitudinal shear force transfer within half of L_{eh}

R_{sl} is the tension resistance of longitudinal reinforcing bars within b_{effh}

L_{eh} is the distance between inflection points in hogging moment region, see Figure 4.8

b_{effh} is the effective width of composite beams in hogging moment region, see 4.4.1

N_{hsh} is the number of headed studs arranged within half of L_{eh}

A_{sl} is the cross-sectional area of longitudinal reinforcing bars within b_{effh}

f_{sd} is the design yield strength of reinforcing bars, see 4.3.3

k_{th} is the reduction factor for shear resistance of a headed stud in hogging moment region

n_{hsh} is the number of headed studs per sheeting rib in hogging moment region

$k_{th,max}$ is the maximum reduction factor for shear resistance of a headed stud in hogging moment region, given in Table 4.7

4.4.3 Shear Resistance

(1) Shear resistance in construction stage

a) Plastic shear resistance of steel beam

The plastic shear resistance of steel beam $V_{pl,a,Rd}$ can be determined by the following equation.

$$V_{pl,a,Rd} = A_V\left(\frac{f_{ayd}}{\sqrt{3}}\right) \tag{4.127}$$

A_V is given by:

$$A_V = \max\left\{A_a\text{-}2B_a t_f\text{+}(t_w\text{+}2r)t_f\,;\ 1.2\left(D_a\text{-}2t_f\right)t_w\right\} \tag{4.128}$$

where

f_{ayd}	is the design yield strength of steel beam, see 4.2.2
A_V	is the shear area of steel beam
A_a	is the cross-sectional area of steel beam
B_a	is the width of steel beam
t_f	is the flange thickness of steel beam
t_w	is the web thickness of steel beam
r	is the root radius of steel beam
D_a	is the depth of steel beam

b) Shear buckling resistance of steel beam

The shear buckling resistance of steel beam $V_{b,a,Rd}$ can be determined by the following equations.

$$V_{b,a,Rd} = V_{pl,a,Rd} \qquad \text{for } \frac{\left(D_a\text{-}2t_f\right)}{t_w} \le \frac{72}{1.2}\sqrt{\frac{235}{f_{ay}}} \tag{4.129}$$

$$V_{b,a,Rd} = \min\left\{\frac{\chi_w f_{wy}\left(D_a\text{-}2t_f\right)t_w}{\sqrt{3}\gamma_a}\ ;\ \frac{1.2 f_{wy}\left(D_a\text{-}2t_f\right)t_w}{\sqrt{3}\gamma_a}\right\} \qquad \text{for } \frac{\left(D_a\text{-}2t_f\right)}{t_w} > \frac{72}{1.2}\sqrt{\frac{235}{f_{ay}}} \tag{4.130}$$

χ_w is given by:

$$\chi_w = 1.2 \qquad \text{for } \lambda_w < \frac{0.83}{1.2} \tag{4.131}$$

$$\chi_w = \frac{0.83}{\lambda_w} \qquad \text{for } \frac{0.83}{1.2} \le \lambda_w \tag{4.132}$$

where

$$\lambda_w = 0.76\sqrt{\frac{f_{wy}}{k_{\tau,min}\left[190000\left\{\frac{t_w}{\left(D_a\text{-}2t_f\right)}\right\}^2\right]}} \tag{4.133}$$

$k_{\tau,min} = 5.34$ (without rigid transverse and longitudinal stiffeners) (4.134)

f_{wy} is the nominal value of yield strength of web of steel beam

χ_w is the factor for contribution of web of steel beam to shear buckling resistance

λ_w is the modified slenderness of web of steel beam

$k_{\tau,min}$ is the minimum shear buckling coefficient

(2) Shear resistance in composite stage

The plastic shear resistance and the shear buckling resistance of composite beam, $V_{pl,Rd}$ and $V_{b,Rd}$, can be equal to those of steel beam.

Commentary:

(2) Shear resistance in composite stage

The floor slab is not considered in resisting the vertical shear force. Therefore, the shear force is resisted by the web of the steel beams.

4.4.4 Moment Resistance

(1) Moment resistance in construction stage

a) Plastic moment resistance of steel beam

The plastic moment resistance of steel beam $M_{pl,a,Rd}$ can be determined by the following equations.

$$M_{pl,a,Rd} = W_{pl,a} f_{ayd} \quad \text{for Class 1 or Class 2 cross-sections} \tag{4.135}$$

$$M_{pl,a,Rd} = W_{eff,pl,a} f_{ayd} \quad \text{for effective Class 2 cross-sections} \tag{4.136}$$

where

$W_{pl,a}$	is the plastic section modulus of steel beam
f_{ayd}	is the design yield strength of steel beam, see 4.2.2
$W_{eff,pl,a}$	is the effective plastic section modulus of steel beam

b) Buckling moment resistance of steel beam

The buckling moment resistance of laterally unrestrained steel beam $M_{LT,a,Rd}$ can be determined by the following equation.

$$M_{LT,a,Rd} = \chi_{LT,a} M_{pl,a,Rd} \tag{4.137}$$

$\chi_{LT,a}$ is given by:

$$\chi_{LT,a} = \min\left(\frac{1}{\Phi_{LT,a} + \sqrt{\Phi_{LT,a}^{2} - 0.75\lambda_{LT,a}^{2}}} \; ; \; 1.0 \; ; \; \frac{1}{\lambda_{LT,a}^{2}} \right) \tag{4.138}$$

where

$$\Phi_{LT,a} = 0.5\left\{1 + \alpha_{LT}\left(\lambda_{LT,a} - 0.4\right) + 0.75\lambda_{LT,a}^{2}\right\} \tag{4.139}$$

$$\lambda_{LT,a} = \sqrt{\frac{M_{pl,a,Rd}}{M_{cr,a}}} \tag{4.140}$$

$$M_{cr,a} = C_1 \frac{\pi^2 E_a I_{az}}{L_{cr,a}^{2}} \sqrt{\frac{I_{w,a}}{I_{az}} + \frac{L_{cr,a}^{2} G_a I_{T,a}}{\pi^2 E_a I_{az}}} \tag{4.141}$$

$\chi_{LT,a}$	is the reduction factor for lateral-torsional buckling of steel beam
$\Phi_{LT,a}$	is the value to determine reduction factor for lateral-torsional buckling of steel beam
α_{LT}	is the imperfection factor corresponding to appropriate lateral-torsional buckling curve, recommended in Table 4.8
$\lambda_{LT,a}$	is the non-dimensional slenderness for lateral-torsional buckling of steel beam
$M_{cr,a}$	is the elastic critical moment for lateral-torsional buckling of steel beam
C_1	is the correction factor for non-uniform bending moment, which can be taken as 1.0 conservatively
E_a	is the modulus of elasticity of steel beam

I_{az} is the second moment of area of steel beam about minor axis (z-z axis)
$L_{cr,a}$ is the length of steel beam between points at which top flange of steel beam is laterally restrained
$I_{w,a}$ is the warping constant of steel beam
G_a is the shear modulus of elasticity of steel beam
$I_{T,a}$ is the torsion constant of steel beam

Table 4.8: Recommended values for imperfection factors for lateral-torsional buckling curve

Cross-section	Limits	Buckling curve	Imperfection factor α_{LT}
Rolled I-sections	$\frac{D_a}{B_a} \leq 2$	b	0.34
	$2 < \frac{D_a}{B_a} \leq 3.1$	c	0.49
	$3.1 < \frac{D_a}{B_a}$	d	0.76
Welded I-sections	$\frac{D_a}{B_a} \leq 2$	c	0.49
	$2 < \frac{D_a}{B_a}$	d	0.76

Note: D_a is the depth of steel beam
B_a is the width of steel beam

(2) Moment resistance in composite stage

a) Plastic sagging moment resistance of composite beam with full shear connection

The plastic sagging moment resistance of composite beam with full shear connection $M_{plf,Rds}$ can be determined by the following equations.

< Class 1 or Class 2 cross-sections >

$$M_{plf,Rds} = R_a \left\{ \frac{D_a}{2} + D_{cs} - \frac{R_a}{R_{cs}} \frac{\left(D_{cs} - D_{ps}\right)}{2} \right\} \tag{4.142}$$

for $R_a \leq R_{cs}$ (*PNA* in concrete flange)

$$M_{plf,Rds} = R_a \frac{D_a}{2} + R_{cs} \left(\frac{D_{cs} + D_{ps}}{2} \right) - \frac{(R_a - R_{cs})^2}{4 B_a f_{ayd}} \tag{4.143}$$

for $R_w \leq R_{cs} < R_a$ (*PNA* in steel flange)

$$M_{plf,Rds} = W_{pl,a} f_{ayd} + R_{cs} \left(\frac{D_a + D_{cs} + D_{ps}}{2} \right) - \frac{R_{cs}{}^2}{4 t_w f_{ayd}} \tag{4.144}$$

for $R_{cs} < R_w$ (*PNA* in steel web)

< Effective Class 2 cross-sections >

$$M_{plf,Rds} = R_a \left\{ \frac{D_a}{2} + D_{cs} - \frac{R_a}{R_{cs}} \frac{\left(D_{cs} - D_{ps}\right)}{2} \right\} \tag{4.145}$$

for $R_a \leq R_{cs}$ (*PNA* in concrete flange)

$$M_{plf,Rds} = R_a \frac{D_a}{2} + R_{cs} \left(\frac{D_{cs} + D_{ps}}{2} \right) - \frac{(R_a - R_{cs})^2}{4 B_a f_{ayd}} \tag{4.146}$$

for $R_{eff,v} \leq R_{cs} < R_a$ (*PNA* in steel flange)

$$M_{plf,Rds} = W_{pl,a} f_{ayd} + R_{cs} \left(\frac{D_a + D_{cs} + D_{ps}}{2} \right) - \frac{R_{cs}{}^2 + (R_v - R_{cs})\left(R_v - R_{cs} - 2R_{eff,v}\right)}{4 t_w f_{ayd}} \tag{4.147}$$

for $R_{cs} < R_{eff,v}$ (*PNA* in steel web)

R_a, R_{cs}, R_w, $R_{eff,v}$, and R_v are given by the following equations.

$$R_a = A_a f_{ayd} \tag{4.148}$$

$$R_{cs} = b_{effs} h_{cs} \left(0.85 f_{cd,cs} \right) \tag{4.149}$$

$$R_w = R_a - 2 B_a t_f f_{ayd} \tag{4.150}$$

$$R_{eff,v} = 40 t_w{}^2 f_{ayd} \sqrt{\frac{235}{f_{ayd}}} \tag{4.151}$$

$$R_v = \left\{ D_a - 2\left(t_f + r\right) \right\} t_w f_{ayd} \tag{4.152}$$

where

D_{cs}	is the overall depth of composite slab
D_{ps}	is the overall depth of profiled steel sheeting
t_w	is the web thickness of steel beam
R_a	is the tension (compression) resistance of steel beam
R_{cs}	is the compression resistance of composite slab within b_{effs}
R_w	is the tension (compression) resistance of overall web of steel beam
$R_{eff,v}$	is the tension (compression) resistance of effective clear web of steel beam
R_v	is the tension (compression) resistance of clear web of steel beam
b_{effs}	is the effective width of composite beams in sagging moment region, see 4.4.1
A_a	is the cross-sectional area of steel beam
h_{cs}	is the thickness of composite slab above profiled steel sheeting
$f_{cd,cs}$	is the design strength of concrete for concrete slab, see 4.4.2
t_f	is the flange thickness of steel beam
r	is the root radius of steel beam

b) Plastic sagging moment resistance of composite beam with partial shear connection

The plastic sagging moment resistance of composite beam with partial shear connection $M_{plp,Rds}$ can be determined by the following equations.

< Class 1 or Class 2 cross-sections >

$$M_{plp,Rds} = M_{plf,Rds} \quad \text{for } \eta_s \geq 1 \tag{4.153}$$

$$M_{plp,Rds} = R_a \frac{D_a}{2} + R_{qs}\left(D_{cs} - \frac{R_{qs}}{R_{cs}} \frac{D_{cs}-D_{ps}}{2}\right) - \frac{\left(R_a - R_{qs}\right)^2}{4B_a f_{ayd}} \tag{4.154}$$

for $\eta_s < 1$ and $R_w \leq R_{qs}$ (*PNA* in steel flange)

$$M_{plp,Rds} = W_{pl,a} f_{ayd} + R_{qs}\left(\frac{D_a}{2} + D_{cs} - \frac{R_{qs}}{R_{cs}} \frac{D_{cs}-D_{ps}}{2}\right) - \frac{R_{qs}^{\;2}}{4t_w f_{ayd}} \tag{4.155}$$

for $\eta_s < 1$ and $R_{qs} < R_w$ (*PNA* in steel web)

< Effective Class 2 cross-sections >

$$M_{plp,Rds} = M_{plf,Rds} \quad \text{for } \eta_s \geq 1 \tag{4.156}$$

$$M_{plp,Rds} = R_a \frac{D_a}{2} + R_{qs}\left(D_{cs} - \frac{R_{qs}}{R_{cs}} \frac{D_{cs}-D_{ps}}{2}\right) - \frac{\left(R_a - R_{qs}\right)^2}{4B_a f_{ayd}} \tag{4.157}$$

for $\eta_s < 1$ and $R_{eff,v} \leq R_{qs}$ (*PNA* in steel flange)

$$M_{plp,Rds} = W_{pl,a} f_{ayd} + R_{qs}\left(\frac{D_a}{2} + D_{cs} - \frac{R_{qs}}{R_{cs}} \frac{D_{cs}-D_{ps}}{2}\right) - \frac{R_{qs}^{\;2} + \left(R_v - R_{qs}\right)\left(R_v - R_{qs} - 2R_{eff,v}\right)}{4t_w f_{ayd}} \tag{4.158}$$

for $\eta_s < 1$ and $R_{qs} < R_{eff,v}$ (*PNA* in steel web)

where

η_s	is the degree of shear connection in sagging moment region, see 4.4.2
R_{qs}	is the longitudinal shear force transfer within the half of L_{es}, see 4.4.2

c) Plastic hogging moment resistance of composite beam with full shear connection

The plastic hogging moment resistance of composite beam with full shear connection $M_{plf,Rdh}$ can be determined by the following equations.

< Class 1 or Class 2 cross-sections >

$$M_{plf,Rdh} = R_a \left(\frac{D_a}{2} + z_{csl\text{-}tf}\right) \tag{4.159}$$

for $R_a \leq R_{sl}$ (*PNA* outside steel beam)

$$M_{plf,Rdh} = R_a \frac{D_a}{2} + R_{sl} z_{csl\text{-}tf} - \frac{(R_a - R_{sl})^2}{4B_a f_{ayd}} \tag{4.160}$$

for $R_w \leq R_{sl} < R_a$ (*PNA* in steel flange)

$$M_{plf,Rdh} = W_{pl,a} f_{ayd} + R_{sl}\left(\frac{D_a}{2} + z_{csl\text{-}tf}\right) - \frac{R_{sl}^{\;2}}{4t_w f_{ayd}} \tag{4.161}$$

for $R_{sl} < R_w$ (*PNA* in steel web)

< Effective Class 2 cross-sections >

$$M_{plf,Rdh} = R_{eff,a}\left(\frac{D_a}{2}+z_{csl\text{-}tf}\right) \tag{4.162}$$

for $R_{eff,a} \leq R_{sl}$ (*PNA* outside steel beam)

$$M_{plf,Rdh} = R_{eff,a}\frac{D_a}{2}+R_{sl}z_{csl\text{-}tf}-\frac{\left(R_{eff,a}\text{-}R_{sl}\right)^2}{4B_af_{ayd}} \tag{4.163}$$

for $R_{eff,v} \leq R_{sl} < R_{eff,a}$ (*PNA* in steel flange)

$$M_{plf,Rdh} = W_{pl,a}f_{ayd}+R_{sl}\left(\frac{D_a}{2}+z_{csl\text{-}tf}\right)-\frac{R_{sl}{}^2+(R_v+R_{sl})\left(R_v+R_{sl}\text{-}2R_{eff,v}\right)}{4t_wf_{ayd}} \tag{4.164}$$

for $R_{sl} < R_{eff,v}$ (*PNA* in steel web)

$R_{eff,a}$ is given by:

$$R_{eff,a} = R_a\text{-}R_v+R_{eff,v} \tag{4.165}$$

where

$z_{csl\text{-}tf}$	is the vertical distance between centre of longitudinal reinforcing bars and top of flange of steel beam
R_{sl}	is the tension resistance of longitudinal reinforcing bars within b_{effh}, see 4.4.2
$R_{eff,a}$	is the tension (compression) resistance of effective steel beam

d) Reduced hogging moment resistance of composite beam making allowance for presence of shear force

The reduced hogging moment resistance of composite beam making allowance for presence of shear force $M_{y,v,Rdh}$ can be determined by the following equations.

$$M_{y,v,Rdh} = M_{plf,Rdh} \qquad \text{for } V_{Ed} \leq \frac{V_{pl,Rd}}{2} \tag{4.166}$$

$$M_{y,v,Rdh} = M_{plf,Rdh}\text{-}\left(M_{plf,Rdh}\text{-}M_{pl,f,Rd}\right)\left(\frac{2V_{Ed}}{V_{pl,Rd}}\text{-}1\right)^2 \qquad \text{for } V_{Ed} > \frac{V_{pl,Rd}}{2} \tag{4.167}$$

$M_{pl,f,Rd}$ is given by:

$$M_{pl,f,Rd} = 2R_f\left(\frac{D_a}{2}+z_{csl\text{-}tf}\right) \qquad \text{for } PNA \text{ outside steel beam} \tag{4.168}$$

$$M_{pl,f,Rd} = R_fD_a+R_{sl}z_{csl\text{-}tf}-\frac{\left(2R_f\text{-}R_{sl}\right)^2}{4B_af_{ayd}} \qquad \text{for } PNA \text{ in steel flange} \tag{4.169}$$

$$M_{pl,f,Rd} = R_fz_{ctf\text{-}cbf}+R_{sl}\left(\frac{D_a}{2}+z_{csl\text{-}tf}\right) \qquad \text{for } PNA \text{ in steel web} \tag{4.170}$$

where

$$R_f = B_at_ff_{ayd} \tag{4.171}$$

V_{Ed}	is the design shear force
$V_{pl,Rd}$	is the plastic shear resistance of composite beam, see 4.4.3

$V_{b,Rd}$ is the shear buckling resistance of composite beam, see 4.4.3

$M_{pl,f,Rd}$ is the plastic moment resistance of composite beam after deducting shear area

R_f is the tension (compression) resistance of flange of steel beam

$z_{ctf\text{-}cbf}$ is the vertical distance between centres of top and bottom flanges of steel beam

e) Buckling moment resistance of laterally unrestrained composite beam

As far as Eq.(4.172) and (4.173) are satisfied, the buckling moment resistance of laterally unrestrained composite beam $M_{LT,Rd}$ can be determined by Eq.(4.174).

$$E_{cm,cs}I_{cs2} \geq 0.35 E_a t_w{}^2 B_b / D_a \tag{4.172}$$

$$p_{ps}/B_a \leq 0.4 f_{hsu} d_{hs}{}^2 \frac{1\text{-}\chi_{LT}\lambda_{LT}{}^2}{k_s \chi_{LT}\lambda_{LT}{}^2} \tag{4.173}$$

$$M_{LT,Rd} = \chi_{LT} M_{plf,Rdh} \tag{4.174}$$

I_{cs2}, χ_{LT}, λ_{LT}, and k_s are given by:

$$I_{cs2} = A_{st} z_{cst\text{-}na}{}^2 + A_{c,c}\left(z_{na\text{-}ccs,c}{}^2 + \frac{D_{ps}{}^2}{12}\right) \tag{4.175}$$

$$\chi_{LT} = \min\left(\frac{1}{\Phi_{LT} + \sqrt{\Phi_{LT}{}^2 \text{-} 0.75\lambda_{LT}{}^2}}\ ;\ 1.0\ ;\ \frac{1}{\lambda_{LT}{}^2}\right) \tag{4.176}$$

$$\lambda_{LT} = \sqrt{\frac{M_{pl,Rkh}}{M_{cr}}} \tag{4.177}$$

where

$$k_s = \frac{k_1 k_2}{k_1 + k_2} \tag{4.178}$$

$$\Phi_{LT} = 0.5\left\{1 + \alpha_{LT}(\lambda_{LT}\text{-}0.4) + 0.75\lambda_{LT}{}^2\right\} \tag{4.179}$$

$$M_{cr} = \frac{k_c C_4}{L_{cr}} \sqrt{\left[\left\{\frac{E_a}{2(1+0.3)}\right\} I_{T,a} + \frac{k_s L_{cr}{}^2}{\pi^2}\right] E_a I_{bfz}} \tag{4.180}$$

$$k_c = \frac{\left(\dfrac{z_{ctf\text{-}cbf} I_h}{I_{ay}}\right)}{\left(\dfrac{z_{ctf\text{-}cbf}{}^2}{4} + i_{ax}{}^2\right)/e + z_{ctf\text{-}cbf}} \tag{4.181}$$

$$I_h = I_{ay} + \frac{A_a A_{sl}\left\{D_a + 2\left(D_{ps} + h_{cs} \text{-} z_{tcs\text{-}csl}\right)\right\}^2}{4(A_a + A_{sl})} \tag{4.182}$$

$$i_{ax} = \sqrt{\frac{I_{ay} + I_{az}}{A_a}} \tag{4.183}$$

$$e = \frac{(A_a+A_{sl})I_{ay}}{A_a z_{ccs\text{-}ca} A_{sl}} \tag{4.184}$$

$$I_{bfz} = \frac{t_f B_a^{\ 3}}{12} \tag{4.185}$$

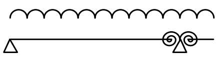
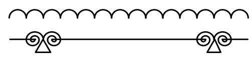
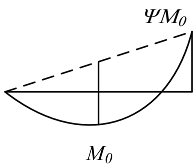
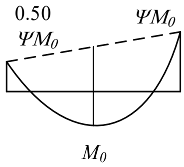
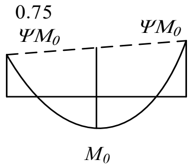
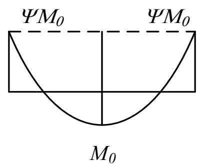
$$k_1 = \frac{4E_a I_{cs2}}{B_b} \tag{4.186}$$

$$k_2 = \frac{E_a t_w^{\ 3}}{4\left(1\text{-}0.3^2\right) z_{ctf\text{-}cbf}} \tag{4.187}$$

B_b	is the beam spacing
p_{ps}	is the pitch of ribs of profiled steel sheeting
f_{hsu}	is the ultimate strength of headed stud
d_{hs}	is the diameter of shank of headed stud
χ_{LT}	is the reduction factor for lateral-torsional buckling of composite beam
I_{cs2}	is the second moment of area of cracked composite slab in direction transverse to steel beam
λ_{LT}	is the non-dimensional slenderness for lateral-torsional buckling of composite beam
k_s	is the transverse (rotational) stiffness per unit length of composite beam
Φ_{LT}	is the value to determine reduction factor for lateral-torsional buckling of composite beam
A_{st}	is the cross-sectional area of transverse reinforcing bars per unit length
$z_{cst\text{-}na}$	is the vertical distance between centre of transverse reinforcing bars and neutral axis of composite slab
$A_{c,c}$	is the area per unit length of concrete slab in compression
$z_{na\text{-}ccs,c}$	is the vertical distance between neutral axis of composite slab and centre of concrete slab in compression
$M_{pl,Rkh}$	is the characteristic value of plastic hogging moment resistance of composite beam calculated by Eq.(4.159) to (4.164) using the characteristic yield strength instead of the design yield strength of reinforcing bars
M_{cr}	is the elastic critical moment for lateral-torsional buckling of composite beam
C_4	is the property of distribution of moment, given in Table 4.9
L_{cr}	is the length of composite beam between points at which bottom flange is laterally restrained
k_c	is the k_c factor
I_{ay}	is the second moment of area of steel beam about y-y axis
I_h	is the second moment of area of composite beam in hogging moment region
A_{sl}	is the cross-sectional area of longitudinal reinforcing bars within b_{effh}
$z_{tcs\text{-}csl}$	is the covering depth of longitudinal reinforcing bars
i_{ax}	is the polar radius of gyration of area of steel beam
e	is the e value

$z_{ccs\text{-}ca}$	is the vertical distance between centre of composite slab and steel beam
I_{bfz}	is the second moment of area of bottom flange of steel beam about minor axis (z-z axis)
k_1	is the flexural stiffness of cracked composite slab in direction transverse to steel beam
k_2	is the flexural stiffness of web of steel beam

Table 4.9: Values of factor C_4 for spans with transverse loading

Loading and support conditions	External beam 	Internal beam 		
Moment diagram				
Ψ = 0.50	41.5	33.9	28.2	21.9
Ψ = 0.75	30.2	22.7	18.0	13.9
Ψ = 1.00	24.5	17.3	13.7	11.0
Ψ = 1.25	21.1	14.1	11.7	9.6
Ψ = 1.50	19.0	13.0	10.6	8.8
Ψ = 1.75	17.5	12.0	10.0	8.3
Ψ = 2.00	16.5	11.4	9.5	8.0
Ψ = 2.25	15.7	10.9	9.1	7.8
Ψ = 2.50	15.2	10.6	8.9	7.6

Note: Ψ is the ratio of the design hogging moment to M_0
M_0 is the mid-length moment of simply supported beam

Commentary:

(1) Moment resistance in construction stage

Generally, Class 3 cross-sections would assume an elastic distribution of stresses, and the moment resistance can be calculated using their elastic section modulus. However, EN 1993-1-1 makes special allowances for the cross-sections with Class 3 web and Class 1 or 2 flanges by permitting the cross-sections to be classified as effective Class 2 cross-sections. Accordingly, part of the web in compression is neglected, and the plastic section properties for the remainder of the cross-section can be determined. In EN 1993-1-1, the proportion of the web in compression should be replaced by a part of $20\varepsilon t_w$ adjacent to the compression flange measured from the base of the root radius, with another part of $20\varepsilon t_w$ adjacent to the plastic neutral axis of the effective cross-section in accordance with Figure 4.9. A similar distribution can be applied to welded sections with the part of $20\varepsilon t_w$ adjacent to the compression flange measured from the base of the weld.

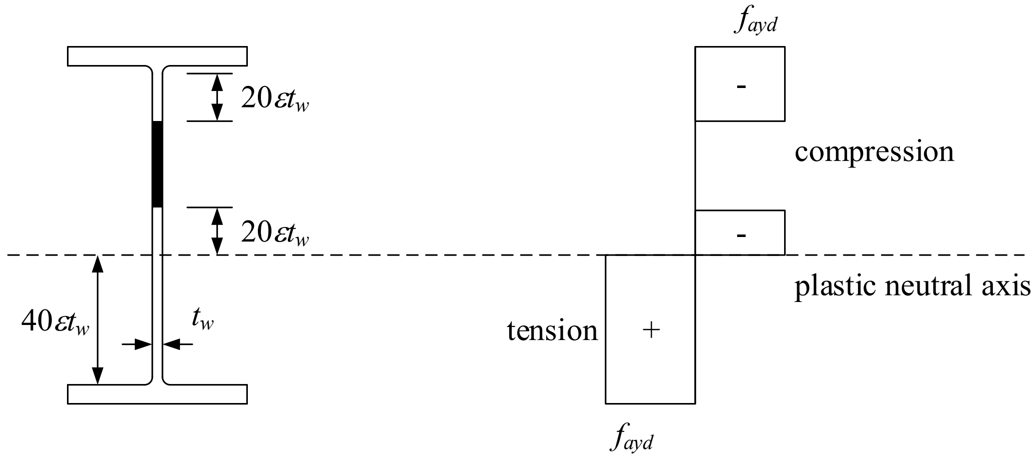


Figure 4.9: Effective Class 2 web

4.4.5 Longitudinal Shear Resistance

(1) Tension resistance of transverse reinforcement per unit length

The tension resistance of transverse reinforcement per unit length $R_{st}+R_{pse}$ can be determined by the following equation.

$$R_{st}+R_{pse}=A_{st}f_{sd}+A_{pse}f_{psd} \tag{4.188}$$

f_{psd} is given by:

$$f_{psd}=\frac{f_{psk}}{\gamma_{ps}} \tag{4.189}$$

where

A_{st}	is the cross-sectional area of transverse reinforcing bars per unit length
f_{sd}	is the design yield strength of reinforcing bars, see 4.3.3
A_{pse}	is the effective cross-sectional area of profiled steel sheeting per unit length
f_{psd}	is the design yield strength of profiled steel sheeting
f_{psk}	is the characteristic yield strength of profiled steel sheeting
γ_{ps}	is the partial factor of profiled steel sheeting

(2) Crushing shear stress of concrete slab

The crushing shear stress of concrete slab v_{Rd} can be determined by Eq.(4.190).

$$v_{Rd}=0.6\left(1-\frac{f_{ck}}{250}\right)f_{cd}\sin\theta\cos\theta \tag{4.190}$$

where

f_{ck}	is the characteristic cylinder strength of concrete
f_{cd}	is the design strength of concrete, see 4.4.2
θ	is the angle between diagonal strut and axis of beam, $26.5° \le \theta \le 45°$ for concrete flange in compression, $38.6° \le \theta \le 45°$ for concrete flange in tension

Chapter 5 Structural Analysis

5.1 General

(1) Concept

In structural analysis, semi-rigid composite joints should be modelled as rotational springs at beam ends. Design moment and deflection of composite beams with semi-rigid ends and other supporting members, such as beams, walls and columns, should be analysed considering the moment-rotation characteristics of the rotational springs. Note that the joints shall be subjected to hogging moment so that the contact parts of the joints are always in compression.

(2) Analysis method

Elastic-plastic analysis is recommended as the analysis method. In this method, the moment-rotation characteristics (M_j-ϕ_j curves) of the rotational springs should be simplified to yield conservative prediction on the beam responses.

Commentary:

(1) Concept

When the structural behaviour of building frames is affected by the joint structural properties, structural analysis is generally carried out modelling the joints as rotational springs. In this design guide, composite beams are designed with semi-rigid ends, so that the design moment and deflection of the beam members are affected by the rotational stiffness and the moment resistance of the joints. Therefore, composite joints should be modelled as rotational springs and then the design moment and deflection of composite beams with semi-rigid ends should be analysed in structural analysis. The contact parts of composite joints shall always be under compression; otherwise, the joints cannot perform as semi-rigid. In other words, composite joints shall be subjected to hogging moment at both ultimate and serviceability limit states.

(2) Analysis method

In this design guide, elastic-plastic analysis is recommended as the analysis method because structural resistance of composite beams at ultimate limit state can be checked utilizing the rotational capacity of the composite joints. The structural properties of the joints can be expressed in the form of moment-rotation characteristics (M_j-ϕ_j curves). However, they are often non-linear in most cases as the joint rotation occurs due to the deformation of each joint component, and it is not practical to accurately consider the nonlinearity in structural analysis. Therefore, M_j-ϕ_j curves of the rotational springs should be simplified to yield conservative prediction on the beam responses.

5.2 Structural Modelling of Composite Joint

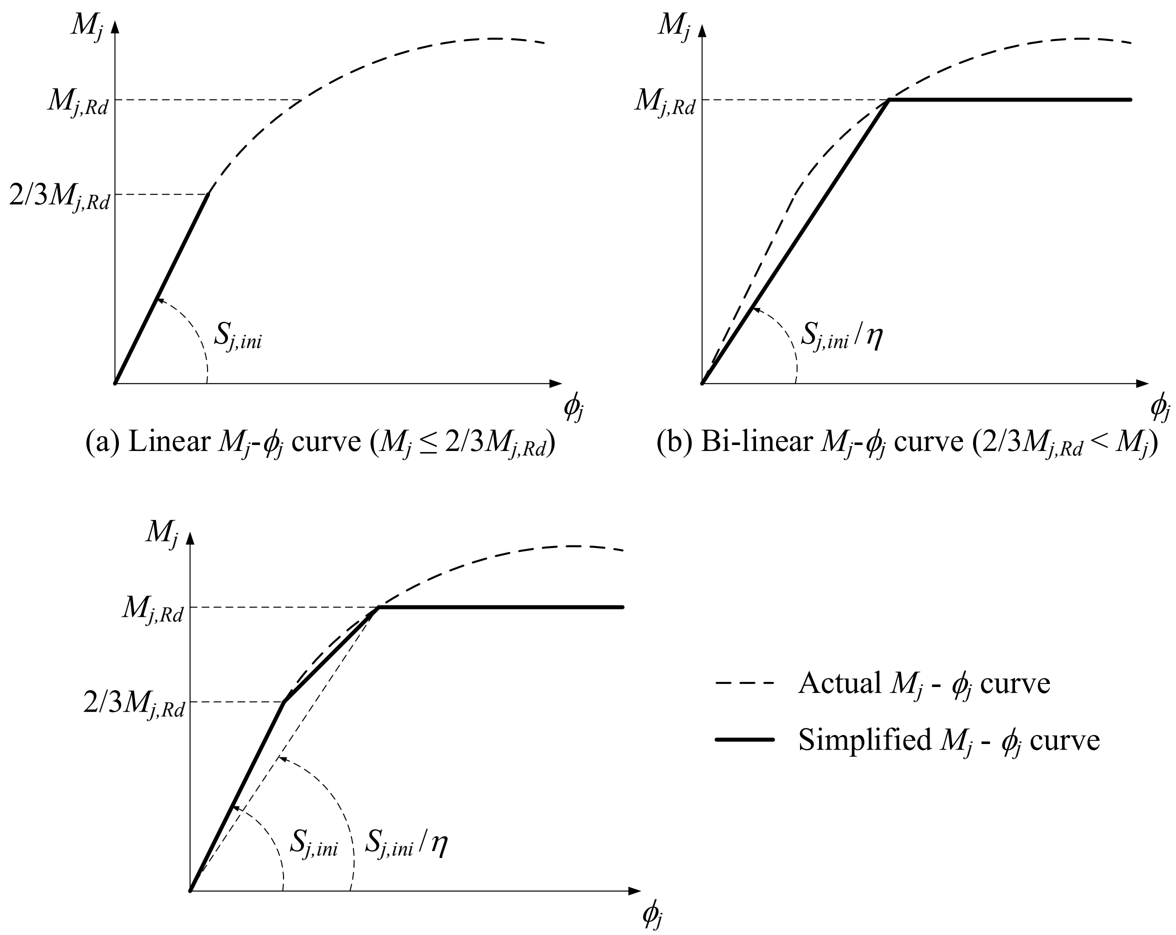
(1) Simplified moment-rotation characteristics

Unless more accurate methods are provided, the following simplified M_j-ϕ_j curves can be applied to the rotational springs.

a) When the joint moment M_j is less than or equal to the elastic moment resistance $2/3M_{j,Rd}$, the linear M_j-ϕ_j curve with the rotational stiffness S_j taken as the initial rotational stiffness $S_{j,ini}$ can be applied as shown in Figure 5.1 (a). $M_{j,Rd}$ is the moment resistance of the connection.

b) When the joint moment M_j is more than the elastic moment resistance $2/3M_{j,Rd}$, the bi-linear M_j-ϕ_j curve with the rotational stiffness S_j taken as $S_{j,ini}/\eta$ can be applied as shown in Figure 5.1 (b). The stiffness modification coefficient η for composite joints with contact plates should be taken as 1.5.

c) Besides the aboves, the tri-linear M_j-ϕ_j curve combining a) and b) can be also applied as shown in Figure 5.1 (c).



(a) Linear M_j-ϕ_j curve ($M_j \leq 2/3M_{j,Rd}$)

(b) Bi-linear M_j-ϕ_j curve ($2/3M_{j,Rd} < M_j$)

(c) Tri-linear M_j-ϕ_j curve

Figure 5.1: Simplified moment-rotation characteristics (M_j-ϕ_j curves)

Commentary:

(1) Simplified moment-rotation characteristics

In EN 1993-1-8, the simplified M_j-ϕ_j curves shown in Figure 5.1 (a) and (b) are recommended for the rotational springs in elastic-plastic analysis. According to this simplified M_j-ϕ_j curves, the rotational stiffness S_j can be taken as the initial rotational stiffness $S_{j,ini}$ when the joint moment M_j is less than or equal to the elastic moment resistance $2/3M_{j,Rd}$ as shown in Figure 5.1 (a), where $M_{j,Rd}$ is the joint's moment resistance. On the other hand, the rotational stiffness S_j can be taken as $S_{j,ini}/\eta$ when the joint moment M_j is more than the elastic moment resistance $2/3M_{j,Rd}$ as shown in Figure 5.1 (b). The latter S_j is the intermediate value between $S_{j,ini}$ and the secant stiffness of the moment resistance $M_{j,Rd}$, and this is the convenient constant value of S_j which actually changes due to the nonlinear behaviour. The stiffness modification coefficient η for composite joints with contact plates is proposed as 1.5 in EN 1994-1-1. Besides the aboves, the simplified M_j-ϕ_j curve shown in Figure 5.1 (c) combining (a) and (b) can be also applied to get closer to the actual M_j-ϕ_j curve.

5.3 Design Moment and Deflection of Composite Beam

(1) Concept

The effects of cracking of floor slab should be considered in the structural analysis of composite beams with semi-rigid ends.

(2) Calculation of design moment and deflection

As flexural rigidity of composite beams in the hogging moment region may be smaller than that in the sagging moment region because of cracking of floor slab, these beams should be designed as non-uniform sections with different flexural rigidities within the same beam span. When uniformly distributed load is considered, the design moment and deflection of composite beams with semi-rigid ends can be calculated by the following equations. In this method, however, if the joint moment M_j is more than the moment resistance of the composite joint $M_{j,Rd}$, M_j should be reduced to $M_{j,Rd}$ and the corresponding redistributed moment should be taken as the design moment.

a) Composite beam supported by composite joints with equal rotational stiffness

$$M(x) = -\frac{w}{2}x^2+\frac{wL_b}{2}x-M_j \tag{5.1}$$

$$\delta_{max} = \frac{1}{(EI)_h}\left(A(x_0)\frac{L_b}{2}-C(x_0)\right)+\frac{1}{(EI)_s}\left(B\left(\frac{L_b}{2}\right)-A(x_0)\frac{L_b}{2}+C(x_0)\right)+\frac{M_j}{S_j}\cdot\frac{L_b}{2} \tag{5.2}$$

M_j is given by Eq.(4.195) using convergence calculation.

$$\frac{1}{(EI)_h}A(x_0)+\frac{1}{(EI)_s}\left(A\left(\frac{L_b}{2}\right)-A(x_0)\right)+\frac{M_j}{S_j} = 0 \tag{5.3}$$

where

$$(EI)_h = E_a\left[I_{ay}+\frac{A_aA_{sl}\{D_a+2(D_{ps}+h_{cs}-z_{tcs\text{-}csl})\}^2}{4(A_a+A_{sl})}\right] \tag{5.4}$$

$$(EI)_s = E_a\left\{\frac{A_a(D_{cs}+D_{ps}+D_a)^2}{4\left(1+\frac{n_0A_a}{b_{effs}h_{cs}}\right)}+\frac{b_{effs}h_{cs}^{\ 3}}{12\left(\frac{2E_a}{E_{cm,cs}}\right)}+I_{ay}\right\} \tag{5.5}$$

$$A(x) = \frac{w}{6}x^3-\frac{wL_b}{4}x^2+M_jx \tag{5.6}$$

$$B(x) = \frac{w}{24}x^4-\frac{wL_b}{12}x^3+\frac{M_j}{2}x^2 \tag{5.7}$$

$$C(x) = \frac{w}{8}x^4-\frac{wL_b}{6}x^3+\frac{M_j}{2}x^2 \tag{5.8}$$

$$x_0 = \frac{L_b}{2}\left(1-\sqrt{1-\frac{8M_j}{wL_b^{\ 2}}}\right) \tag{5.9}$$

$M(x)$ is the moment of composite beam along x-axis

w is the uniformly distributed load

L_b	is the beam length
δ_{max}	is the maximum deflection of composite beam
S_j	is the rotational stiffness of composite joint
M_j	is the joint moment
$(EI)_h$	is the hogging flexural rigidity of composite beam
$(EI)_s$	is the sagging flexural rigidity of composite beam
E_a	is the modulus of elasticity of steel beam
I_{ay}	is the second moment of area of steel beam about major axis (y-y axis)
A_a	is the cross-sectional area of steel beam
A_{sl}	is the cross-sectional area of longitudinal reinforcing bars within b_{effh}
b_{effh}	is the effective width of composite beam in hogging moment region, see 4.4.1
D_a	is the depth of steel beam
D_{ps}	is the overall depth of profiled steel sheeting
h_{cs}	is the thickness of composite slab above profiled steel sheeting
$z_{tcs\text{-}csl}$	is the covering depth of longitudinal reinforcing bars
D_{cs}	is the overall depth of composite slab
b_{effs}	is the effective width of composite beam in sagging moment region, see 4.4.1
$E_{cm,cs}$	is the secant modulus of elasticity of concrete for concrete slab
x_0	is the x-coordinate at inflection point

b) Composite beam supported by composite joint at one end

$$M(x) = -\frac{w}{2}x^2+\left(\frac{wL_b}{2}-\frac{M_j}{L_b}\right)x \tag{5.10}$$

$$\delta_{max} = \frac{1}{(EI)_s}E(x_\delta)+\theta_{pin}x_\delta \tag{5.11}$$

M_j and x_δ are given by Eq.(4.204) and (4.205) using convergence calculation.

$$\frac{1}{(EI)_h}\left(D(L_b)-D(x_0)\right)+\frac{1}{(EI)_s}D(x_0)+\theta_{pin}+\frac{M_j}{S_j} = 0 \tag{5.12}$$

$$\frac{1}{(EI)_s}D(x_\delta)+\theta_{pin} = 0 \tag{5.13}$$

where

$$D(x) = \frac{w}{6}x^3-\frac{1}{2}\left(\frac{wL_b}{2}-\frac{M_j}{L_b}\right)x^2 \tag{5.14}$$

$$E(x) = \frac{w}{24}x^4-\frac{1}{6}\left(\frac{wL_b}{2}-\frac{M_j}{L_b}\right)x^3 \tag{5.15}$$

$$F(x) = \frac{w}{8}x^4-\frac{1}{3}\left(\frac{wL_b}{2}-\frac{M_j}{L_b}\right)x^3 \tag{5.16}$$

$$x_0 = L_b-\frac{2M_j}{wL_b} \tag{5.17}$$

$$\theta_{pin} = -\frac{1}{(EI)_h L_b}\left(E(L_b)\text{-}D(x_0)L_b+F(x_0)\right)-\frac{1}{(EI)_s L_b}\left(D(x_0)L_b\text{-}F(x_0)\right) \quad (5.18)$$

x_δ is the x-coordinate where deflection is maximized

θ_{pin} is the rotation of pin joint

c) Composite beam supported by composite joints with unequal rotational stiffness

$$M(x) = -\frac{w}{2}x^2+\left(\frac{wL_b}{2}+\frac{M_{j,l}\text{-}M_{j,r}}{L_b}\right)x\text{-}M_{j,l} \quad (5.19)$$

$$\delta_{max} = \frac{1}{(EI)_{h,l}}\left(G(x_0)x_\delta\text{-}I(x_0)\right)+\frac{1}{(EI)_s}\left(H(x_\delta)\text{-}G(x_0)x_\delta+I(x_0)\right)+\frac{M_{j,l}}{S_{j,l}}x_\delta \quad (5.20)$$

M_j and x_δ are given by the Eq.(4.213), (4.214), and (4.215) using convergence calculation.

$$\frac{1}{(EI)_{h,l}}G(x_0)+\frac{1}{(EI)_s}\left(G(x_0')\text{-}G(x_0)\right)+\frac{1}{(EI)_{h,r}}\left(G(L_b)\text{-}G(x_0')\right)+\frac{M_{j,l}}{S_{j,l}}+\frac{M_{j,r}}{S_{j,r}} = 0 \quad (5.21)$$

$$\left\{\begin{aligned}&\frac{1}{(EI)_{h,l}}\left(G(x_0)L_b\text{-}I(x_0)\right)+\frac{1}{(EI)_s}\left(G(x_0')L_b\text{-}G(x_0)L_b\text{-}I(x_0')+I(x_0)\right)\\&\quad+\frac{1}{(EI)_{h,r}}\left(H(L_b)\text{-}G(x_0')L_b+I(x_0')\right)+\frac{M_{j,l}}{S_{j,l}}L_b\end{aligned}\right\} = 0 \quad (5.22)$$

$$\frac{1}{(EI)_{h,l}}G(x_0)+\frac{1}{(EI)_s}\left(G(x_\delta)\text{-}G(x_0)\right)+\frac{M_{j,l}}{S_{j,l}} = 0 \quad (5.23)$$

where

$$G(x) = \frac{w}{6}x^3\text{-}\frac{1}{2}\left(\frac{wL_b}{2}+\frac{M_{j,l}\text{-}M_{j,r}}{L_b}\right)x^2+M_{j,l}x \quad (5.24)$$

$$H(x) = \frac{w}{24}x^4\text{-}\frac{1}{6}\left(\frac{wL_b}{2}+\frac{M_{j,l}\text{-}M_{j,r}}{L_b}\right)x^3+\frac{M_{j,l}}{2}x^2 \quad (5.25)$$

$$I(x) = \frac{w}{8}x^4\text{-}\frac{1}{3}\left(\frac{wL_b}{2}+\frac{M_{j,l}\text{-}M_{j,r}}{L_b}\right)x^3+\frac{M_{j,l}}{2}x^2 \quad (5.26)$$

$$x_0 = \frac{1}{w}\left\{\left(\frac{wL_b}{2}+\frac{M_{j,l}\text{-}M_{j,r}}{L_b}\right)\text{-}\sqrt{\left(\frac{wL_b}{2}+\frac{M_{j,l}\text{-}M_{j,r}}{L_b}\right)^2\text{-}2wM_{j,l}}\right\} \quad (5.27)$$

$$x_0' = \frac{1}{w}\left\{\left(\frac{wL_b}{2}+\frac{M_{j,l}\text{-}M_{j,r}}{L_b}\right)+\sqrt{\left(\frac{wL_b}{2}+\frac{M_{j,l}\text{-}M_{j,r}}{L_b}\right)^2\text{-}2wM_{j,l}}\right\} \quad (5.28)$$

$S_{j,l}$ is the rotational stiffness of composite joint at left side

$S_{j,r}$ is the rotational stiffness of composite joint at right side

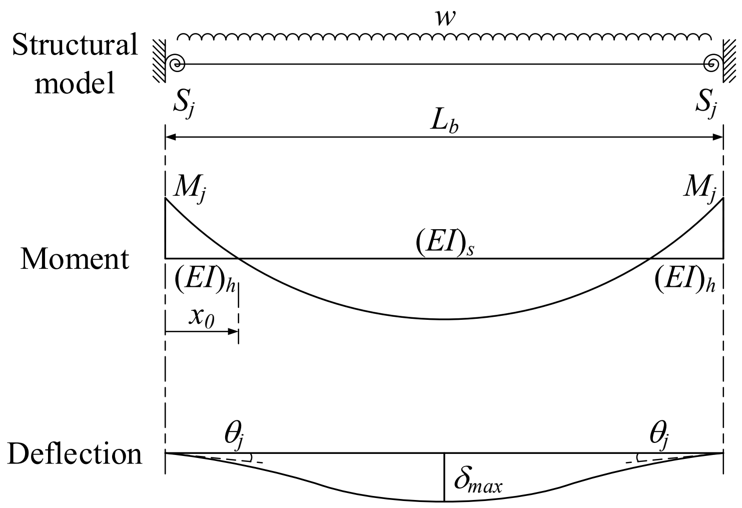
$M_{j,l}$ is the joint moment at left side

$M_{j,r}$ is the joint moment at right side

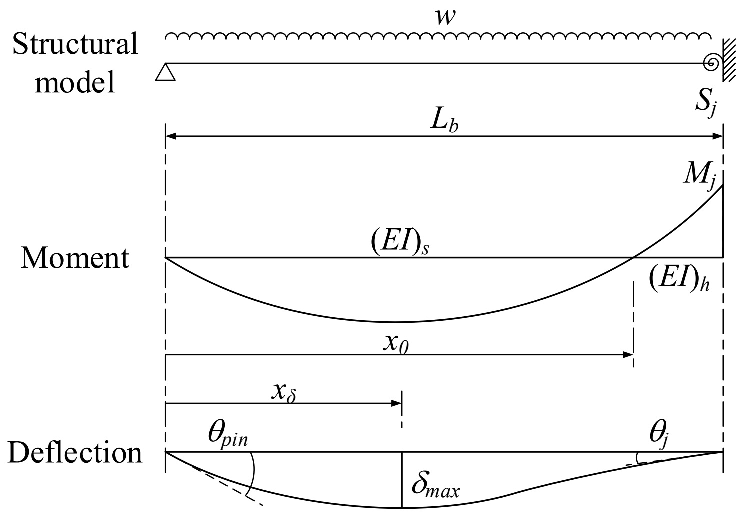
$(EI)_{h,l}$ is the hogging flexural rigidity of composite beam at left side

$(EI)_{h,r}$ is the hogging flexural rigidity of composite beam at right side

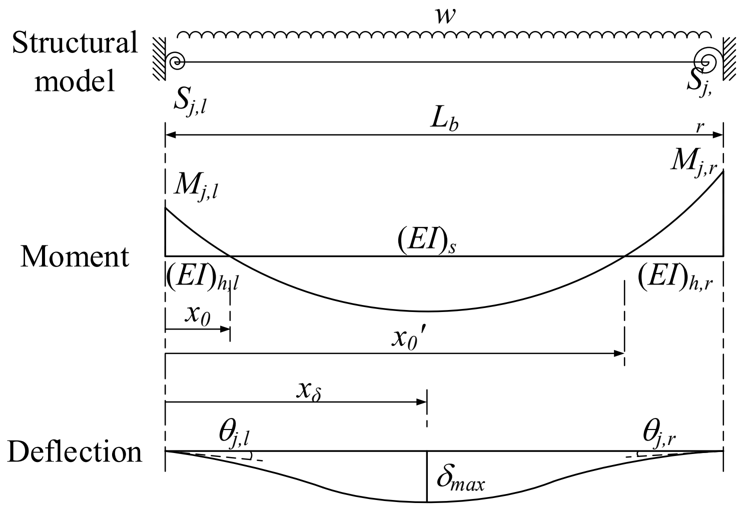
x_0' is the x-coordinate at inflection point



(a) Composite beam supported by composite joints with equal rotational stiffness



(b) Composite beam supported by composite joint at one end



(c) Composite beam supported by composite joints with unequal rotational stiffness

Figure 5.2: Design moment and deflection

(3) Effect of loading patterns

As shown in Figure 5.3, the design moment and deflection of the secondary composite beams with beam-to-beam composite joints may be also affected by loading patterns because the moment between the adjacent secondary beams may be continuous through the primary beam. Therefore, in a precise manner, all the secondary composite beams should be individually designed based on their own critical loading pattern selected from all the possible loading patterns. Unless more accurate methods are provided, the simplified analysis method, in which all the secondary composite beams are classified into external or internal beams and the virtual floor plans including the adjacent composite beams are extracted as shown in Figure 5.4, can be applied to calculate the design moment and deflection of the beams considering the effects of loading patterns. The details procedures are described in **Appendix II**.

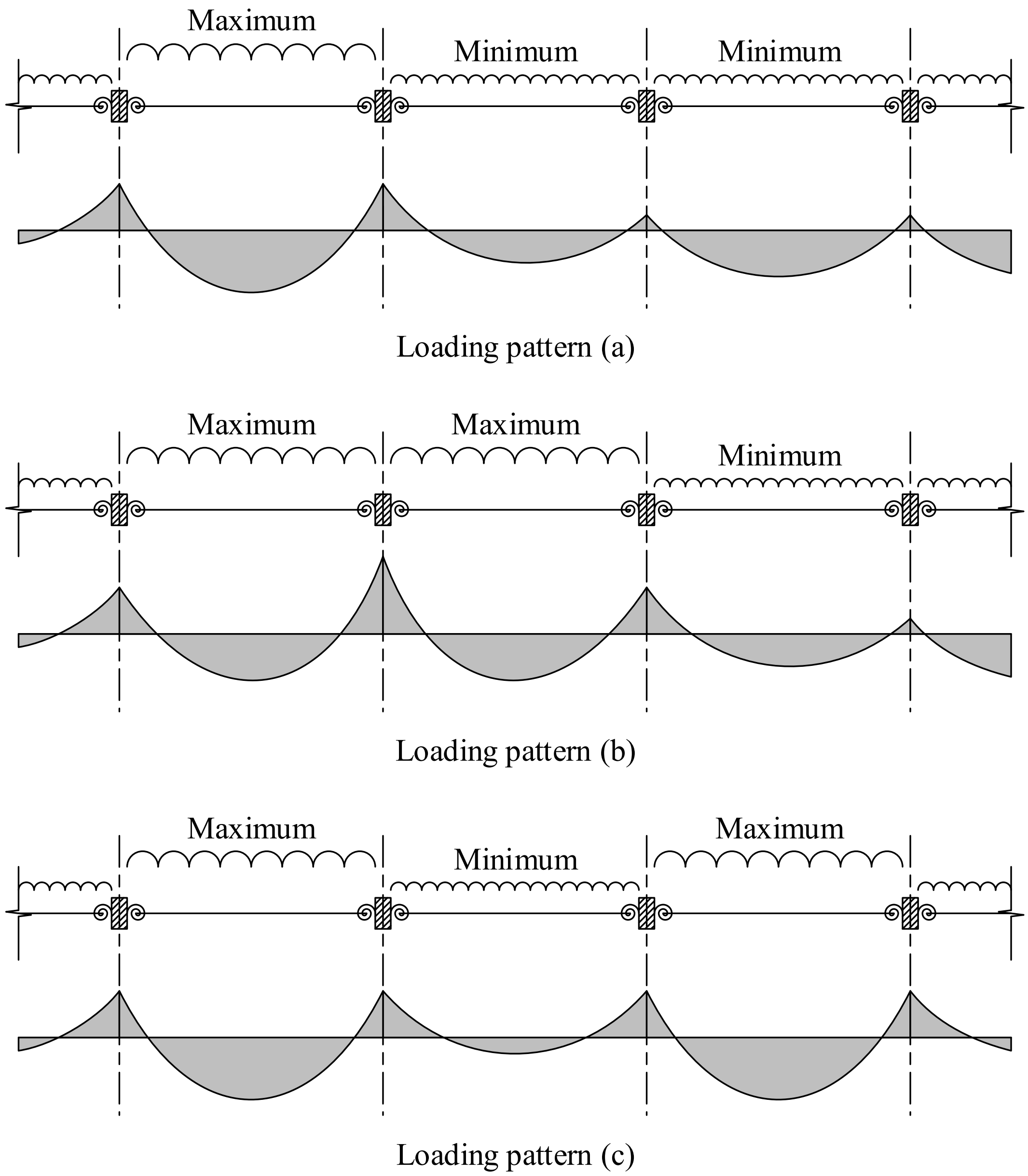
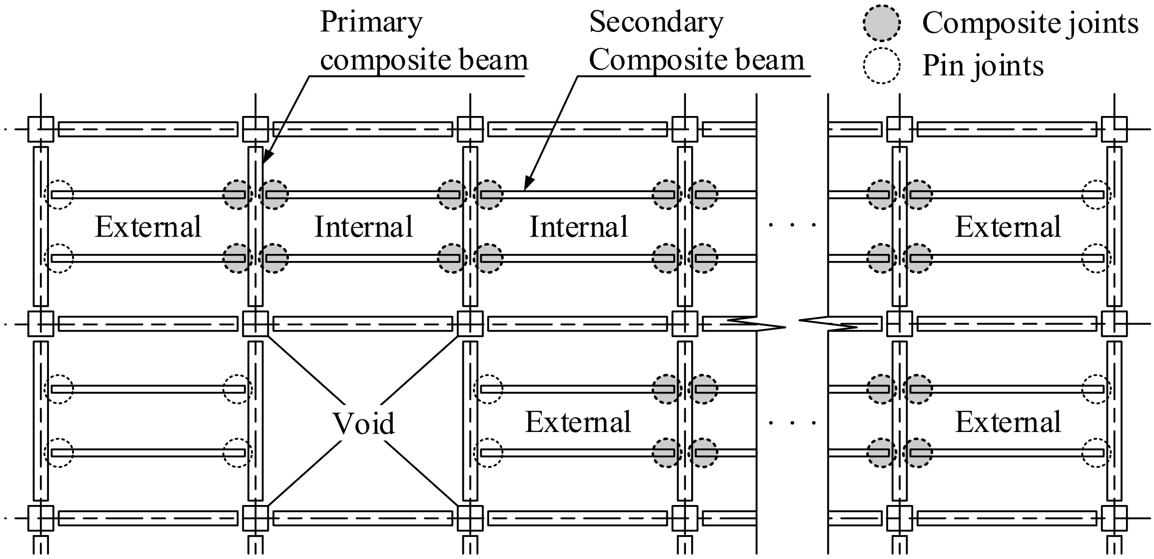
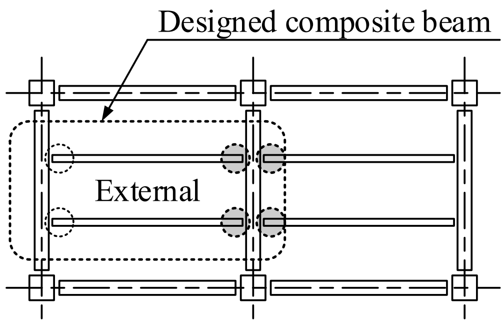


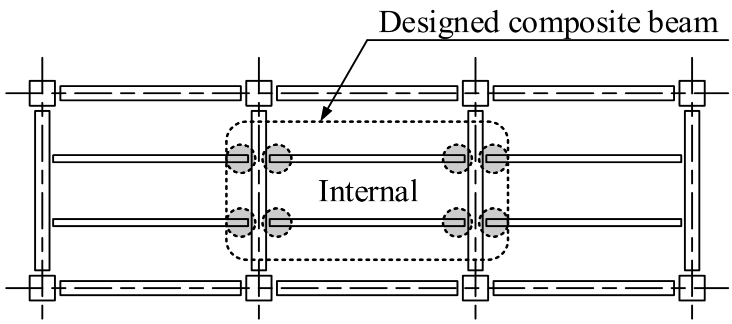
Figure 5.3: Moment distribution in response to loading patterns



(a) Actual floor plan



(b) Virtual floor plan for external beam (continuous 2-span)



(c) Virtual floor plan for internal beam (continuous 3-span)

Figure 5.4: Virtual floor plans in simplified analysis method

Chapter 6 Application to Construction

6.1 General

(1) Basis of construction

The structural performance of composite joints with contact plates may be affected by the detailing of contact plates and reinforcing bars in concrete slab. Therefore, on-site construction should be implemented in accordance with the constructional requirements described in this chapter to ensure that the joints can achieve the expected rotational stiffness and moment capacity.

(2) Construction process

On-site construction including the installation of contact plates can be conducted according to the construction process shown in Figure 6.1. Contact plates can be installed at any time during the construction process, if good contact is achieved at the composite stage. The installed conditions of contact plates should be checked during the final stage of construction.

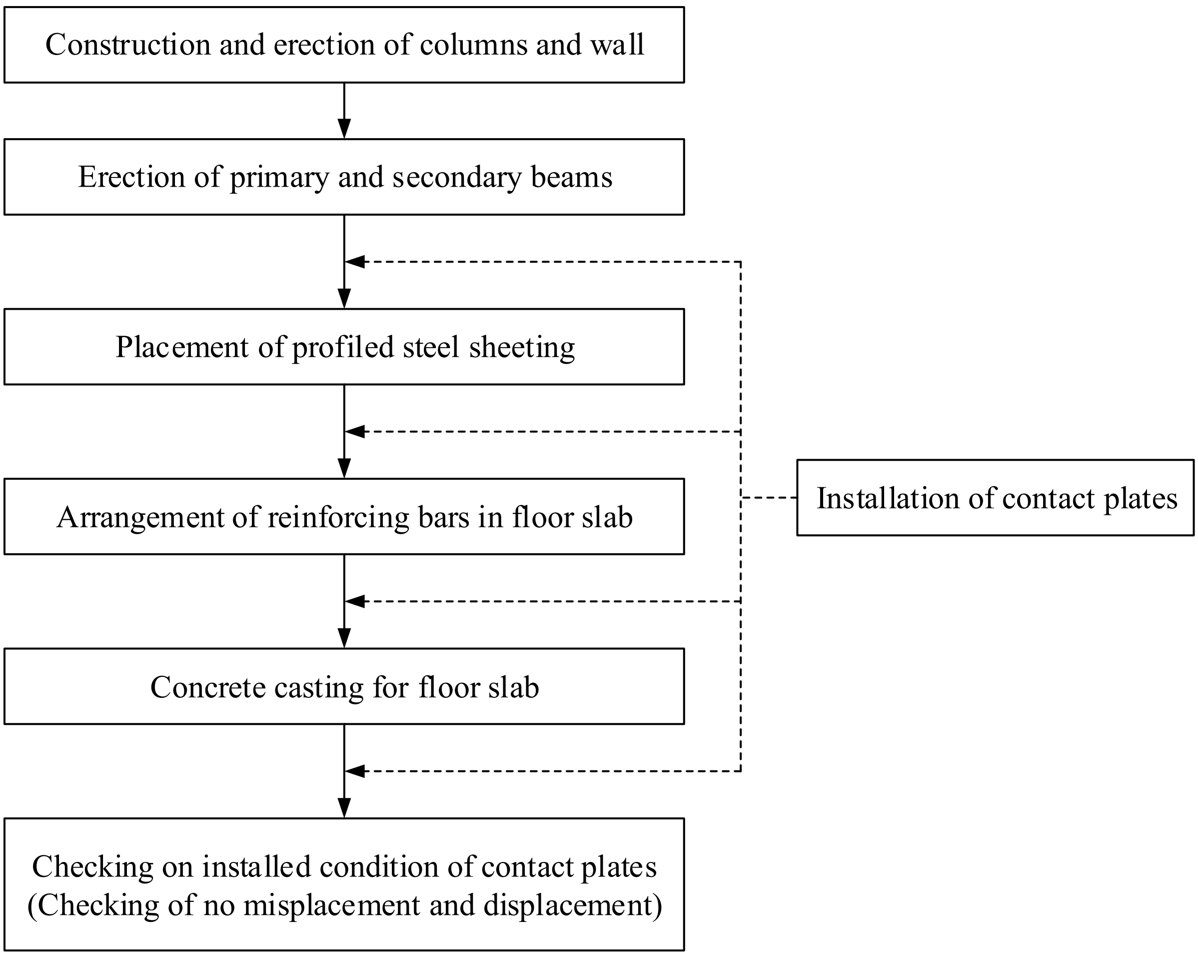


Figure 6.1: Installation of contact plate during construction

6.2 Constructional Requirements

6.2.1 Contact Plates

(1) Detailing and installation method

As a rule, the detailing and installation method of contact plates shall be specified for each project to ensure a reliable contact area is developed to achieve the required rotational stiffness of composite joints.

(2) Quality assurance

Proper measures should be put in place to ensure that contact plates are not susceptible to displacement or falling during the construction stage and throughout the service life of the structure.

(3) Proper contact

One way to achieve proper contact is by directly welding the bottom flange of the steel beam to the stiffener or end plate through the gap as shown in Figure 6.2. Another way to achieve good contact is to insert contact plates with some form of adjustment to ensure that the gap is closed. Examples of contact plate are shown in Figure 6.3.

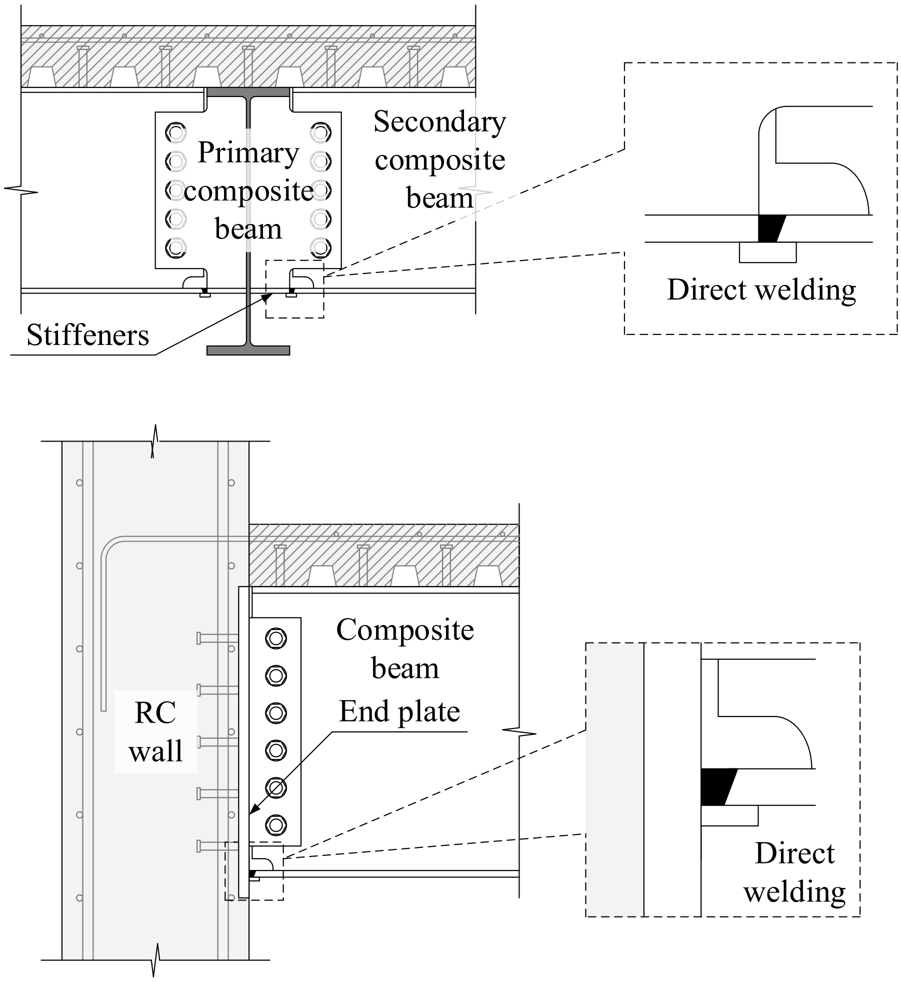
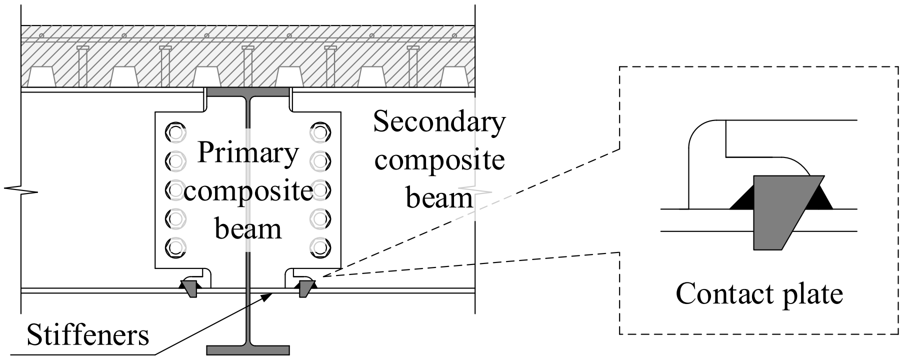
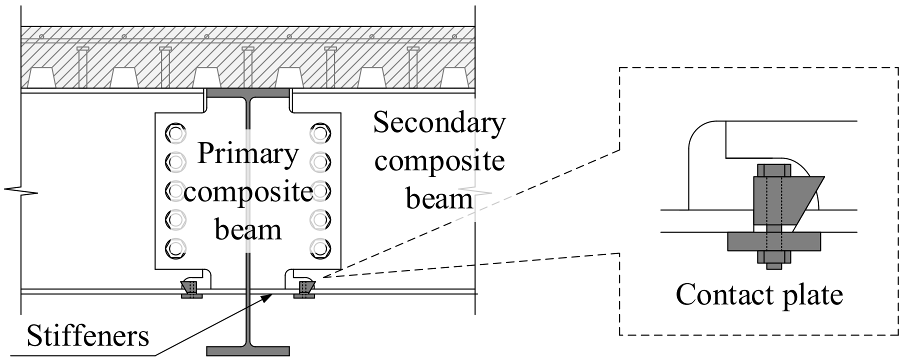


Figure 6.2: Direct welding through gap



(a) Welded contact plate



(b) Bolted contact plate

Figure 6.3: Examples of contact plate

Commentary:

(1) Detailing and installation method

Contact plate detailing and installation method should be pre-qualified to ensure effective contact to transfer the compression force when the beam is loaded. The gap width between the bottom flange of steel beam and the stiffener or the end plate shown in Figure 6.4 may fluctuate due to fabrication and construction tolerances. Therefore, it is necessary to provide the contact plate that can be adjusted to fit the gap caused by these tolerances.

(2) Quality assurance

When contact plates are installed before concrete casting as shown in Figure 6.1, the contact plates may not be in full contact and not perfectly fitted in the construction stage. Even if contact plates are always under compression after concrete casting, it may be displaced due to an unexpected accident. Therefore, some kinds of preventable measures are needed to ensure that contact plates are not susceptible to displacement and falling during the construction stage and throughout the service life of the structure.

(3) Proper contact

In the direct welding method, the yield strength of weld metal shall be higher than or equal to the yield strength of the contact plate assumed in structural design. Also, the welded area shall be larger than or equal to the required contact area to achieve the design moment resistance of the joint. In the contact plate method, the wedge-shaped contact plate is an option to ensure the gap is closed as shown in Figure 6.3. In this case, the upward displacement of the contact plate due to slippage shall be prevented, so that they should be fixed by welding or bolting, or the wedge angle should be designed considering the friction resistance of the contact surface.

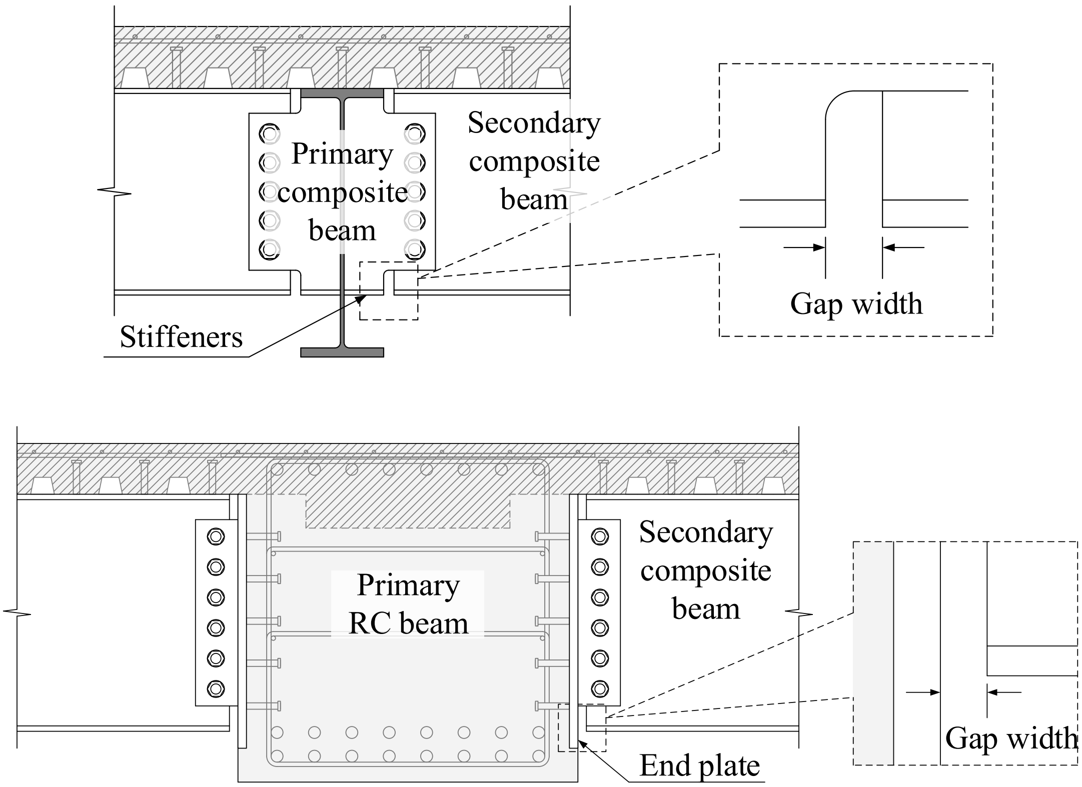


Figure 6.4: Gap width

6.2.2 Reinforcing Bars in Concrete Slab

(1) Concrete cover

The concrete cover for the reinforcing bars in concrete slab should be more than or equal to the nominal cover c_{nom} specified in EN 1992-1-1. This value takes into account the acceptable negative deviation from the required minimum cover c_{min}.

(2) Additional reinforcing bars

As described in subsection 4.2.1, additional reinforcing bars can be arranged in the hogging moment regions apart from anti-crack reinforcing bars. They should be continuous or anchored at composite joints when designed to contribute to the joint performance. The arrangement length in longitudinal direction l_{sl} should be the length of the hogging moment region illustrated in Figure 6.5. The arrangement width in transverse direction b_{sl} to determine the number of additional reinforcing bars can be arbitrarily taken within the beam spacing or the column width, whichever is smaller. However, only the reinforcing bars within the effective width of composite joints $b_{eff,j}$ can contribute to the structural performance of the joints. Therefore, it is desirable to make b_{sl} less than $b_{eff,j}$ for economical design.

(3) Detailing of laps

In the case for composite joints with bent reinforcing bars, the laps of the starter bars and the longitudinal bars may be required considering the construction sequence as shown in Figure 6.6. In this manner, when the longitudinal reinforcing bars to transfer the tension force are lapped in the hogging moment region, the arrangement of lapped bars should comply with EN1992-1-1. Also, the lap length should be larger than the design lap length l_0 calculated by the following equation unless the fire resistance design like membrane action is considered. Here, α_1, α_2, α_3, α_5, α_6, and l_b should be referred to EN1992-1-1.

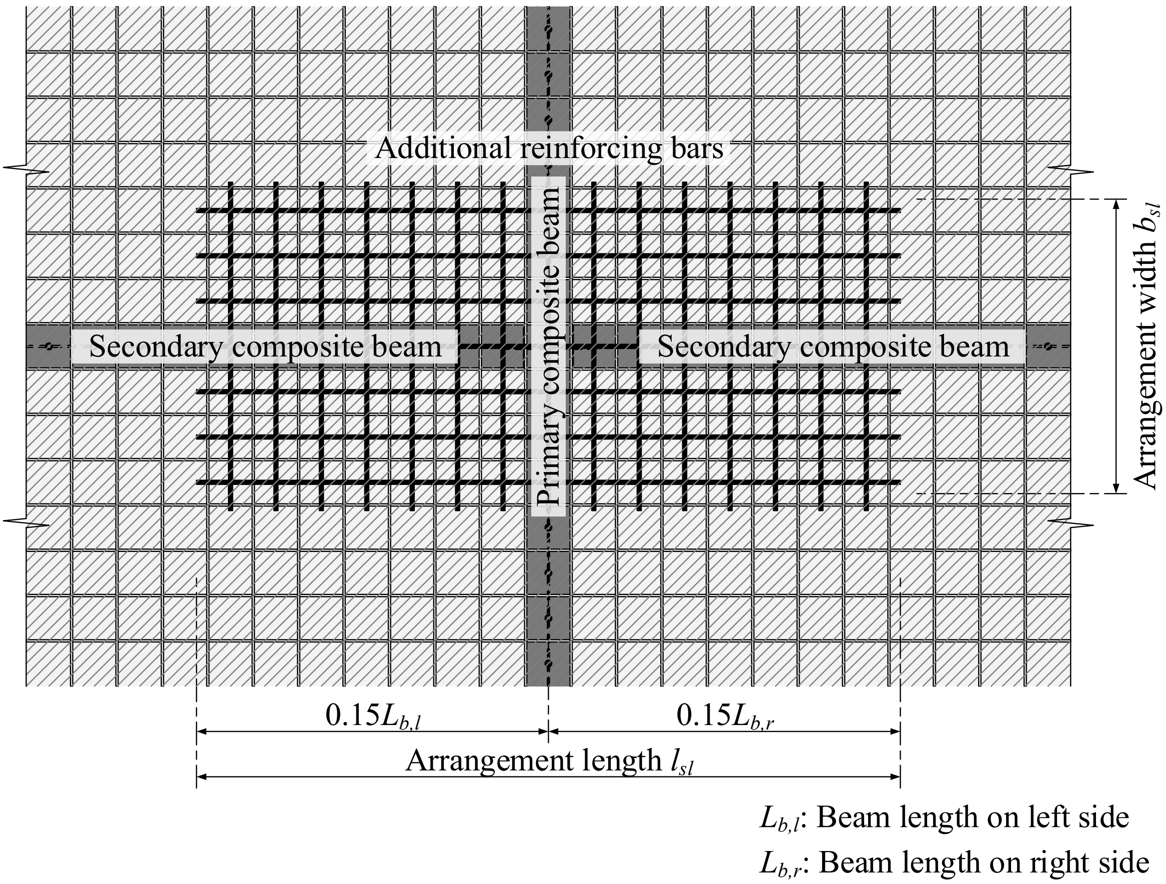
$$l_0 = \min\left(\alpha_1 \alpha_2 \alpha_3 \alpha_5 \alpha_6 l_{b,rqd} \, ; \, l_{0,min}\right) \tag{6.1}$$

$l_{0,min}$ is given by:

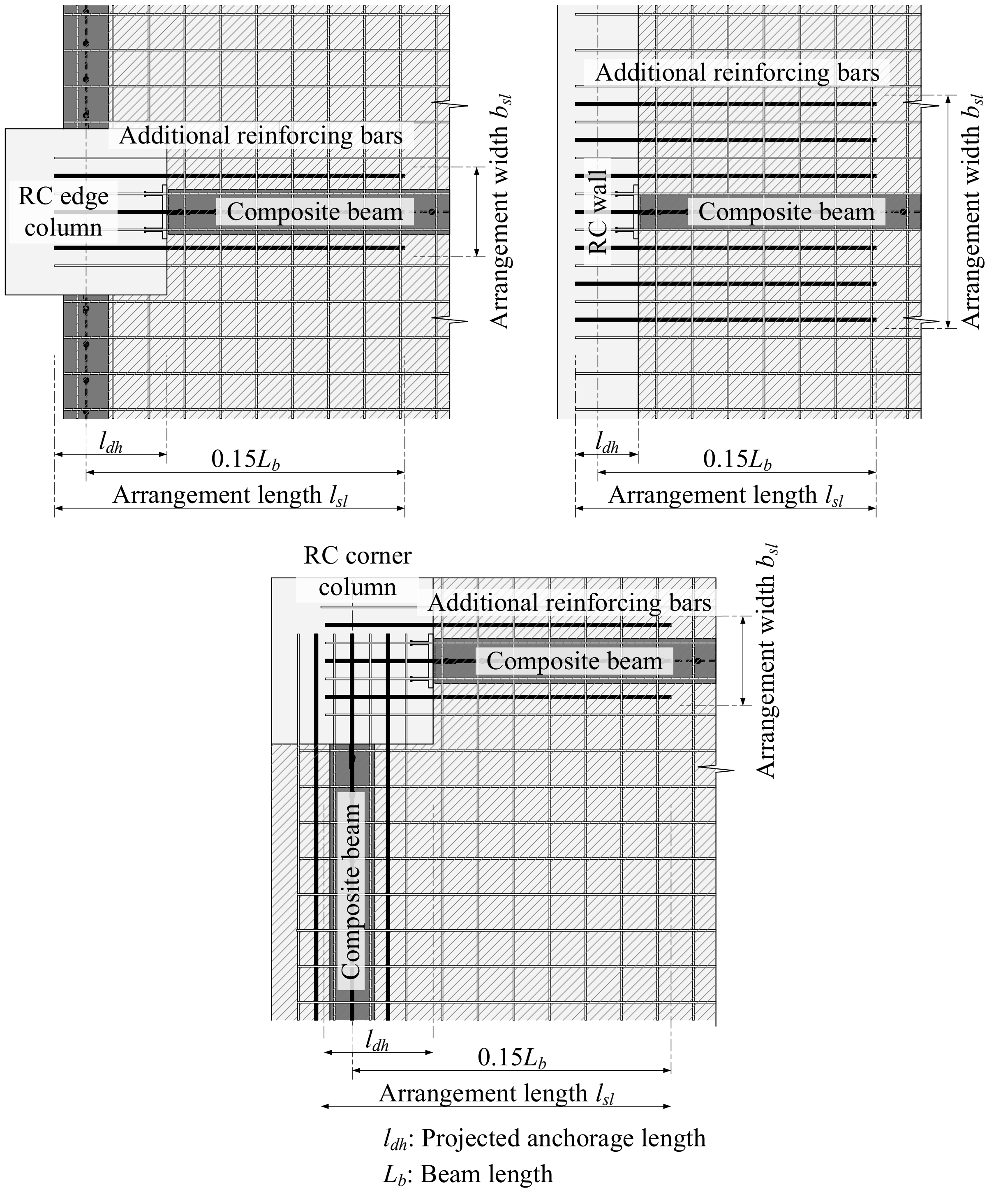
$$l_{0,min} = \max\left(0.3\alpha_6 l_{b,rqd} \, ; \, 15\phi_{sl} \, ; \, 200\right) \tag{6.2}$$

where

α_1	is the coefficient considering shape of reinforcing bars
α_2	is the coefficient considering concrete cover
α_3	is the coefficient considering confinement by transverse reinforcing bars
α_5	is the coefficient considering confinement by transverse pressure
α_6	is the coefficient considering percentage of lapped reinforcing bars
$l_{b,rqd}$	is the basic required anchorage length of longitudinal reinforcing bars
$l_{0,min}$	is the minimum lap length of longitudinal reinforcing bars
ϕ_{sl}	is the diameter of longitudinal reinforcing bars



(a) Composite joints with straight reinforcing bars (continuous reinforcing bras)



(b) Composite joints with bent reinforcing bars (anchored reinforcing bars)

Figure 6.5: Arrangement of additional reinforcing bars

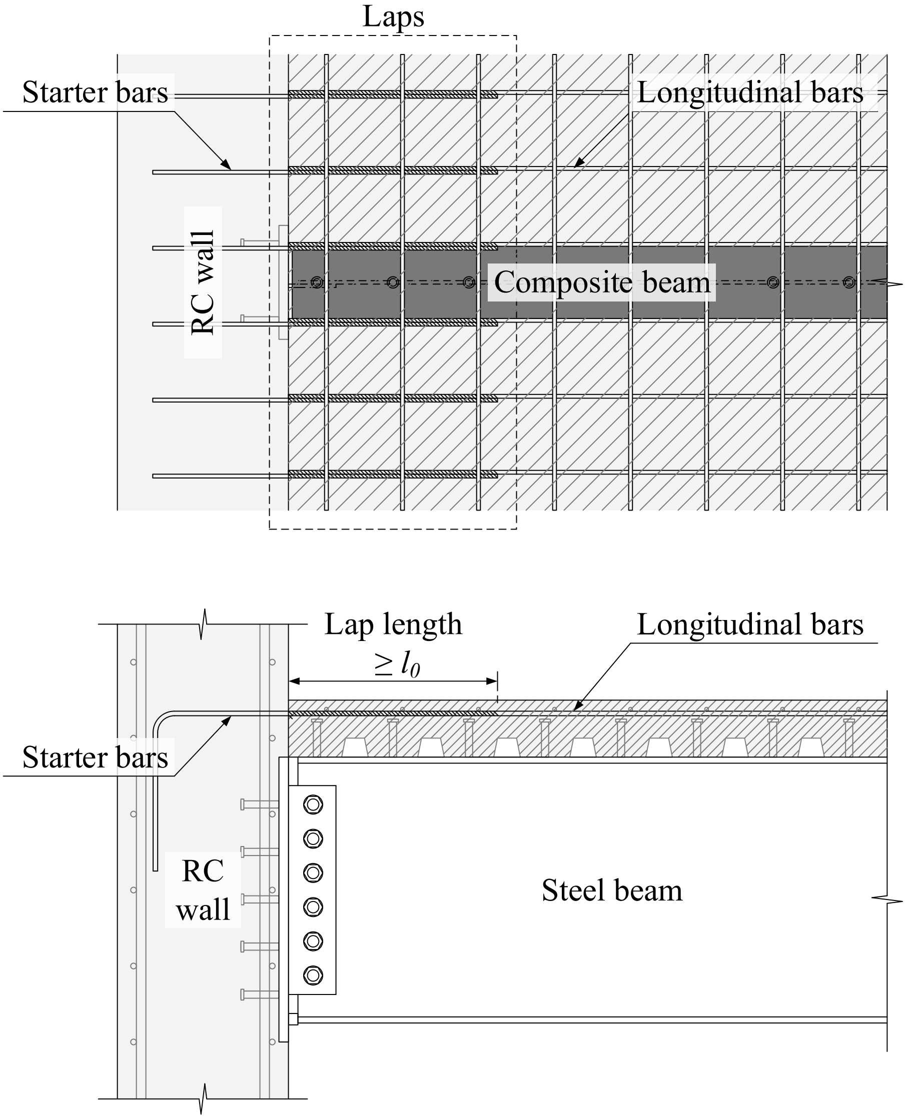


Figure 6.6: Laps of starter bars and longitudinal bars

(4) Mandrel diameter and straight extension length

In the case for composite joints with bent reinforcing bars, the mandrel diameter of bent reinforcing bars ϕ_m should not be less than the minimum values $\phi_{m,min}$ recommended in EN 1992-1-1 to avoid failure of steel bars due to bending operation and failure of the concrete inside the bends of the bars. In addition, the straight extension length l_v (the length past the end of the bend) should be more than $5\phi_{sl}$ so that the bends contribute to the anchorage strength. When the joint details comply with the pre-qualified specifications in Table 4.1 and Table 4.2, the mandrel diameter ϕ_m and the projected extension length l_{dv} should be $12\phi_{sl}$ and 420 [mm] respectively as shown in Figure 6.7, which can conservatively satisfy both the above conditions.

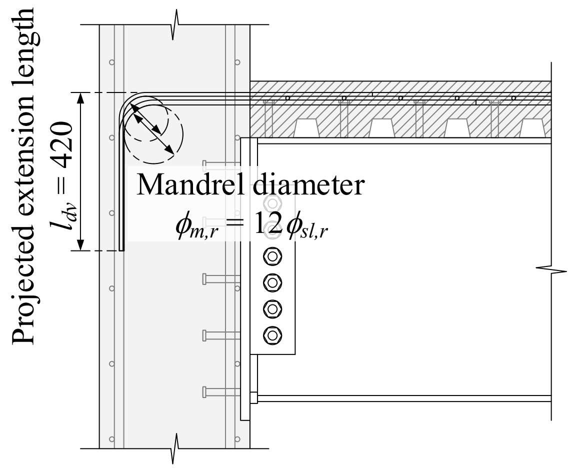


Figure 6.7: Mandrel diameter and projected extension length for pre-qualified specifications

Commentary:

(1) Concrete cover

The concrete cover for the reinforcing bars in concrete slab is defined by the distance between the surface of the reinforcing bars closest to the nearest concrete surface. In EN 1992-1-1, the minimum values of the concrete cover c_{min} is specified in order to ensure the safe transmission of bond forces, the protection of the steel against corrosion, and an adequate fire resistance. This value is determined by the requirements for both bond and environmental conditions. When the additional reinforcing bars mentioned in (2) are arranged on top of anti-crack reinforcing bars, the concrete cover is considered as the distance between the surface of the additional reinforcing bars closest to the nearest concrete surface.

(2) Additional reinforcing bars

The rotational stiffness and the moment resistance of composite joints may not be sufficient with only the anti-crack reinforcing bars depending on the design conditions. Therefore, additional reinforcing bars can be arranged in the hogging moment regions apart from anti-crack reinforcing bars to enhance the rotational stiffness and moment resistance of the composite joints.

(3) Detailing of laps

Generally, the longitudinal reinforcing bars are recommended to be lapped at a region where the existing stress of the reinforcing bars is relatively small, for example the sagging moment region in which floor slab is subjected to compression force. However, when they are lapped in the hogging moment region, the arrangement of lapped bars should comply with EN1992-1-1 and the lap length should be larger than the design lap length l_0. If the diameters of anti-crack and additional reinforcing bars are different, the design lap lengths may also be different accordingly.

(4) Mandrel diameter and straight extension length

The minimum mandrel diameter for bars and wire $\phi_{m,min}$ shown in Table 6.1 is recommended in EN1992-1-1. Japanese architectural standard specification for reinforced concrete work JASS 5[21] also recommends the values in Table 6.2 as the minimum mandrel diameter according to the bar strength, in which the strength class of SD490 is almost same as that of B500. The pre-qualified specifications where the bar diameter ϕ_{sl} is limited to 16 [mm] or less requires $12\phi_{sl}$ as the mandrel diameter ϕ_m, which is conservatively larger than the above minimum values.

EN1992-1-1 also specifies that the straight extension length l_v of bent bars should be more than $5\phi_{sl}$ to expect contribution of the length past the end of the bend. According to design guidelines for earthquake resistant reinforced concrete building based on inelastic displacement concept[11], it has been experimentally verified that the anchorage strength is not significantly increased even if the straight extension length for 90-degree bent bars is longer than $10\phi_{sl}$. However, it is desirable to take the above length longer than $10\phi_{sl}$ in terms of the deterioration of anchorage performance assuming the repeated loads. Therefore, in the pre-qualified specifications, the projected extension length l_{dv} of 420 [mm] is on the safe side.

Table 6.1: Minimum mandrel diameter for bars and wire

Bar diameter	Minimum mandrel diameter for bends, hooks and loops
$\phi_{sl} \leq 16$ [mm]	$4\phi_{sl}$
$\phi_{sl} > 16$ [mm]	$7\phi_{sl}$

Table 6.2: Minimum mandrel diameter for bent bars

Bending angle	Strength class	Bar diameter	Minimum mandrel diameter
180° 135° 90°	SD295 SD345	$\phi_{sl} \leq 16$ [mm]	$3\phi_{sl}$
		19 [mm] $\leq \phi_{sl} \leq$ 41 [mm]	$4\phi_{sl}$
	SD390	$\phi_{sl} \leq 41$ [mm]	$5\phi_{sl}$
90°	SD490	$\phi_{sl} \leq 25$ [mm]	
		29 [mm] $\leq \phi_{sl} \leq$ 41 [mm]	$6\phi_{sl}$

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Appendix I Anchorage Strength and Panel Shear Resistance of Reinforced Concrete

This appendix presents the calculation procedures for the anchorage strength and panel shear resistance of reinforced concrete members using 90° hooked bars. These procedures refer to the design guidelines of the Architectural Institute of Japan[11].

The evaluation formulas proposed below are meant for beam-to-column joints in reinforced concrete structures and their applicability to composite joints in which bending moment can be transferred by slab reinforcement and contact parts has not been fully verified. For the joint details complying with the pre-qualified specifications, the recent experiments showed that the anchorage strength and panel shear resistance of supporting members can be roughly estimated by these formulas. However, the number of experiments is still limited and their accuracy under arbitrary design conditions needs to be further investigated. Therefore, when using the joint details other than the pre-qualified specifications, it is preferable to get experimental or numerical evidence in addition to the design checks by these formulas.

(1) Anchorage strength

a) Tensile strength of reinforcing bars due to raking-out failure

The ultimate tensile strength of reinforcing bars due to raking-out failure T_{ro} can be determined by the following equation.

$$T_{ro} = T_c + T_w \tag{AI.1}$$

T_c and T_w are given by:

$$T_c = 0.313 b_e d_e \sqrt{\frac{f_{ck}}{\gamma_c}} \frac{\sqrt{{l_{dh}}^2 + {z_{csl\text{-}cc}}^2}}{z_{csl\text{-}cc}} \tag{AI.2}$$

$$T_w = 0.7 A_w \left(\frac{f_{wk}}{\gamma_w}\right) \tag{AI.3}$$

where

T_c	is the contribution of concrete for T_{ro}
T_w	is the contribution of spreader bars for T_{ro}
b_e	is the effective width of raking-out region
d_e	is the effective depth of raking-out region
f_{ck}	is the characteristic cylinder strength of concrete
γ_c	is the partial factor of concrete
l_{dh}	is the projected anchorage length of bent reinforcing bars
$z_{csl\text{-}cc}$	is the vertical distance between bent reinforcing bars and centre of contact part
A_w	is the total cross-sectional area of crossties or hoops within raking-out region

f_{wk} is the characteristic yield strength of crossties or hoops
γ_w is the partial factor of crossties or hoops

b) Tensile strength of reinforcing bars due to local compression failure

The ultimate tensile strength of reinforcing bars due to local compression failure T_{lc} can be determined by the following equation.

$$T_{lc} = 210 k_c k_j k_d k_s \left(\frac{f_{ck}}{\gamma_c}\right)^{0.4} A_{sl,j} \quad \text{(AI.4)}$$

k_c, k_j, k_d, and k_s are given by:

$$k_c = \min\left(0.4+\frac{0.1C_0}{\phi_{sl}}\ ;\ 1.0\right) \quad \text{(AI.5)}$$

$$k_j = \min\left(0.6+\frac{0.4l_{dh}}{z_{csl\text{-}cc}}\ ;\ 1.0\right) \quad \text{(AI.6)}$$

$$k_d = \min\left(0.5+\frac{l_{dh}}{30\phi_{sl}}\ ;\ 1.0\right) \quad \text{(AI.7)}$$

$$k_s = \min\left(0.7+\frac{0.5{\phi_w}^2}{{\phi_{sl}}^2}\ ;\ 1.0\right) \quad \text{(AI.8)}$$

where

$A_{sl,j}$ is the cross-sectional area of bent reinforcing bars effective for joint design
k_c is the coefficient for effect of horizontal cover
k_j is the coefficient for effect of bent position
k_d is the coefficient for effect of projected anchorage length
k_s is the coefficient for effect of crossties or hoops
C_0 is the horizontal covering depth of bent reinforcing bars
ϕ_{sl} is the diameter of bent reinforcing bars
ϕ_w is the diameter of crossties or hoops

(2) Panel shear resistance

The shear resistance of joint panel V_{jp} can be determined by the following equation.

$$V_{jp} = \kappa \phi F_{jp} b_{jp} l_{dh} \quad \text{(AI.9)}$$

F_{jp} and b_{jp} are given by:

$$F_{jp} = 0.8\left(\frac{f_{ck}}{\gamma_c}\right)^{0.7} \quad \text{(AI.10)}$$

$$b_{jp} = B_{ep}+\frac{l_{dh}}{2} \quad \text{(AI.11)}$$

where

κ is the coefficient for effect of joint type
ϕ is the correction coefficient due to presence of transverse beam

F_{jp}	is the nominal value of shear resistance of joint panel
b_{jp}	is the effective width of joint panel
B_{ep}	is the width of end plate

Appendix II Simplified Analysis Method

This appendix presents the simplified analysis method for the design moment and deflection of secondary composite beams with beam-to-beam composite joints considering the effects of loading patterns. Step-by-step procedures utilizing the equations in Section 5.3 and the moment distribution method are described.

Typical composite floor plan shown in Figure AII.1 is considered. It should be noted that the single-sided beam-to-beam composite joint with primary composite beam located at the outer periphery or around the voids are designed as pinned joints because neither the tension nor compression force is transferred at the joint unless special measures are taken to ensure proper anchorage of the reinforcing bars and to prevent the out-of-plane deformation of the primary beam. Here, the step-by-step procedures of the simplified analysis method are as follows.

(1) Classification of secondary composite beams into external or internal beams
(2) Extraction of virtual floor plans
(3) Setting of end-restraint conditions and critical loading patterns
(4) Analysis of design moment and deflection of secondary composite beams

In the practical design, the above series of the procedures should be performed for all secondary composite beams individually. However, except for singular design conditions, beam span, beam spacing, steel beam, arrangement of reinforcing bars, and applied loads are often uniform within the same floor plan. Therefore, according to this simplified analysis method based on the virtual floor plans discussed later, the number of secondary composite beams to be checked in structural design can be reduced compared with the procedures based on the actual floor plan. Also, the analysis of the design moment and deflection of the beams can be significantly easier in the simplified analysis method. The details of each procedure are discussed below.

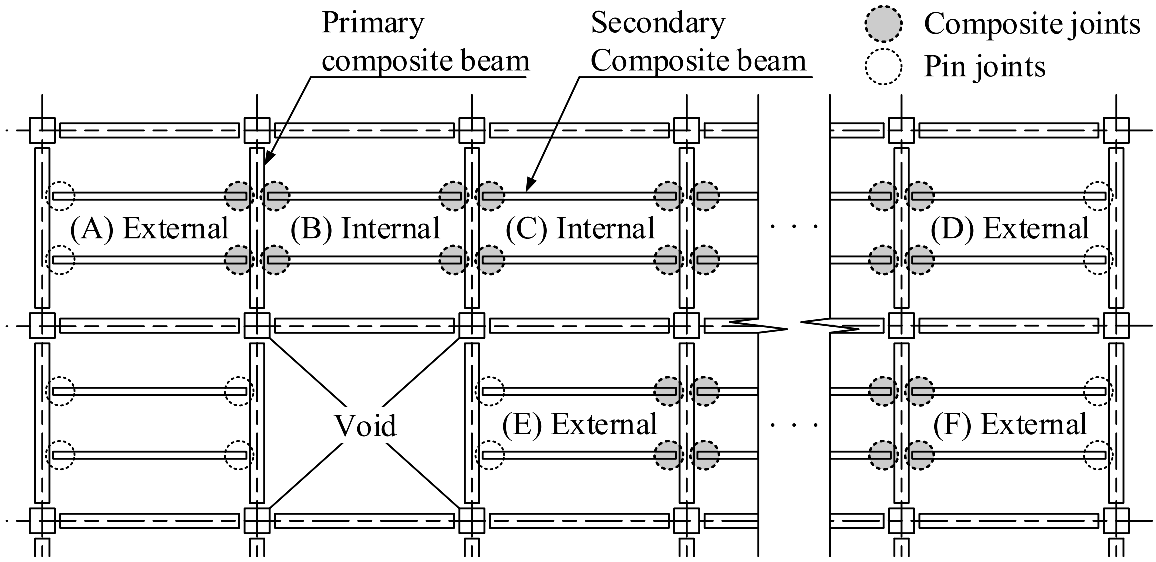


Figure AII.1: Typical composite floor plan

(1) Classification of secondary composite beams into external or internal beams

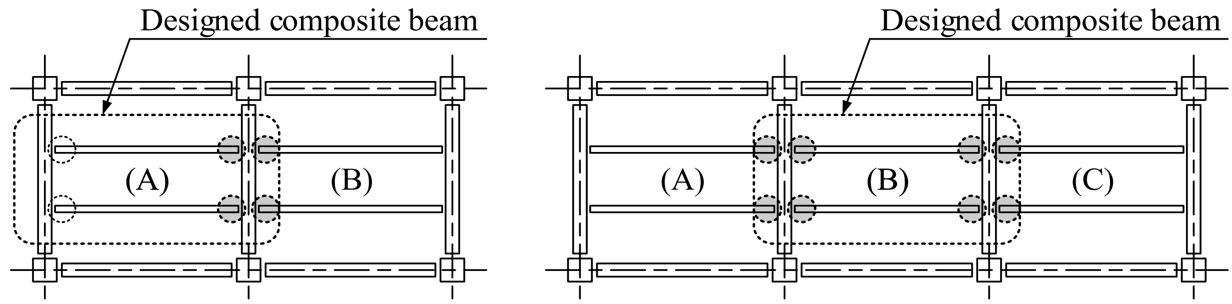
Firstly, all secondary composite beams are classified into two the types of beams, external beams or internal beams. Here, the external beams are the secondary composite beams with beam-to-beam composite joint at one side, and the internal beams are the secondary composite beams with beam-to-beam composite joints at both sides. Accordingly, referring to the composite beams in the typical composite floor plan shown in Figure AII.1, the composite beams, (A), (D), (E), and (F) are classified into external beams, and the composite beams, (B) and (C) are classified into internal beams.

(2) Extraction of virtual floor plans

Next, the virtual floor plans are extracted for each designed composite beam. For the external beam, the virtual floor plan (continuous 2-span) including its adjacent beam is extracted. Similarly for the internal beam, the virtual floor plan (continuous 3-span) including its adjacent beams on both sides is extracted. For example, when the composite beam (A) is designed, the virtual floor plan including the composite beam (A) and (B) is extracted as shown in Figure AII.2 (a). When the composite beam (B) is designed, the virtual floor plan including the composite beam (A), (B), and (C) is extracted as shown in Figure AII.2 (b).

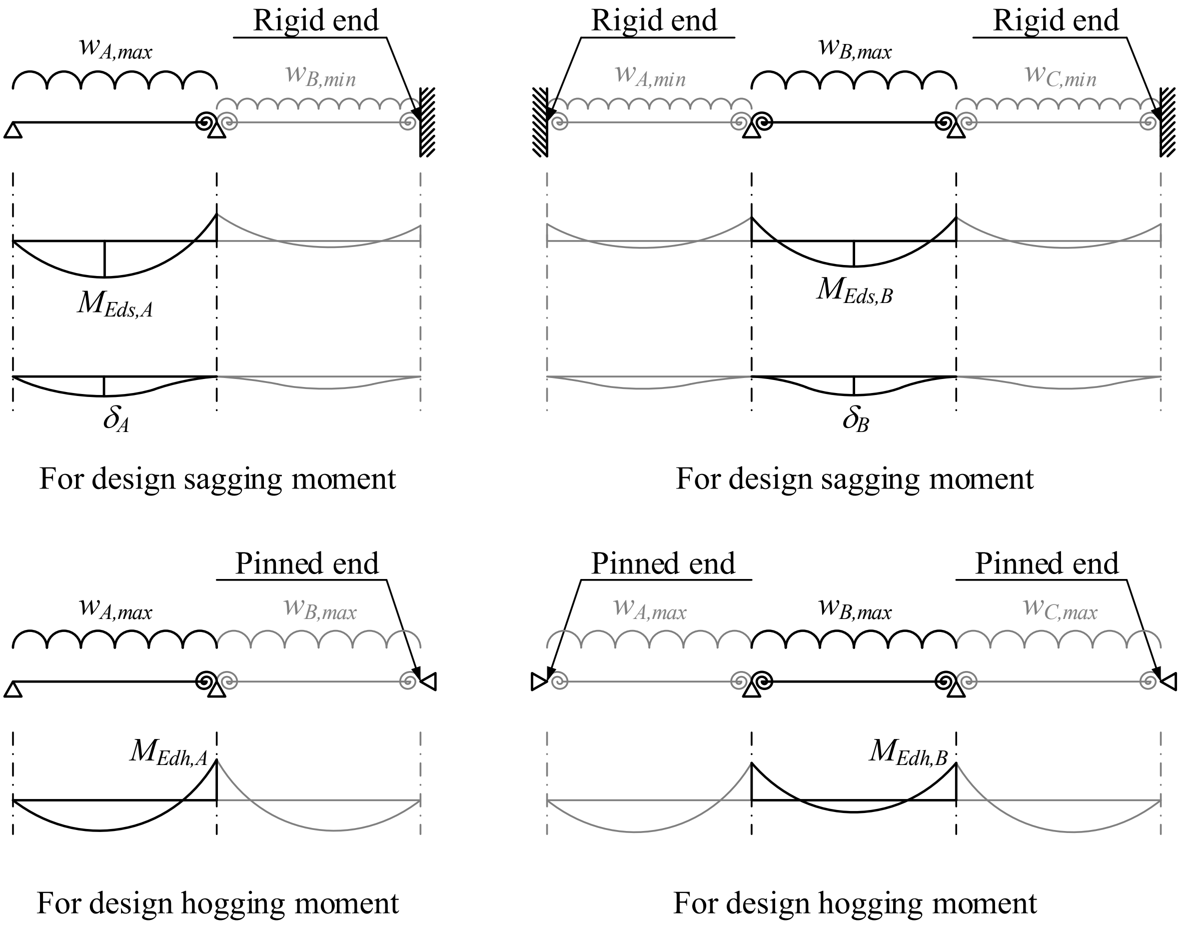
(3) Setting of end-restraint conditions and critical loading patterns

Subsequently, the end-restraint conditions and the critical loading patterns that can contribute to the maximum moment and deflection of the designed composite beams are set appropriately. It should be noted that they should be set for each of the following two cases: one is to generate a maximum sagging moment and the other is to generate a maximum hogging moment in the continuous beam. This is because the moment resistance of composite beams in the sagging moment region and in the hogging moment region may be different because of cracking of reinforced concrete slab. The end-restraint conditions and the critical loading patterns for the design moment and deflection are shown in Figure AII.3. With respect to the virtual floor plan for external beams, the sagging moment may be maximized when the rightmost end is assumed to be a rigid end and the distributed loads on the composite beam (A) and (B) are maximized and minimized respectively. Also, the hogging moment may be maximized when the rightmost end is assumed to be a pinned end and the distributed loads on both the composite beam (A) and (B) are maximized. On the other hand, with respect to the virtual floor plan for internal beams, the sagging moment may be maximized when both ends are assumed to be rigid ends and the distributed loads on the composite beam (B) and the other composite beams are maximized and minimized respectively. Also, the hogging moment may be maximized when both ends are assumed to be pinned ends and the distributed loads on all the composite beams are maximized. Incidentally, in both the virtual floor plans, the end-restraint conditions and the critical loading patterns that can maximize the deflection of the composite beams are the same as those that can maximize the sagging moment.



(a) Virtual floor plan for external beam (b) Virtual floor plan for internal beam

Figure AII.2: Virtual floor plans in simplified analysis method



(a) Virtual floor plan for external beam (b) Virtual floor plan for internal beam

Figure AII.3: End-restraint conditions and critical loading patterns

(4) Analysis of design moment and deflection of secondary composite beams

Finally, the design moment and deflection of secondary composite beams are analyzed in accordance with the above end-restraint conditions and the critical loading patterns. Note that they cannot be obtained only with the force equilibrium since the composite beams are statically indeterminate beams. However, the maximum design moment and deflection shown in Figure AII.3 can be calculated by utilizing the equations in Section 5.3 and the moment distribution method. Here, as an example, the calculation processes for the design sagging moment $M_{Eds,A}$ and deflection δ_A of the virtual floor plan for external beam shown in Figure AII.3 (a) are introduced.

Based on the principle of the moment distribution method, the actual moment distribution can be obtained by overlapping the moment distribution assuming the internal support as a rigid end and the moment distribution considering the release of the rigid end as shown in Figure AII.4. In this figure, $M_{h(wA,max)}$ and $M_{h(wB,min)}$ are the end moment of the composite beam (A) and (B) due to the uniformly distributed load $w_{A,max}$ and $w_{B,min}$, and these end moments can be obtained from the equations in Section 5.3. Also, $\mu_A M_{h(wA,max)}$ and $\mu_A M_{h(wB,min)}$ are the end moment of the composite beam (A) due to the release of $M_{h(wA,max)}$ and $M_{h(wB,min)}$. Here, the distribution factor μ_A is defined by Eq.(AII.1):

$$\mu_A = \frac{S_{j,A}}{S_{j,A}+S_{j,B}} \tag{AII.1}$$

where $S_{j,A}$ and $S_{j,B}$ are the rotational stiffness of the beam-to-beam composite joints applied to the composite beam (A) and (B). Therefore, the actual end moment of the composite beam (A) $M_{h,A}$ can be obtained by Eq.(AII.2).

$$M_{h,A} = M_{h,(wA,max)} - \mu_A M_{h,(wA,max)} + \mu_A M_{h,(wB,min)} \tag{AII.2}$$

On the other hand, the moment of the composite beam (A) along x-axis $M(x)$ can be expressed by Eq.(AII.3) considering the force equilibrium.

$$M(x) = -\frac{w_{A,max}}{2}x^2 + \left(\frac{w_{A,max}L_{b,A}}{2} - \frac{M_{h,A}}{L_{b,A}}\right)x \tag{AII.3}$$

Therefore, the design sagging moment $M_{Eds,A}$ which is the local maximum value of Eq.(AII.3) can be expressed by Eq.(AII.4) and calculated by substituting Eq.(AII.2) into Eq.(AII.4).

$$M_{Eds,A} = \frac{1}{2w_{A,max}}\left(\frac{w_{A,max}L_{b,A}}{2} - \frac{M_{h,A}}{L_{b,A}}\right) \tag{AII.4}$$

In addition, the deflection δ_A can be also obtained from the equations in Section 5.3 by substituting $M_{h,A}$. Note that the moment resistance of the beam-to-beam composite joints applied to the composite beam (A) $M_{j,Rd,A}$ is assumed to be more than $M_{h,A}$ in the above calculation processes, but if $M_{j,Rd,A}$ is less than $M_{h,A}$, $M_{j,Rd,A}$ should be taken as $M_{h,A}$. With the same calculation processes as above, the design moment and deflection in accordance with the other end-restraint conditions and the critical loading patterns shown in Figure AII.3 can be also calculated by utilizing the equations in Section 5.3 and the moment distribution method.

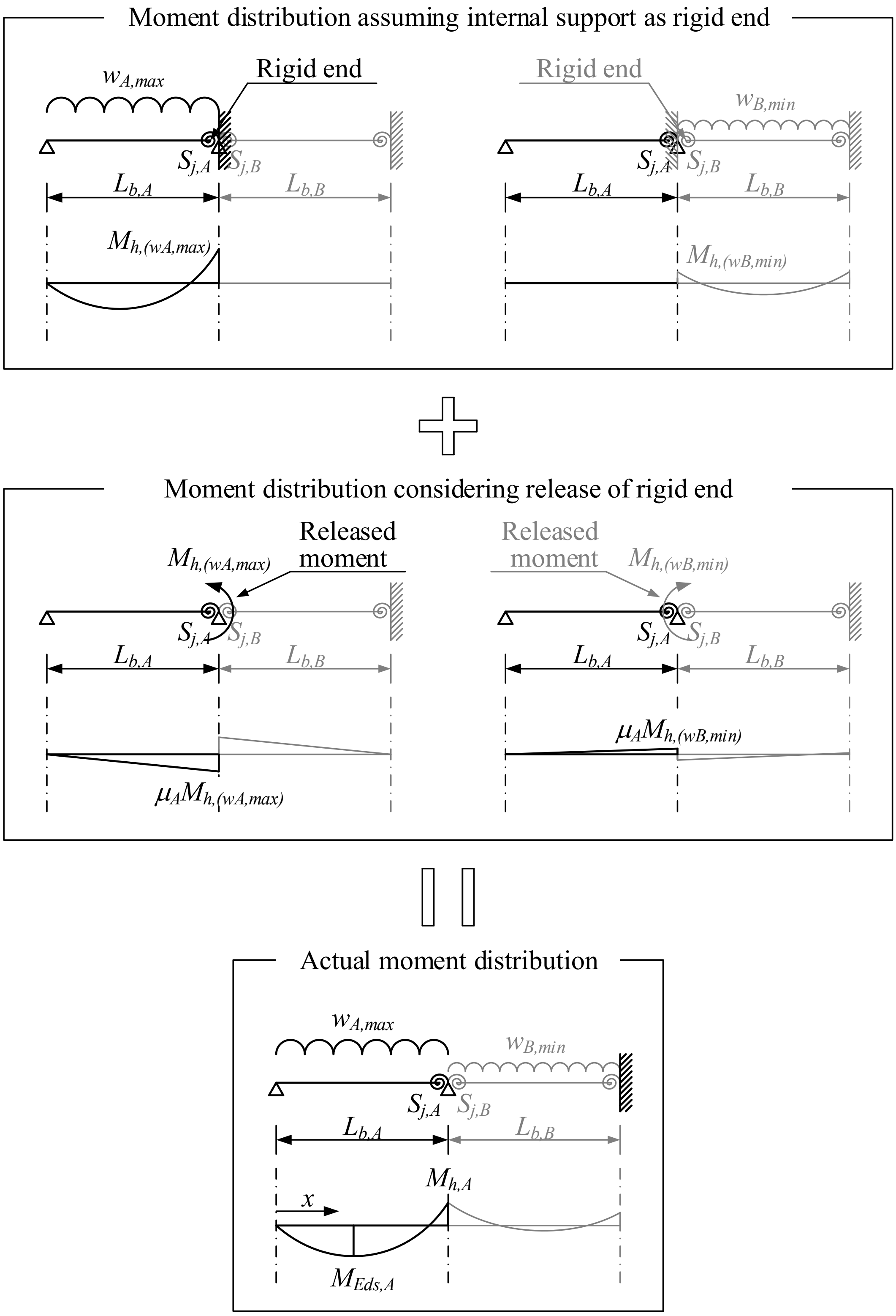


Figure AII.4: Principle of moment distribution method

Appendix III Design Example 1

This appendix presents the design example for a secondary composite beam with beam-to-beam composite joints at both sides in accordance with Eurocode approach.

Figure AIII.1 shows the composite floor plan in this design example. All the beams are composite beams and the designed beam is an internal secondary composite beam supported by contact-type beam-to-beam composite joints at both sides. The joint details at both sides are the same including the cross-sections of primary composite beams. The designed beam is subjected to only the gravity load, and its design moment and deflection are analysed by the elastic-plastic global analysis in which the joints are allowed to behave as plastic hinges in inelastic region. Therefore, the cross-section of the composite beam should be at least in Class 2. In addition, the effects of loading patterns are considered in accordance with the simplified analysis method described in **Appendix II**.

In this design example, structural resistance checks at ultimate limit state and serviceability checks are carried out. Based on Eurocode, serviceability limit state in buildings should consider the criteria related to the floor stiffness. These stiffness criteria may be expressed in terms of the limits for the vertical deflections and vibrations, and should be specified in each project. However, in this design, the limit of the deflection due to variable actions is $L_b/360$, the limit of the deflection due to permanent and variable actions is $L_b/250$, and the minimum natural frequency is taken as 4 [Hz]. Besides, the limit of the crack width recommended in Eurocode 2, is taken as 0.3 [mm], for the other serviceability criteria.

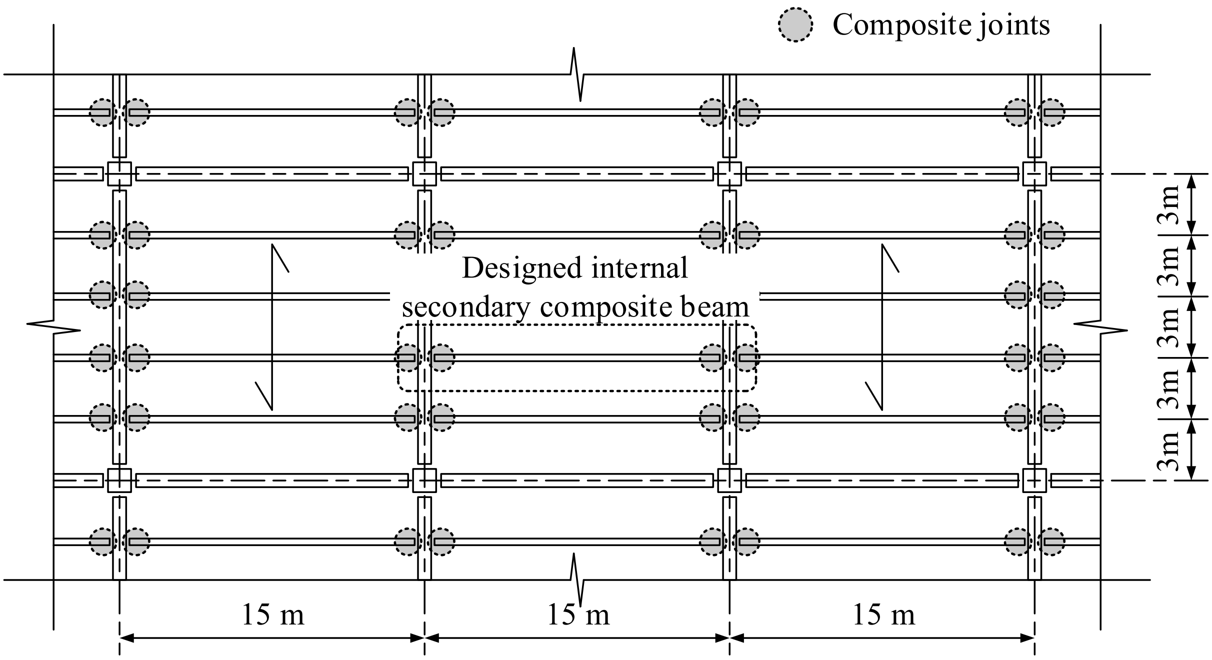


Figure AIII.1: Composite floor plan

Incidentally, as the composite beams are assumed to be un-propped in construction stage, the verifications should be carried out not only in composite stage but also in construction stage. The direction of the ribs of the profiled steel sheeting is perpendicular to the secondary beams. Noted that the steel beams are uniform without any haunches within each beam span and arranged at equal spacing. In addition, the specifications of the floor slab are uniform within the same floor plan.

Design Conditions

[Span and spacing]

Beam span: L_b = 15.0 [m]
Beam spacing: B_b = 3.0 [m]

[Steel beam]

Cross-section: H700x200x9x16 (JIS cross-section)
Mass per metre: g_a = 99.6 [kg/m]
Depth: D_a = 700 [mm]
Width: B_a = 200 [mm]
Web thickness: t_w = 9 [mm]
Flange thickness: t_f = 16 [mm]
Root radius: r = 18 [mm]
Cross-sectional area: A_a = 126.9 [cm^2]
Second moment of area about major axis (y-y axis): I_{ay} = 100255 [cm^4]
Second moment of area about minor axis (z-z axis): I_{az} = 2140 [cm^4]
Plastic section modulus: $W_{pl,a}$ = 3285 [cm^3]
Warping constant: $I_{w,a}$ = 2.50 [dm^6]
Torsion constant: $I_{T,a}$ = 81.8 [dm^4]
Steel grade: S355
Nominal value of yield strength: f_{ay} = 355 [N/mm^2] (for t_f = 16 [mm])
Ultimate tensile strength: f_{au} = 470 [N/mm^2] (for t_f = 16 [mm])
Modulus of elasticity: E_a = 210000 [N/mm^2]
Partial factor of resistance of members and cross-sections: γ_a = 1.00
Partial factor of resistance of plates in bearing: $\gamma_{a,2}$ = 1.25
Design yield strength: $f_{ayd} = f_{ay}/\gamma_a$ = 355 [N/mm^2]

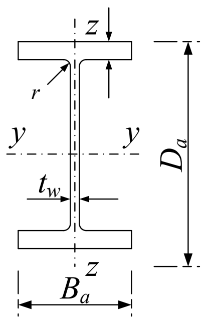


Figure AIII.2: Cross-section of steel beam

[Profiled steel sheeting]

Steel sheeting type: BONDEK 1.0
Mass per metre: g_{ps} = 13.79 [kg/m]
Overall depth: D_{ps} = 51 [mm]
Pitch of ribs: p_{ps} = 200 [mm]
Minimum width for re-entrant: $b_{0,min}$ = 168 [mm]
Maximum width for re-entrant: $b_{0,max}$ = 187 [mm]
Thickness: t_{ps} = 1.00 [mm]
Characteristic yield strength: f_{psk} = 550 [N/mm^2]
Partial factor: γ_{ps} = 1.00
Design yield strength: $f_{psd} = f_{psk}/\gamma_{ps}$ = 550 [N/mm^2]

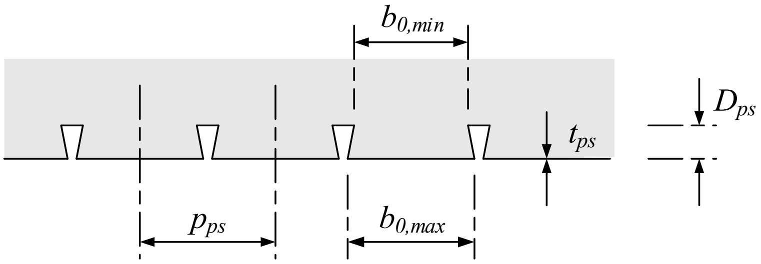


Figure AIII.3: Cross-section of profiled steel sheeting

[Concrete slab]

Overall depth: D_{cs} = 150 [mm]
Thickness above profiled steel sheeting: $h_{cs} = D_{cs}-D_{ps}$ = 99 [mm]
Strength class of concrete: C25/30
Characteristic cylinder strength: $f_{ck,cs}$ = 25.0 [N/mm^2]
Mean value of tensile strength: f_{ctm} = 2.6 [N/mm^2]
Secant modulus of elasticity: $E_{cm,cs}$ = 31000 [N/mm^2]
Partial factor: $\gamma_{c,cs}$ = 1.50
Design strength: $f_{cd,cs} = f_{ck,cs}/\gamma_{c,cs}$ = 16.7 [N/mm^2]
Dry density: ρ_c = 2400 [kg/m^3]

[Reinforcing bar]

Diameter of anti-crack longitudinal rebars (row 1): $\phi_{sl,1}$ = 10 [mm]
Diameter of anti-crack transverse rebars (row 1): $\phi_{st,1}$ = 10 [mm]
Diameter of additional longitudinal rebars (row 2): $\phi_{sl,2}$ = 13 [mm]
Diameter of additional transverse rebars (row 2): $\phi_{st,2}$ = 13 [mm]
Pitch of anti-crack longitudinal rebars (row 1): $p_{sl,1}$ = 200 [mm]
Pitch of anti-crack transverse rebars (row 1): $p_{st,1}$ = 200 [mm]
Pitch of additional longitudinal rebars (row 2): $p_{sl,2}$ = 100 [mm]
Pitch of additional transverse rebars (row 2): $p_{st,2}$ = 200 [mm]
Covering depth of anti-crack longitudinal rebars (row 1): $z_{tcs\text{-}csl,1}$ = 60 [mm]
Covering depth of anti-crack transverse rebars (row 1): $z_{tcs\text{-}cst,1}$ = 50 [mm]

Covering depth of additional longitudinal rebars (row 2): $z_{tcs\text{-}csl,2}$ = 36 [mm]
Covering depth of additional transverse rebars (row 2): $z_{tcs\text{-}cst,2}$ = 49 [mm]
Arrangement width of additional longitudinal rebars (row 2): $b_{sl,2}$ = 1500 [mm]
Strength class: B500C
Characteristic yield strength: f_{sk} = 500 [N/mm^2]
Modulus of elasticity: E_s = 210000 [N/mm^2]
Partial factor: γ_s = 1.15
Design yield strength: $f_{sd} = f_{sk}/\gamma_s$ = 435 [N/mm^2]

[Headed stud]

Diameter of shank: d_{hs} = 19 [mm]
Overall height: h_{hs} = 100 [mm]
Ultimate strength: f_{hsu} = 450 [N/mm^2]
Partial factor: γ_V = 1.25
Number per sheeting rib in sagging moment region: n_{hss} = 2
Number per sheeting rib in hogging moment region: n_{hsh} = 2
Distance between centres of outstand headed studs in sagging moment region: b_{0s} = 100 [mm]
Distance between centres of outstand headed studs in hogging moment region: b_{0h} = 100 [mm]
Distance between surface of primary beam and first headed stud: $h_{pb\text{-}fhs}$ = 50 [mm]
Distance between centre of joint and first headed stud: $h_{cj\text{-}fhs}$ = 200 [mm]

[Fin plate]

Depth: D_{fp} = 520 [mm]
Thickness: t_{fp} = 10 [mm]
Radius of gyration of area about minor axis (z-z axis): i_{fpz} = 2.89 [mm]
Leg length of fillet weld: s_{fp} = 10 [mm]
Steel grade: S355
Nominal value of yield strength: f_{fpy} = 355 [N/mm^2] (for t_{fp} = 10 [mm])
Ultimate tensile strength: f_{fpu} = 470 [N/mm^2] (for t_{fp} = 10 [mm])
Modulus of elasticity: E_{fp} = 210000 [N/mm^2]
Partial factor of resistance of members and cross-sections: γ_{fp} = 1.00
Partial factor of resistance of plates in bearing: $\gamma_{fp,2}$ = 1.25

[Contact plate]

Nominal value of yield strength: f_{cpy} = 345 [N/mm^2]
Partial factor of resistance of members and cross-sections: γ_{cp} = 1.00
Partial factor of resistance of plates in bearing: $\gamma_{cp,2}$ = 1.25
Design yield strength: $f_{cpyd} = f_{cpy}/\gamma_{cp}$ = 345 [N/mm^2]

[Bolt]

Size: M20 (d_b = 20 [mm])

Tensile stress area: A_b = 2.45 [cm^2]
Hole diameter: d_0 = 22 [mm]
Strength class: 8.8
Ultimate tensile strength: f_{bu} = 800 [N/mm^2]
Partial factor: γ_b = 1.25
Number on vertical line: $n_{b,v}$ = 7
Number on horizontal line: $n_{b,h}$ = 1
Pitch on vertical line: $p_{b,v}$ = 70 [mm]
Pitch on horizontal line: $p_{b,h}$ = 60 [mm]
Edge distance for fin plate on vertical line: $e_{b\text{-}fp,v}$ = 50 [mm]
Edge distance for fin plate on horizontal line: $e_{b\text{-}fp,h}$ = 50 [mm]
Edge distance for web of steel beam on vertical line: $e_{b\text{-}bw,v}$ = 90 [mm]
Edge distance for web of steel beam on horizontal line: $e_{b\text{-}bw,h}$ = 50 [mm]

Design Loads

[Permanent actions (dead loads and superimposed dead loads)]

Area per unit length of concrete slab:

$$A_c = 1000h_{cs} + \frac{(b_{0,min}+b_{0,max})D_{ps}}{2}\frac{1000}{p_{ps}} = 1443 \text{ [cm}^2\text{/m]}$$

Weight per unit area of concrete slab and rebars:

$$A_c\left(\frac{\rho_c}{100}+2\right) = 3.75 \text{ [kN/m}^2\text{]} \quad \text{(wet concrete)}$$

$$A_c\left(\frac{\rho_c}{100}+1\right) = 3.61 \text{ [kN/m}^2\text{]} \quad \text{(dry concrete)}$$

Weight per unit area of profiled steel sheeting:

$$9.8g_{ps} = 0.14 \text{ [kN/m}^2\text{]}$$

Weight per unit area of steel beam:

$$\frac{9.8g_a}{B_b} = 0.33 \text{ [kN/m}^2\text{]}$$

Dead load per unit area in construction stage:

$$g_{k,1} = 3.75+0.14+0.33 = 4.21 \text{ [kN/m}^2\text{]}$$

Dead load per unit area in composite stage:

$$g_{k,2} = 3.61+0.14+0.33 = 4.07 \text{ [kN/m}^2\text{]}$$

Superimposed dead load per unit area in composite stage:

$$g_{k,3} = 3.00 \text{ [kN/m}^2\text{]}$$

[Variable actions (live loads)]

Construction load per unit area in construction stage:

$$q_{k,1} = 0.50 \text{ [kN/m}^2\text{]}$$

Imposed floor load per unit area in composite stage:
$q_{k,2}$ = 5.00 [kN/m^2]

[Partial factors]

Partial factor for permanent actions (unfavourable): $\gamma_{G,sup}$ = 1.35
Partial factor for permanent actions (favourable): $\gamma_{G,inf}$ = 1.00
Partial factor for variable actions (unfavourable): γ_Q = 1.50
Partial factor for variable actions (favourable): γ_{Qi} = 0.00

<u>Design of Semi-rigid Composite Joints</u>

[Verifications of joint classification]

<u>Check initial rotational stiffness and moment resistance</u>

Vertical distance between centre of longitudinal rebars and centre of contact part for row 1:

$$z_{csl,1\text{-}cc} = D_a + D_{cs} - z_{tcs\text{-}csl,1} - \frac{t_f}{2} = 782 \text{ [mm]}$$

Vertical distance between centre of longitudinal rebars and centre of contact part for row 2:

$$z_{csl,2\text{-}cc} = D_a + D_{cs} - z_{tcs\text{-}csl,2} - \frac{t_f}{2} = 807 \text{ [mm]}$$

Effective width:

$$b_{eff,j} = b_{0h} + \min\left\{\frac{2(0.15L_b)}{4}\ ;\ B_b - b_{0h}\right\} = 1225.0 \text{ [mm]}$$

Cross-sectional area of longitudinal rebars within $b_{eff,j}$ for row 1:

$$A_{sl,1} = \pi\left(\frac{\phi_{sl,1}}{2}\right)^2 \left\lfloor\frac{b_{eff,j}}{p_{sl,1}}\right\rfloor = 4.7 \text{ [cm}^2\text{]}$$

Cross-sectional area of longitudinal rebars within $b_{eff,j}$ for row 2:

$$A_{sl,2} = \pi\left(\frac{\phi_{sl,2}}{2}\right)^2 \left\lfloor\frac{b_{eff,j}}{p_{sl,2}}\right\rfloor = 15.9 \text{ [cm}^2\text{]}$$

Effective length for row 1:

$$l_{eff,1} = \min\left(\frac{2h_{cj\text{-}fhs}}{2}\ ;\ 20\phi_{sl,1}\right) = 200 \text{ [mm]}$$

Effective length for row 2:

$$l_{eff,2} = \min\left(\frac{2h_{cj\text{-}fhs}}{2}\ ;\ 20\phi_{sl,2}\right) = 200 \text{ [mm]}$$

Equivalent stiffness coefficient of longitudinal rebars:

$$k_{sl,eq} = \frac{A_{sl,1}}{l_{eff,1}} + \frac{A_{sl,2}}{l_{eff,2}} = 10.32 \text{ [mm]}$$

Length of composite beam in hogging moment region adjacent to joint:
$l = 0.15L_b = 2250$ [mm]

Number of headed studs distributed over length l:

$$N = \left\lceil \frac{\left(l\text{-}h_{cj\text{-}fhs}\right)}{p_{ps}} \right\rceil n_{hsh} = 22$$

Stiffness of one headed stud with 19 [mm] diameter of shank:

k_{sc} = 100 [kN/mm]

Equivalent vertical distance between longitudinal rebars and centre of contact part:

$$z_{sl,eq\text{-}cc} = D_a + D_{cs} - z_{tcs\text{-}sl,eq} - \frac{t_f}{2} = 801 \text{ [mm]}$$

Equivalent vertical distance between longitudinal rebars and centre of steel beam:

$$z_{sl,eq\text{-}ca} = \frac{D_a}{2} + D_{cs} - z_{tcs\text{-}sl,eq} = 459 \text{ [mm]}$$

Parameter related to deformation of headed studs:

$$\xi = \frac{E_a I_{ay}}{z_{sl,eq\text{-}ca}{}^2 E_s \left(A_{sl,1} + A_{sl,2}\right)} = 2.31$$

$$\nu = \sqrt{\frac{(1+\xi) N k_{sc} l z_{sl,eq\text{-}ca}{}^2}{E_a I_{ay}}} = 4.05$$

Stiffness related to headed studs:

$$K_{sc} = \frac{N k_{sc}}{\nu - \left(\frac{\nu - 1}{1+\xi}\right)\left(\frac{z_{sl,eq\text{-}cc}}{z_{sl,eq\text{-}ca}}\right)} = 902267 \text{ [N/mm]}$$

Stiffness reduction factor due to deformation of headed studs:

$$k_{slip} = \frac{1}{1 + \left(\frac{E_s k_{sl,eq}}{K_{sc}}\right)} = 0.29$$

Initial rotational stiffness:

$$S_{j,ini} = E_s k_{slip} k_{sl,eq} z_{csl,eq\text{-}cc}{}^2 = 408636 \text{ [kNm/rad]}$$

Effective width of composite beam assuming simply supported condition:

$$b_{eff,b} = b_{0s} + \min\left(\frac{L_b}{4} \ ; B_b - b_{0s}\right) = 3000.0 \text{ [mm]}$$

Second moment of area of composite beam assuming simply supported condition:

$$I_b = \frac{A_a \left(h_{cs} + 2D_{ps} + D_a\right)^2}{4\left(1 + \frac{2E_a}{E_{cm,cs}} \frac{A_a}{b_{eff,b} h_{cs}}\right)} + \frac{b_{eff,b} h_{cs}{}^3}{12\left(\frac{2E_a}{E_{cm,cs}}\right)} + I_{ay} = 265164 \text{ [cm}^4\text{]}$$

Upper boundary of rotational stiffness for nominally pinned joint:

$$\frac{0.5 E_a I_b}{L_b} = 18561 \text{ [kNm/rad]}$$

$$\therefore\ S_{j,ini} > \frac{0.5E_aI_b}{L_b} \quad \text{OK} \quad \left(\frac{\left(\frac{0.5E_aI_b}{L_b}\right)}{S_{j,ini}} = 0.05 \right)$$

Cross-sectional area of longitudinal rebars within $b_{eff,j}$:

$A_{sl,j} = A_{sl,1}+A_{sl,2} = 20.6$ [cm^2]

Tension resistance of longitudinal rebars within $b_{eff,j}$:

$R_{sl,j} = A_{sl,j}f_{sd} = 897.4$ [kN]

Cross-sectional area of bottom flange of steel beam:

$A_{bf} = B_at_f = 32.0$ [cm^2]

Cross-sectional area of contact plate (depends on contact plate detailing):

$A_{cp} = 144.0$ [cm^2]

Bearing area of contact plate (depends on contact plate detailing):

$A_{bea} = 25.6$ [cm^2]

Compression resistance of contact part (stiffeners are welded to fin plates and width, thickness, and nominal value of yield strength of stiffeners are more than or equal to those of bottom flange of steel beams):

$$R_{con} = \min\left\{A_{bf}f_{ayd}\ ;\ A_{cp}f_{cpyd}\ ;\ 1.5A_{bea}\min\left(\frac{f_{ay}}{\gamma_{a,2}}\ ;\ \frac{f_{cpy}}{\gamma_{cp,2}}\right)\right\} = 1059.8 \text{ [kN]}$$

Moment resistance:

$M_{j,Rd} = z_{sl,eq\text{-}cc}\min\left(R_{sl,j}\ ;\ R_{con}\right) = 718.7$ [kNm]

Plastic moment resistance of composite beam (This value will be calculated in the design of composite beam):

$M_{pl,Rd} = 1274.6$ [kNm]

Upper boundary of moment resistance for nominally pinned joint:

$0.25M_{pl,Rd} = 318.6$ [kNm]

$$\therefore\ M_{j,Rd} > 0.25M_{pl,Rd} \quad \text{OK} \quad \left(\frac{0.25M_{pl,Rd}}{M_{j,Rd}} = 0.44\right)$$

[Verifications of structural resistance in composite stage]

<u>Check bolt group resistance</u>

Correction factor for bolt shear resistance:

$\alpha_{bV} = 0.60$ (for strength class 8.8)

Shear resistance of a single bolt:

$$F_{bV,Rd} = \frac{\alpha_{bV}f_{bu}A_b}{\gamma_b} = 94.1 \text{ [kN]}$$

Distance between face of support and assumed line of shear transfer:

$z_{fs\text{-}b}$ = 60 [mm]

α factor:

$\alpha = 0.00 \ \left(\text{for } n_{b,h} = 1\right)$

β factor:

$$\beta = \frac{6z_{fs\text{-}b}}{n_{b,v}\left(n_{b,v}+1\right)p_{b,v}} = 0.09 \ \left(\text{for } n_{b,h} = 1\right)$$

Bolt shear resistance:

$$V_{b,Rd} = \frac{n_{b,v}n_{b,h}F_{bV,Rd}}{\sqrt{\left(1+\alpha n_{b,v}n_{b,h}\right)^2+\left(\beta n_{b,v}n_{b,h}\right)^2}} = 554.0 \text{ [kN]}$$

Design shear force (This value will be calculated in the design of composite beam):

V_{Ed} = 383.4 [kN]

$$\therefore \ V_{b,Rd} > V_{Ed} \quad \text{OK} \ \left(\frac{V_{Ed}}{V_{b,Rd}} = 0.69\right)$$

< In fin plate >

k_1 factor for vertical bolt bearing resistance:

$$k_{1,vbb} = \min\left(2.8\frac{e_{b\text{-}fp,h}}{d_0}\text{-}1.7 \ ; 2.5\right) = 2.50 \ \left(\text{for } n_{b,h} = 1\right)$$

Correction factor for vertical bolt bearing resistance:

$$\alpha_{vbb} = \min\left(\frac{e_{b\text{-}fp,v}}{3d_0} \ ; \frac{p_{b,v}}{3d_0}\text{-}\frac{1}{4} \ ; \frac{f_{bu}}{f_{fpu}} \ ; 1.0\right) = 0.76$$

Vertical bearing resistance of a single bolt:

$$F_{vbb,Rd} = \frac{k_{1,vbb}\,\alpha_{vbb}f_{fpu}d_b t_{fp}}{\gamma_b} = 142.4 \text{ [kN]}$$

k_1 factor for horizontal bolt bearing resistance:

$$k_{1,hbb} = \min\left(2.8\frac{e_{b\text{-}fp,v}}{d_0}\text{-}1.7 \ ; 1.4\frac{p_{b,v}}{d_0}\text{-}1.7 \ ; 2.5\right) = 2.50$$

Correction factor for horizontal bolt bearing resistance:

$$\alpha_{hbb} = \min\left(\frac{e_{b\text{-}fp,h}}{3d_0} \ ; \frac{f_{bu}}{f_{fpu}} \ ; 1.0\right) = 0.76 \ \left(\text{for } n_{b,h} = 1\right)$$

Horizontal bearing resistance of a single bolt:

$$F_{hbb,Rd} = \frac{k_{1,hbb}\,\alpha_{hbb}f_{fpu}d_b t_{fp}}{\gamma_b} = 142.4 \text{ [kN]}$$

Bolt bearing resistance:

$$V_{bb,Rd} = \frac{n_{b,v}n_{b,h}}{\sqrt{\left(\frac{1+\alpha n_{b,v}n_{b,h}}{F_{vbb,Rd}}\right)^2 + \left(\frac{\beta n_{b,v}n_{b,h}}{F_{hbb,Rd}}\right)^2}} = 838.6 \text{ [kN]}$$

< In web of steel beam >

Nominal value of yield strength of web of steel beam:

f_{wy} = 355 [N/mm^2] (for t_w = 9 [mm])

Ultimate tensile strength of web of steel beam:

f_{wu} = 470 [N/mm^2] (for t_w = 9 [mm])

k_1 factor for vertical bolt bearing resistance:

$$k_{1,vbb} = \min\left(2.8\frac{e_{b\text{-}bw,h}}{d_0}\text{-}1.7\ ;\ 2.5\right) = 2.50\ \left(\text{for } n_{b,h} = 1\right)$$

Correction factor for vertical bolt bearing resistance:

$$\alpha_{vbb} = \min\left(\frac{e_{b\text{-}bw,v}}{3d_0}\ ;\ \frac{p_{b,v}}{3d_0}\text{-}\frac{1}{4}\ ;\ \frac{f_{bu}}{f_{wu}}\ ;\ 1.0\right) = 0.81$$

Vertical bearing resistance of a single bolt:

$$F_{vbb,Rd} = \frac{k_{1,vbb}\,\alpha_{vbb} f_{wu} d_b t_w}{\gamma_b} = 137.2 \text{ [kN]}$$

k_1 factor for horizontal bolt bearing resistance:

$$k_{1,hbb} = \min\left(2.8\frac{e_{b\text{-}bw,v}}{d_0}\text{-}1.7\ ;\ 1.4\frac{p_{b,v}}{d_0}\text{-}1.7\ ;\ 2.5\right) = 2.50$$

Correction factor for horizontal bolt bearing resistance:

$$\alpha_{hbb} = \min\left(\frac{e_{b\text{-}bw,h}}{3d_0}\ ;\ \frac{f_{bu}}{f_{wu}}\ ;\ 1.0\right) = 0.76\ \left(\text{for } n_{b,h} = 1\right)$$

Horizontal bearing resistance of a single bolt:

$$F_{hbb,Rd} = \frac{k_{1,hbb}\,\alpha_{hbb} f_{wu} d_b t_w}{\gamma_b} = 128.2 \text{ [kN]}$$

Bolt bearing resistance:

$$V_{bb,Rd} = \frac{n_{b,v}n_{b,h}}{\sqrt{\left(\frac{1+\alpha n_{b,v}n_{b,h}}{F_{vbb,Rd}}\right)^2 + \left(\frac{\beta n_{b,v}n_{b,h}}{F_{hbb,Rd}}\right)^2}} = 791.0 \text{ [kN]}$$

$$\therefore\ V_{bb,Rd} > V_{Ed} \quad \text{OK} \quad \left(\frac{V_{Ed}}{V_{bb,Rd}} = 0.48\right)$$

<u>Check fin plate resistance</u>

Shear resistance for gross section:

$$V_{fp,Rd,g} = \frac{D_{fp} t_{fp}}{1.27} \frac{f_{fpy}}{\sqrt{3}\gamma_{fp}} = 839.2 \text{ [kN]}$$

Shear area for net section:

$$A_{fpV,n} = t_{fp}\left(D_{fp}\text{-}n_{b,v} d_0\right) = 36.6 \text{ [cm}^2\text{]}$$

Shear resistance for net section:

$$V_{fp,Rd,n} = A_{fpV,n} \frac{f_{fpu}}{\sqrt{3}\gamma_{fp,2}} = 794.5 \text{ [kN]}$$

Net area subjected to tension:

$$A_{fp,nt} = t_{fp}\left(e_{b\text{-}fp,h}\text{-}\frac{d_0}{2}\right) = 3.9 \text{ [cm}^2\text{]} \left(\text{for } n_{b,h} = 1\right)$$

Net area subjected to shear:

$$A_{fp,nV} = t_{fp}\left\{D_{fp}\text{-}e_{b\text{-}fp,v}\text{-}\left(n_{b,v}\text{-}0.5\right)d_0\right\} = 32.7 \text{ [cm}^2\text{]}$$

Block shear resistance:

$$V_{fp,Rd,b} = \frac{0.5 f_{fpu} A_{fp,nt}}{\gamma_{fp,2}} + \frac{f_{fpy} A_{fp,nV}}{\sqrt{3}\gamma_{fp}} = 743.5 \text{ [kN]}$$

$$\therefore \min\left(V_{fp,Rd,g}\ ;\ V_{fp,Rd,n}\ ;\ V_{fp,Rd,b}\right) > V_{Ed} \quad \text{OK} \quad \left(\frac{V_{Ed}}{\min\left(V_{fp,Rd,g}\ ;\ V_{fp,Rd,n}\ ;\ V_{fp,Rd,b}\right)} = 0.52\right)$$

Elastic moment resistance:

$$M_{el,fp,Rd} = \frac{t_{fp} {D_{fp}}^2}{6} \frac{f_{fpy}}{\gamma_{fp}} = 160.0 \text{ [kNm]}$$

$D_{fp} > 2.73 z_{fs\text{-}b}$ $\rightarrow$ no need to check

Fin plate type:

$$e_{b\text{-}bw,h}+10 > \frac{t_{fp}}{0.15} \quad \rightarrow \text{short fin plate}$$

Non-dimensional slenderness for lateral torsional buckling:

$$\lambda_{LT,fp} = \frac{e_{b\text{-}bw,h}+10}{\pi i_{fpz}} \sqrt{\frac{f_{fpy}}{E_{fp}}} = 0.27$$

Value to determine reduction factor for lateral-torsional buckling:

$$\Phi_{LT,fp} = 0.5\left\{1+0.49\left(\lambda_{LT,fp}\text{-}0.2\right)+{\lambda_{LT,fp}}^2\right\} = 0.55$$

Reduction factor for lateral-torsional buckling:

$$\chi_{LT,fp} = \min\left(\frac{1}{\Phi_{LT,fp}+\sqrt{{\Phi_{LT,fp}}^2\text{-}{\lambda_{LT,fp}}^2}}\ ;\ 1.0\right) = 0.96$$

Lateral torsional buckling moment resistance:

$M_{LT,fp,Rd} = M_{el,fp,Rd} = 160.0$ [kNm] (for short fin plate)

$$\therefore\ M_{LT,fp,Rd} > V_{Ed} z_{fs\text{-}b} \quad \text{OK} \ \left(\frac{V_{Ed} z_{fs\text{-}b}}{M_{LT,fp,Rd}} = 0.14 \right)$$

Check resistance of web of steel beam

Shear area for gross section:

$A_{bwV,g} = A_a\text{-}2B_a t_f + (t_w + 2r) t_f = 70.1$ [cm^2]

Shear resistance for gross section:

$$V_{bw,Rd,g} = A_{bwV,g} \frac{f_{wy}}{\sqrt{3}\gamma_a} = 1436.8 \text{ [kN]}$$

Shear area for net section:

$A_{bwV,n} = A_{bwV,g}\text{-}n_{b,v} d_0 t_w = 56.2$ [cm^2]

Shear resistance for net section:

$$V_{bw,Rd,n} = A_{bwV,n} \frac{f_{wu}}{\sqrt{3}\gamma_{a,2}} = 1220.9 \text{ [kN]}$$

$$\therefore\ \min\left(V_{bw,Rd,g}\ ;\ V_{bw,Rd,n}\right) > V_{Ed} \quad \text{OK} \ \left(\frac{V_{Ed}}{\min\left(V_{bw,Rd,g}\ ;\ V_{bw,Rd,n}\right)} = 0.31 \right)$$

Elastic moment resistance on vertical line of bolts:

$$M_{el,vbw,Rd} = \frac{t_w \left\{ \left(n_{b,v}\text{-}1\right) p_{b,v} \right\}^2}{6} \frac{f_{wy}}{\gamma_a} = 93.9 \text{ [kNm]}$$

Plastic shear resistance on top and bottom horizontal line of bolts:

$$V_{pl,hbw,Rd} = t_w e_{b\text{-}bw,h} \frac{f_{wy}}{\sqrt{3}\gamma_a} = 92.2 \text{ [kN]} \ \left(\text{for } n_{b,h} = 1\right)$$

Design shear force on vertical line of bolts:

$$V_{vbw,Ed} = V_{Ed} \frac{\left(n_{b,v}\text{-}1\right) p_{b,v}}{D_a} = 230.0 \text{ [kN]}$$

Plastic shear resistance on vertical line of bolts:

$$V_{pl,vbw,Rd} = t_w \left(n_{b,v}\text{-}1\right) p_{b,v} \frac{f_{wy}}{\sqrt{3}\gamma_a} = 774.7 \text{ [kN]}$$

Reduced moment resistance on vertical line of bolts making allowance for presence of shear force:

$$M_{y,v,vbw,Rd} = M_{el,vbw,Rd} = 93.9 \text{ [kNm]} \ \left(\text{for } V_{vbw,Ed} \le \frac{V_{pl,vbw,Rd}}{2}\right)$$

$$\therefore \min\left(M_{el,vbw,Rd}\ ;\ M_{y,v,vbw,Rd}\right)+V_{pl,hbw,Rd}\left(n_{b,v}\text{-}1\right)p_{b,v} > V_{Ed}z_{fs\text{-}b} \quad \text{OK}$$

$$\left(\frac{V_{Ed}z_{fs\text{-}b}}{\min\left(M_{el,vbw,Rd}\ ;\ M_{y,v,vbw,Rd}\right)+V_{pl,hbw,Rd}\left(n_{b,v}\text{-}1\right)p_{b,v}} = 0.17\right)$$

Check fillet weld of fin plate

Effective throat thickness of fillet weld:

$a_{fp} = 0.7s_{fp} = 7.0$ [mm]

Required minimum throat thickness of fillet weld:

$a_{fp,req} = 0.6t_{fp} = 6.0$ [mm] ((for S355 steel grade))

$$\therefore a_{fp} > a_{fp,req} \quad \text{OK} \quad \left(\frac{a_{fp,req}}{a_{fp}} = 0.86\right)$$

[Verification of serviceability in composite stage]

Analysis of design moment

Design moment in composite stage can be calculated by the simplified analysis method described in **Appendix II**.

Maximum design distributed load:

$w_{com,max} = B_b\left(g_{k,3}+q_{k,2}\right) = 24.0$ [kN/m]

Design hogging moment (Load-case 2):

$M_{Edh} = 401.4$ [kNm]

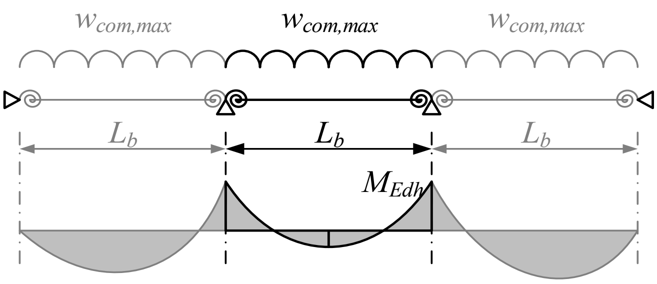


Figure AIII.4: Design load with corresponding moment (Load-case 2)

Maximum diameter of longitudinal rebars:

$$\phi_{sl}^{\ *} = \phi_{sl,max}\left(\frac{2.9}{f_{ctm}}\right) = 14.5 \text{ [mm]}$$

Limit of stress permitted in longitudinal rebars immediately after cracking:

$\sigma_{sl,lim} = 320$ [N/mm^2] (for $w_k = 0.3$ [mm] and $p_{sl} = 100$ [mm])

$$\therefore z_{sl,eq\text{-}cc}\sum A_{sl,r}\,\sigma_{sl,lim} > M_{Edh} \quad \text{OK}$$

$$\left(\frac{M_{Edh}}{z_{sl,eq\text{-}cc}\sum A_{sl,r}\,\sigma_{sl,lim}} = 0.76\right)$$

Design of Internal Composite Beam

[Verifications of structural resistance in construction stage]

Analysis of design moment and shear force

Maximum design distributed load:

$$w_{con,max} = B_b \left(g_{k,1}\gamma_{G,sup} + q_{k,1}\gamma_Q\right) = 19.3 \text{ [kN/m]}$$

Design sagging moment:

$$M_{Eds} = \frac{w_{con,max}L_b^{\,2}}{8} = 543.0 \text{ [kNm]}$$

Design shear force:

$$V_{Ed} = \frac{w_{con,max}L_b}{2} = 144.8 \text{ [kN]}$$

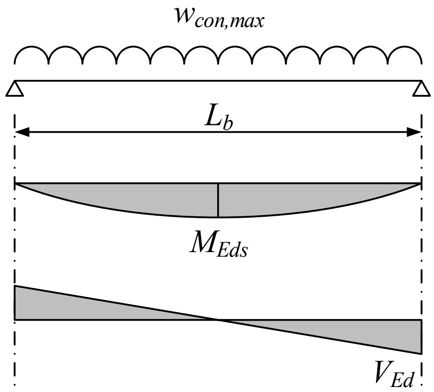


Figure AIII.5: Design load with corresponding moment and shear force

Check section classification

Classification of steel flange:

$$\frac{B_a - t_w - 2r}{2t_f} = 4.84 < 9\sqrt{\frac{235}{f_{ay}}} = 7.32 \quad \rightarrow \text{Class 1}$$

Classification of steel web:

$$\frac{D_a - 2t_f - 2r}{t_w} = 70.2 < 124\sqrt{\frac{235}{f_{ay}}} = 100.9 \quad \rightarrow \text{Class 3}$$

∴ Class 1 steel flange & Class 3 steel web → Effective Class 2 OK

Check shear resistance and moment resistance

Shear area:

$$A_V = \max\left\{A_a - 2B_a t_f + (t_w + 2r)t_f \,;\, 1.2D_a t_w\right\} = 72.1 \text{ [cm}^2\text{]}$$

Plastic shear resistance:

$$V_{pl,a,Rd} = \frac{A_V f_{ayd}}{\sqrt{3}} = 1478.7 \text{ [kN]}$$

Nominal value of yield strength of web:

$f_{wy} = 355$ [N/mm^2] (for $t_w = 9$ [mm])

Minimum shear buckling coefficient:

$k_{\tau,min} = 5.34$ (without rigid transverse and longitudinal stiffeners)

Modified slenderness of web:

$$\lambda_w = 0.76\sqrt{\frac{f_{wy}}{k_{\tau,min}\left\{190000\left(\frac{t_w}{D_a - 2t_f}\right)^2\right\}}} = 1.06$$

Factor for contribution of web to the shear buckling resistance:

$$\chi_w = \frac{0.83}{\lambda_w} = 0.79 \left(\text{for } \frac{0.83}{1.2} \le \lambda_w\right)$$

Shear buckling resistance:

$$V_{b,a,Rd} = \min\left\{\frac{\chi_w f_{wy}\left(D_a\text{-}2t_f\text{-}2r\right)t_w}{\sqrt{3}\gamma_a} ; \frac{1.2 f_{wy}\left(D_a\text{-}2t_f\text{-}2r\right)t_w}{\sqrt{3}\gamma_a}\right\} = 969.3 \text{ [kN]}$$

$$\left(\text{for } \frac{D_a\text{-}2t_f}{t_w} > \frac{72}{1.2}\sqrt{\frac{235}{f_{ay}}}\right)$$

$$\therefore \min\left(V_{pl,a,Rd} ; V_{b,a,Rd}\right) > V_{Ed} \quad \text{OK} \left(\frac{V_{Ed}}{\min\left(V_{pl,a,Rd} ; V_{b,a,Rd}\right)} = 0.15\right)$$

Effective plastic section modulus:

$$W_{eff,pl,a} = \left\{\begin{array}{c} B_a t_f\left(D_a\text{-}t_f\right)+0.4292r^2\left(D_a\text{-}2t_f\text{-}0.4467r\right)+t_w r\left(D_a\text{-}2t_f\text{-}r\right) \\ +\frac{t_w}{2}\left(40t_w\sqrt{\frac{235}{f_{ay}}}\right)^2+\frac{t_w}{2}\left(20t_w\sqrt{\frac{235}{f_{ay}}}\right)^2 \\ +t_w\left(20t_w\sqrt{\frac{235}{f_{ay}}}\right)\left(D_a\text{-}t_f\text{-}r\text{-}40t_w\sqrt{\frac{235}{f_{ay}}}\text{-}t_f\text{-}r\text{-}10t_w\sqrt{\frac{235}{f_{ay}}}\right)\end{array}\right\} = 3219 \text{ [cm}^3\text{]}$$

Plastic moment resistance:

$M_{pl,a,Rd} = W_{eff,pl,a} f_{ayd} = 1142.7$ [kNm] (for effective Class 2 cross-section)

$$\therefore M_{pl,a,Rd} > M_{Eds} \quad \text{OK} \left(\frac{M_{Eds}}{M_{pl,a,Rd}} = 0.48\right)$$

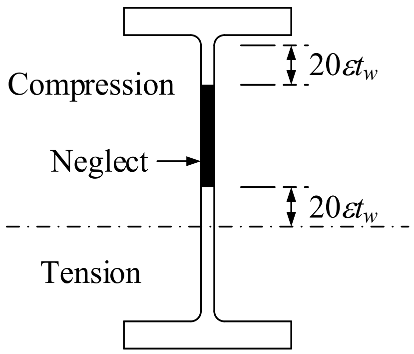


Figure AIII.6: Effective cross-section for effective Class 2

[Verifications of structural resistance in composite stage]

Analysis of design moment and shear force

Design moment and shear force in composite stage can be calculated by the simplified analysis method described in **Appendix II**.

Maximum design distributed load:

$$w_{com,max} = B_b\left(g_{k,1}\gamma_{G,sup}+g_{k,3}\gamma_{G,sup}+q_{k,1}\gamma_Q\right) = 51.1 \text{ [kN/m]}$$

Minimum design distributed load:

$$w_{com,min} = B_b\left(g_{k,1}\gamma_{G,inf}+g_{k,3}\gamma_{G,inf}+q_{k,1}\gamma_{Qi}\right) = 21.2 \text{ [kN/m]}$$

Design sagging moment (Load-case 1):

M_{Eds} = 950.8 [kNm]

Design hogging moment (Load-case 2):

M_{Edh} = 718.7 [kNm]

Design shear force:

V_{Ed} = 383.4 [kN]

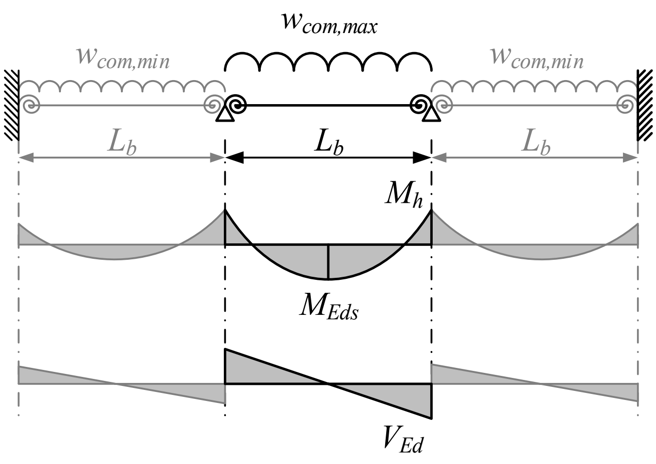


Figure AIII.7: Design load with corresponding moment and shear force (Load-case 1)

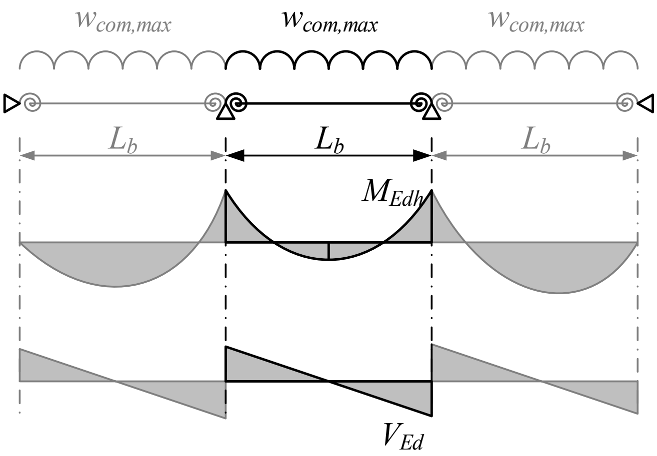


Figure AIII.8: Design load with corresponding moment and shear force (Load-case 2)

<u>Check section classification</u>

< Load-case 1 for maximizing sagging moment >

Classification of steel flange in sagging moment region:
Bottom flange is in tension → Class 1

Effective width in sagging moment region:
b_{effs} = 3000.0 [mm]

Compression resistance of composite slab within b_{effs}:

$$R_{cs} = b_{effs} h_{cs} \left(0.85 f_{cd,cs}\right) = 4207.5 \text{ [kN]}$$

Portion of part of cross-section in compression:

$$\alpha = \frac{\left(\frac{D_a\text{-}2t_f\text{-}2r}{2}\right) - \left(\frac{R_{cs}}{2t_w f_{ayd}}\right)}{D_a\text{-}2t_f\text{-}2r} = \text{-}0.54$$

Classification of steel web in sagging moment region:
Full web is in tension → Class 1

∴ Class 1 steel flange & Class 1 steel web → Class 1 OK

< Load-case 2 for maximizing hogging moment >

Classification of steel flange in hogging moment region:

$$\frac{(B_a\text{-}t_w\text{-}2r)}{2t_f} = 4.84 < 9\sqrt{\frac{235}{f_{ay}}} = 7.32 \quad \rightarrow \text{Class 1}$$

Effective width in hogging moment region:
b_{effh} = 1117.7 [mm]

Cross-sectional area of longitudinal rebars within b_{effh}:

$$A_{sl} = \pi\left(\frac{\phi_{sl,1}}{2}\right)^2 \left\lfloor\frac{b_{effh}}{p_{sl,1}}\right\rfloor + \pi\left(\frac{\phi_{sl,2}}{2}\right)^2 \min\left(\left\lfloor\frac{b_{effh}}{p_{sl,2}}\right\rfloor ; \left\lfloor\frac{b_{sl,2}}{p_{sl,2}}\right\rfloor\right) = 18.53 \text{ [cm}^2\text{]}$$

Tension resistance of longitudinal rebars within b_{effh}:

$$R_{sl} = A_{sl} f_{sd} = 805.5 \text{ [kN]}$$

Portion of part of cross-section in compression:

$$\alpha = \frac{\left(\frac{D_a\text{-}2t_f\text{-}2r}{2}\right) + \left(\frac{R_{sl}}{2t_w f_{ayd}}\right)}{D_a\text{-}2t_f\text{-}2r} = 0.70$$

Equivalent vertical distance between longitudinal rebars and bottom of flange of steel beam:

$$z_{sl,eq\text{-}bf} = D_a + D_{cs} - z_{tcs\text{-}sl,eq} = 809.3 \text{ [mm]}$$

Stress or strain ratio:

$$\psi = 1 - \frac{D_a\text{-}2t_f\text{-}2r}{\frac{A_a D_a + 2A_{sl} z_{sl,eq\text{-}bf}}{2(A_a + A_{sl})} - t_f - r} = -0.69$$

Classification of steel web in hogging moment region:

$$\frac{D_a\text{-}2t_f\text{-}2r}{t_w} = 70.2 < \frac{42\sqrt{\frac{235}{f_{ay}}}}{0.67 + 0.33\psi} = 77.1 \quad \rightarrow \text{Class 3}$$

∴ Class 1 steel flange & Class 3 steel web → Effective Class 2 OK

Second moment of area in hogging moment region:

$$I_h = I_{ay} + \frac{A_a A_{sl} \left\{D_a + 2\left(D_{ps} + h_{cs} - z_{tcs\text{-}sl,eq}\right)\right\}^2}{4(A_a + A_{sl})} = 134362 \text{ [cm}^4\text{]}$$

Elastic hogging moment resistance:

$$M_{el,Rdh} = \min\left\{\frac{I_h f_{sd}}{z_{sl,eq\text{-}bf} - \frac{A_a D_a + 2A_{sl} z_{sl,eq\text{-}bf}}{2(A_a + A_{sl})}} \; ; \; \frac{I_h f_{ayd}}{\frac{A_a D_a + 2A_{sl} z_{sl,eq\text{-}bf}}{2(A_a + A_{sl})}}\right\} = 1167.6 \text{ [kNm]}$$

Modular ratio for short-term loading:

$$n_0 = \frac{E_a}{E_{cm,cs}} = 6.77$$

Vertical distance between centre of un-cracked concrete flange and un-cracked composite section:

$$z_0 = \frac{(A_a + A_{sl})\left(0.5D_a + D_{ps} + 0.5h_{cs}\right)}{A_a + A_{sl} + \left(\frac{h_{cs} b_{effh}}{n_0}\right)} = 212.2 \text{ [mm]}$$

Coefficient taking into account of stress distribution within section immediately prior to cracking:

$$k_c = \min\left\{\frac{1}{1+\left(\frac{h_{cs}}{2z_0}\right)}+0.3\ ;\ 1.0\right\} = 1.00$$

Required minimum reinforcement ratio:

$$\rho_{sl,req} = \frac{f_{ay}}{235}\frac{f_{ctm}}{f_{sk}}\sqrt{k_c} = 0.79\%\ (\text{for Class 2 cross-section})$$

Cross-sectional area of composite slab within b_{effh} above profiled steel sheeting:

$$A_{cs} = b_{effh}h_{cs} = 1106.5\ [\text{cm}^2]$$

Required minimum cross-sectional area of longitudinal rebars within b_{effh}:

$$A_{sl,req} = \rho_{sl,req}A_{cs} = 8.69\ [\text{cm}^2]$$

$M_{el,Rdh} > M_{j,Rd}$ $\rightarrow$ no need to check $A_{sl} > A_{sl,req}$

Check minimum degree of shear connection

< Load-case 1 for maximizing sagging moment >

Distance between inflection points in sagging moment region:

$$L_{es} = L_b\text{-}2x_0 = 12198\ [\text{mm}]$$

Correction factor of headed stud taking into account h_{hs}/d_{hs}:

$$\alpha_{hs} = 1.00\ \left(\text{for } \frac{h_{hs}}{d_{hs}} > 4\right)$$

Shear resistance of a headed stud:

$$P_{Rd} = \min\left(\frac{0.8f_{hsu}\pi d_{hs}^{\ 2}}{4\gamma_V}\ ;\ \frac{0.29\alpha d_{hs}^{\ 2}\sqrt{f_{ck,cs}E_{cm,cs}}}{\gamma_V}\right) = 73.7\ [\text{kN}]$$

Number of headed studs arranged within half of L_{es}:

$$N_{hss} = \left\lfloor\frac{\left(\frac{L_{es}}{2}\right)}{p_{ps}}\right\rfloor n_{hss} = 62$$

Maximum reduction factor for shear resistance of a headed stud in sagging moment region:

$$k_{ts,max} = 0.60\ \left(\text{for } n_{hss} = 2,\ t_{ps} \le 1,\ d_{hs} \le 20, \text{ and sheeting with holes}\right)$$

Reduction factor for shear resistance of a headed stud in sagging moment region:

$$k_{ts} = \min\left\{\frac{0.7}{\sqrt{\min(n_{hss}\ ;\ 2)}}\frac{b_{0,min}}{D_{ps}}\left(\frac{h_{hs}}{D_{ps}}\text{-}1\right)\ ;\ k_{ts,max}\right\} = 0.60$$

Longitudinal shear force transfer within half of L_{es}:

$$R_{qs} = N_{hss}k_{ts}P_{Rd} = 2742.8\ [\text{kN}]$$

Tension (Compression) resistance of steel beam:
$R_a = A_a f_{ayd} = 4505.0$ [kN]

Degree of shear connection in sagging moment region:
$$\eta_s = \frac{R_{qs}}{\min(R_a\ ;\ R_{cs})} = 0.65$$

Required minimum degree of shear connection in sagging moment region:
$$\eta_{s,req} = \max\left\{1\text{-}\left(\frac{355}{f_{ayd}}\right)(0.75\text{-}0.03L_{es});\ 0.4\right\} = 0.62\ (\text{for } L_{es} \leq 25\ [\text{m}])$$

$$\therefore\ \eta_s > \eta_{s,req}\quad \text{OK}\ \left(\frac{\eta_{s,req}}{\eta_s} = 0.94\right)$$

< Load-case 2 for maximizing hogging moment >

Half of distance between inflection points in hogging moment region:
$$\frac{L_{eh}}{2} = x_0 = 2196\ [\text{mm}]$$

Number of headed studs arranged within half of L_{eh}:
$$N_{hsh} = \left\lfloor \frac{\left(\frac{L_{eh}}{2}\right)\text{-}h_{cj\text{-}fhs}}{p_{ps}} \right\rfloor n_{hsh} = 20$$

Maximum reduction factor for shear resistance of a headed stud in hogging moment region:
$k_{th,max} = 0.60$ (for $n_{hsh} = 2$, $t_{ps} \leq 1$, $d_{hs} \leq 20$, and sheeting with holes)

Reduction factor for shear resistance of a headed stud in hogging moment region:
$$k_{th} = \min\left\{\frac{0.7}{\sqrt{\min(n_{hsh}\ ;\ 2)}}\frac{b_{0,min}}{D_{ps}}\left(\frac{h_{hs}}{D_{ps}}\text{-}1\right)\ ;\ k_{th,max}\right\} = 0.60$$

Longitudinal shear force transfer within half of L_{eh}:
$R_{qh} = N_{hsh} k_{th} P_{Rd} = 884.8$ [kN]

Degree of shear connection in hogging moment region:
$$\eta_h = \frac{R_{qh}}{\min(R_a\ ;\ R_{sl})} = 1.10$$

Required minimum degree of shear connection in hogging moment region:
$\eta_{h,req} = 1.00$ (full shear connection)

$$\therefore\ \eta_h > \eta_{h,req}\quad \text{OK}\ \left(\frac{\eta_{h,req}}{\eta_h} = 0.91\right)$$

Check shear resistance and moment resistance

Plastic shear resistance:
$V_{pl,Rd} = 1478.7$ [kN]

Shear buckling resistance:
$V_{b,Rd} = 969.3$ [kN]

$$\therefore \min\left(V_{pl,Rd}\ ;\ V_{b,Rd}\right) > V_{Ed} \quad \text{OK} \quad \left(\frac{V_{Ed}}{\min\left(V_{pl,Rd}\ ;\ V_{b,Rd}\right)} = 0.40\right)$$

< Load-case 1 for maximizing sagging moment >

Tension (Compression) resistance of overall web of steel beam:
$R_w = R_a\text{-}2B_a t_f f_{ayd} = 2233.0$ [kN]

Tension (Compression) resistance of clear web of steel beam:
$R_v = \left(D_a\text{-}2t_f\text{-}2r\right)t_w f_{ayd} = 2019.2$ [kN]

Tension (Compression) resistance of effective clear web of steel beam:

$$R_{eff,v} = 40{t_w}^2 f_{ayd}\sqrt{\frac{235}{f_{ay}}} = 935.8 \text{ [kN]}$$

Location of plastic neutral axis for full shear connection:
$R_w \leq R_{cs} < R_a \quad \rightarrow PNA$ in steel flange

Plastic sagging moment resistance with full shear connection:

$$M_{plf,Rds} = R_a\frac{D_a}{2} + R_{cs}\left(\frac{D_{cs}+D_{ps}}{2}\right) - \frac{(R_a\text{-}R_{cs})^2}{4B_a f_{ayd}} = 1999.3 \text{ [kNm]}$$

Location of plastic neutral axis for partial shear connection:
$R_w \leq R_{qs} \quad \rightarrow PNA$ in steel flange

Plastic sagging moment resistance with partial shear connection:

$$M_{plp,Rds} = R_a\frac{D_a}{2} + R_{qs}\left(D_{cs} - \frac{R_{qs}}{R_{cs}}\frac{D_{cs}\text{-}D_{ps}}{2}\right) - \frac{\left(R_a\text{-}R_{qs}\right)^2}{4B_a f_{ayd}} = 1888.7 \text{ [kNm]}$$

$$\therefore \min\left(M_{plf,Rds}\ ;\ M_{plp,Rds}\right) > M_{Eds} \quad \text{OK} \quad \left(\frac{M_{Eds}}{\min\left(M_{plf,Rds}\ ;\ M_{plp,Rds}\right)} = 0.50\right)$$

< Load-case 2 for maximizing hogging moment >

Equivalent vertical distance between longitudinal rebars and top of flange of steel beam:
$z_{sl,eq\text{-}tf} = D_{cs}\text{-}z_{tcs\text{-}sl,eq} = 109.3$ [mm]

Tension (Compression) resistance of effective steel beam:
$R_{eff,a} = R_a\text{-}R_v + R_{eff,v} = 3421.6$ [kN]

Location of plastic neutral axis for with full shear connection:
$R_{sl} < R_{eff,v} \quad \rightarrow PNA$ in steel web

Plastic hogging moment resistance with full shear connection:

$$M_{plf,Rdh} = \left\{ \begin{array}{c} W_{pl,a}f_{ayd}+R_{sl}\left(\dfrac{D_a}{2}+z_{sl,eq\text{-}tf}\right) \\ -\dfrac{R_{sl}{}^2+(R_v+R_{sl})\left(R_v+R_{sl}\text{-}2R_{eff,v}\right)}{4t_wf_{ayd}} \end{array} \right\} = 1274.6 \text{ [kNm]}$$

Tension (Compression) resistance of flange of steel beam:

$$R_f = B_at_ff_{ayd} = 1136.0 \text{ [kN]}$$

Vertical distance between centres of top and bottom flange of steel beam:

$$z_{ctf\text{-}cbf} = D_a\text{-}t_f = 684.0 \text{ [mm]}$$

Plastic moment resistance after deducting shear area:

$$M_{pl,f,Rd} = R_fz_{ctf\text{-}cbf}+R_{sl}\left(\frac{D_a}{2}+z_{sl,eq\text{-}tf}\right) = 1147.0 \text{ [kNm]}$$

Reduced hogging moment resistance making allowance for presence of shear force:

$$M_{y,v,Rdh} = M_{plf,Rdh} = 1274.6 \text{ [kNm]} \left(\text{for } V_{Ed} \leq \frac{V_{pl,Rd}}{2}\right)$$

$$\therefore \min\left(M_{plf,Rdh}\ ;\ M_{y,v,Rdh}\right) > M_{Edh} \quad \text{OK} \quad \left(\frac{M_{Edh}}{\min\left(M_{plf,Rdh}\ ;\ M_{y,v,Rdh}\right)} = 0.56\right)$$

Check lateral-torsional buckling

< Load-case 2 for maximizing hogging moment >

Polar radius of gyration of area of steel beam:

$$i_{ax} = \sqrt{\frac{\left(I_{ay}+I_{az}\right)}{A_a}} = 284.1 \text{ [mm]}$$

Vertical distance between centre of composite slab and centre of steel beam:

$$z_{ccs\text{-}ca} = \frac{D_a}{2}+\frac{D_{cs}}{2} = 425.0 \text{ [mm]}$$

e value:

$$e = \frac{(A_a+A_{sl})I_{ay}}{A_az_{ccs\text{-}ca}A_{sl}} = 1459.1 \text{ [mm]}$$

k_c factor:

$$k_c = \frac{\dfrac{z_{ctf\text{-}cbf}I_h}{I_{ay}}}{\left(\dfrac{z_{ctf\text{-}cbf}{}^2}{4}+i_{ax}{}^2\right)/e+z_{ctf\text{-}cbf}} = 1.12$$

Property of distribution of moment:

$$C_4 = 21.9 \left(\text{for } 0.50 < \psi = \frac{M_{Edh}}{\left(\frac{w_{com,max}L_b^{\,2}}{8}\right)} < 0.75 \right)$$

Length between points at which bottom flange of steel beam is laterally restrained:

$L_{cr} = L_b = 15000$ [mm]

Cross-sectional area of transverse rebars per unit length:

$$A_{st} = \left\lfloor \frac{1000}{p_{st,1}} \right\rfloor \pi \left(\frac{\phi_{st,1}}{2}\right)^2 + \left\lfloor \frac{1000}{p_{st,2}} \right\rfloor \pi \left(\frac{\phi_{st,2}}{2}\right)^2 = 17.20 \text{ [cm}^2\text{/m]}$$

Area per unit length of concrete slab in compression:

$$A_{c,c} = \frac{A_c\text{-}1000h_{cs}}{\left(\frac{E_a}{E_{cm,cs}}\right)} = 66.8 \text{ [cm}^2\text{/m]}$$

Equivalent vertical distance between transverse rebars and concrete slab in compression:

$$z_{st,eq\text{-}ccs,c} = D_{cs}\text{-}z_{tcs\text{-}st,eq}\text{-}\frac{D_{ps}}{2} = 75.7 \text{ [mm]}$$

Equivalent Vertical distance between transverse rebars and neutral axis of composite slab:

$$z_{st,eq\text{-}na} = \frac{z_{st,eq\text{-}ccs,c}}{\left(1+\frac{A_{st}}{A_{c,c}}\right)} = 60.2 \text{ [mm]}$$

Vertical distance between neutral axis of composite slab and centre of concrete slab in compression:

$z_{na\text{-}ccs,c} = z_{st,eq\text{-}ccs,c}\text{-}z_{st,eq\text{-}na} = 15.5$ [mm]

Second moment of area of cracked composite slab in direction transverse to steel beam:

$$I_{cs2} = A_{st}z_{st,eq\text{-}na}{}^2 + A_{c,c}\left(z_{na\text{-}ccs,c}{}^2 + \frac{D_{ps}{}^2}{12}\right) = 927.8 \text{ [cm}^4\text{]}$$

Cracked flexural stiffness per unit length of composite slab:

$E_aI_{cs2} = 1948.4$ [kNm^2/m]

Flexural stiffness of cracked composite slab in direction transverse to steel beam:

$$k_1 = \frac{4E_aI_{cs2}}{B_b} = 2597.9 \text{ [kN/rad] (for continuous slab)}$$

Flexural stiffness of web of steel beam:

$$k_2 = \frac{E_a t_w{}^3}{4\left(1\text{-}0.3^2\right)z_{ctf\text{-}cbf}} = 61.5 \text{ [kN/rad]}$$

Transverse (rotational) stiffness per unit length:

$$k_s = \frac{k_1k_2}{k_1+k_2} = 60.1 \text{ [kN/rad]}$$

Second moment of area of bottom flange of steel beam about minor axis (z-z axis):

$$I_{bfz} = \frac{t_f B_a^{\ 3}}{12} = 1067 \text{ [cm}^4\text{]}$$

Elastic critical moment for lateral-torsional buckling:

$$M_{cr} = \frac{k_c C_4}{L_{cr}} \sqrt{\left[\left\{\frac{E_a}{2(1+0.3)}\right\} I_{T,a} + \frac{k_s L_{cr}^{\ 2}}{\pi^2}\right] E_a I_{bfz}} = 2928.6 \text{ [kNm]}$$

Location of plastic neutral axis for $M_{pl,Rkh}$:

$R_{eff,v} \leq A_{sl} f_{sk} < R_{eff,a} \quad \rightarrow PNA$ in steel flange

Characteristic value of plastic hogging moment resistance:

$$M_{pl,Rkh} = R_{eff,a} \frac{D_a}{2} + A_{sl} f_{sk} z_{csl\text{-}tf} - \frac{\left(R_{eff,a} - A_{sl} f_{sk}\right)^2}{4 B_a f_{ayd}} = 1276.8 \text{ [kNm]}$$

Non-dimensional slenderness for lateral-torsional buckling:

$$\lambda_{LT} = \sqrt{\frac{M_{pl,Rkh}}{M_{cr}}} = 0.66$$

Imperfection factor corresponding to appropriate lateral-torsional buckling curve:

$$\alpha_{LT} = 0.76 \left(\text{for } \frac{D_a}{B_a} = 3.5 > 2.0\right)$$

Value to determine reduction factor for lateral-torsional buckling:

$$\Phi_{LT} = 0.5\left\{1 + \alpha_{LT}(\lambda_{LT} - 0.4) + 0.75 \lambda_{LT}^{\ 2}\right\} = 0.76$$

Reduction factor for lateral-torsional buckling:

$$\chi_{LT} = \min\left(\frac{1}{\Phi_{LT} + \sqrt{\Phi_{LT}^{\ 2} - 0.75 \lambda_{LT}^{\ 2}}} \ ; \ 1.0 \ ; \ \frac{1}{\lambda_{LT}^{\ 2}}\right) = 0.79$$

Buckling moment resistance of laterally unrestrained composite beam:

$M_{LT,Rd} = \chi_{LT} M_{plf,Rdh} = 1006.2$ [kNm]

Requirements to use calculation method in DD ENV 1994-1-1:

$$E_{cm,cs} I_{cs2} \geq 0.35 E_a t_w^{\ 2} \frac{B_b}{D_a} \ \text{ and } \ \frac{p_{ps}}{B_a} \leq 0.4 f_{hsu} d_{hs}^{\ 2} \frac{1 - \chi_{LT} \lambda_{LT}^{\ 2}}{k_s \chi_{LT} \lambda_{LT}^{\ 2}} \quad \rightarrow \text{can be used}$$

$$\therefore \ M_{LT,Rd} > M_{Edh} \quad \text{OK} \left(\frac{M_{Edh}}{M_{LT,Rd}} = 0.71\right)$$

Check longitudinal shear resistance

< Load-case 1 for maximizing sagging moment >

Effective width in hogging moment region:

$b_{effh} = 1522.4$ [mm]

Cross-sectional area of longitudinal rebars within b_{effh}:

$$A_{sl} = \pi\left(\frac{\phi_{sl,1}}{2}\right)^2 \left\lfloor\frac{b_{effh}}{p_{sl,1}}\right\rfloor + \pi\left(\frac{\phi_{sl,2}}{2}\right)^2 \min\left(\left\lfloor\frac{b_{effh}}{p_{sl,2}}\right\rfloor \; ; \; \left\lfloor\frac{b_{sl,2}}{p_{sl,2}}\right\rfloor\right) = 25.41 \text{ [cm}^2\text{]}$$

Tension resistance of longitudinal rebars within b_{effh}:

$$R_{sl} = A_{sl} f_{sd} = 1104.7 \text{ [kN]}$$

Change of longitudinal force in composite slab:

$$\Delta N_L = \left\{ \begin{array}{c} \min(R_a \; ; R_{cs} \; ; N_{hss} P_{Rd}) \\ +\min\left(\left\lceil\dfrac{x_0\text{-}h_{cj\text{-}fhs}}{p_{ps}}\right\rceil n_{hsh} P_{Rd} \; ; R_{sl}\right) \end{array} \right\} = 5240 \text{ [kN]}$$

Design longitudinal shear stress in composite slab:

$$v_{L,Ed} = \frac{\Delta N_L}{2h_{cs}\left(\frac{L_b}{2}\right)} = 3.53 \text{ [N/mm}^2\text{]}$$

Minimum angle to minimize cross-sectional area of transverse rebars:

$$\theta_{min} = 38.6\ °$$

Required tension resistance of transverse reinforcement per unit length:

$$R_{tr,req} = \frac{1000 h_{cs} v_{L,Ed}}{\cot\theta_{min}} = 278.9 \text{ [kN/m]}$$

< Load-case 2 for maximizing hogging moment >

Effective width in sagging moment region:

$$b_{effs} = 2752.0 \text{ [mm]}$$

Compression resistance of composite slab within b_{effs}:

$$R_{cs} = b_{effs} h_{cs}\left(0.85 f_{cd,cs}\right) = 3859.6 \text{ [kN]}$$

Distance between inflection points in sagging moment region:

$$L_{es} = L_b\text{-}2x_0 = 10608 \text{ [mm]}$$

Number of headed studs arranged within half of L_{es}:

$$N_{hss} = \left\lceil\frac{\left(\frac{L_{es}}{2}\right)}{p_{ps}}\right\rceil n_{hss} = 54$$

Change of longitudinal force in composite slab:

$$\Delta N_L = \left\{ \begin{array}{c} \min(R_a \; ; R_{cs} \; ; N_{hss} P_{Rd}) \\ +\min\left(\left\lceil\dfrac{x_0\text{-}h_{cj\text{-}fhs}}{p_{ps}}\right\rceil n_{hsh} P_{Rd} \; ; R_{sl}\right) \end{array} \right\} = 4665 \text{ [kN]}$$

Design longitudinal shear stress in composite slab:

$$v_{L,Ed} = \frac{\Delta N_L}{2h_{cs}\left(\frac{L_b}{2}\right)} = 3.14 \text{ [N/mm}^2\text{]}$$

Minimum angle to minimize cross-sectional area of transverse rebars:

$\theta_{min} = 38.6\ °$

Required tension resistance of transverse reinforcement per unit length:

$$R_{tr,req} = \frac{1000 h_{cs} v_{L,Ed}}{\cot\theta_{min}} = 248.3 \text{ [kN/m]}$$

Cross-sectional area of transverse rebars per unit length for row 1:

$$A_{st,1} = \left\lfloor \frac{1000}{p_{st,1}} \right\rfloor \pi \left(\frac{\phi_{st,1}}{2} \right)^2 = 3.93 \text{ [cm}^2\text{/m]}$$

Tension resistance of transverse reinforcement per unit length:

$R_{st}+R_{pse} = A_{st,1} f_{sd} + A_{pse} f_{psd} = 668.5$ [kN/m]

$$\therefore\ R_{st}+R_{pse} > \max\left(R_{tr,req}\right) \quad \text{OK} \quad \left(\frac{\max\left(R_{tr,req}\right)}{R_{st}+R_{pse}} = 0.42 \right)$$

Required minimum cross-sectional area of transverse rebars per unit length:

$$A_{st,req} = 1000 h_{cs} \frac{0.08\sqrt{f_{ck,cs}}}{f_{sk}} = 0.79 \text{ [cm}^2\text{/m]}$$

$$\therefore\ A_{st,1} > A_{st,req} \quad \text{OK} \quad \left(\frac{A_{st,req}}{A_{st,1}} = 0.20 \right)$$

Crushing shear stress of concrete slab:

$$v_{Rd} = 0.6 \left(1 - \frac{f_{ck,cs}}{250} \right) f_{cd,cs} \sin\theta_{min} \cos\theta_{min} = 4.39 \text{ [N/mm}^2\text{]}$$

$$\therefore\ v_{Rd} > \max\left(v_{L,Ed}\right) \quad \text{OK} \quad \left(\frac{\max\left(v_{L,Ed}\right)}{v_{Rd}} = 0.80 \right)$$

[Verifications of serviceability in construction stage]

<u>Analysis of deflection</u>

Design distributed load due to “dead loads”:

$w_{con,P} = B_b g_{k,1} = 12.6$ [kN/m]

Design distributed load due to “live loads”:

$w_{con,V} = B_b q_{k,1} = 1.5$ [kN/m]

Design distributed load due to “dead loads and live loads”:

$w_{con,P+V} = B_b \left(g_{k,1} + q_{k,1} \right) = 14.1$ [kN/m]

Deflection due to “dead loads”:

$$\delta_P = \frac{w_{con,P} {L_b}^4}{384 E_a I_{ay}} = 39.6 \text{ [mm]}$$

Deflection due to “live loads”:

$$\delta_V = \frac{w_{con,V} {L_b}^4}{384 E_a I_{ay}} = 4.7 \text{ [mm]}$$

Deflection due to “dead loads and live loads”:

$$\delta_{P+V} = \frac{w_{con,P+V} {L_b}^4}{384 E_a I_{ay}} = 44.3 \text{ [mm]}$$

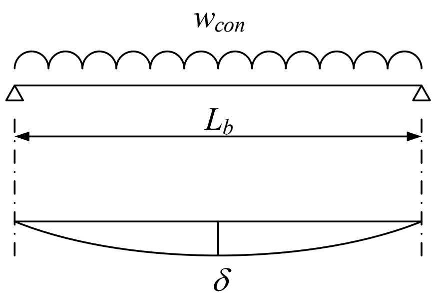


Figure AIII.9: Design load with corresponding deflection

Check deflection

Limit of deflection due to “live loads”:

$$\delta_{V,lim} = \frac{L_b}{360} = 41.7 \text{ [mm]}$$

Limit of deflection due to “dead loads and live loads”:

$$\delta_{P+V,lim} = \frac{L_b}{250} = 60.0 \text{ [mm]}$$

$$\therefore\ \delta_V < \delta_{V,lim} \quad \text{OK} \left(\frac{\delta_V}{\delta_{V,lim}} = 0.11 \right)$$

$$\therefore\ \delta_{P+V} < \delta_{P+V,lim} \quad \text{OK} \left(\frac{\delta_{P+V}}{\delta_{P+V,lim}} = 0.74 \right)$$

[Verifications of serviceability in composite stage]

Analysis of deflection

Deflection in composite stage can be calculated by the simplified analysis method described in **Appendix II**.

Design distributed load due to “superimposed dead loads”:

$w_{com,P} = B_b g_{k,3} = 9.0$ [kN/m]

Maximum design distributed load due to “live loads”:

$w_{com,V,max} = B_b q_{k,2} = 15.0$ [kN/m]

Minimum design distributed load due to “live loads”:

$w_{com,V,min} = 0.0$ [kN/m]

Deflection due to “superimposed dead loads”:

$\delta_P = 4.0$ [mm]

Deflection due to "live loads" (Load-case 1):

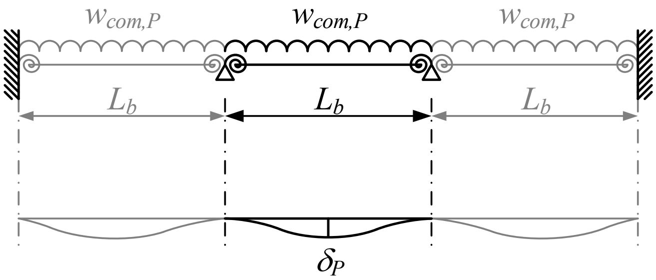
$\delta_V = 12.3$ [mm]

Deflection due to "dead loads and superimposed dead loads":

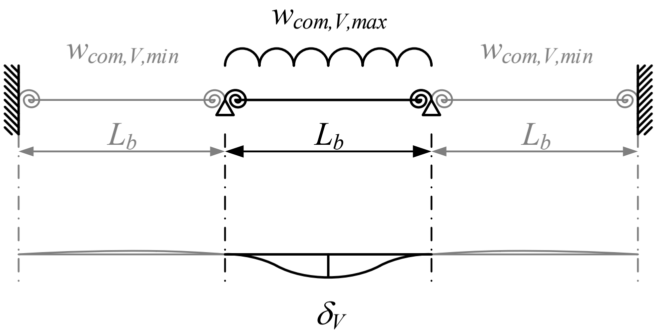
$\delta_{tP} = 42.2$ [mm]

Deflection due to "dead loads, superimposed dead loads, and live loads":

$\delta_{tP+V} = \delta_{tP} + \delta_V = 54.5$ [mm]



(a) Superimposed dead loads



(b) Live loads (Load-case 1)

Figure AIII.10: Design load with corresponding deflection

<u>Check deflection</u>

Limit of deflection due to "live loads":

$$\delta_{V,lim} = \frac{L_b}{360} = 41.7 \text{ [mm]}$$

Limit of deflection due to "dead loads, superimposed dead loads, and live loads":

$$\delta_{P+V,lim} = \frac{L_b}{250} = 60.0 \text{ [mm]}$$

$$\therefore \ \delta_V < \delta_{V,lim} \quad \text{OK} \left(\frac{\delta_V}{\delta_{V,lim}} = 0.29 \right)$$

$$\therefore \ \delta_{tP+V} < \delta_{P+V,lim} \quad \text{OK} \left(\frac{\delta_{tP+V}}{\delta_{P+V,lim}} = 0.91 \right)$$

Analysis of natural frequency

Natural frequency in composite stage can be obtained from the deflection due to "dead loads, superimposed dead loads, and 10% of live loads". Also the deflection can be calculated by the simplified analysis method described in **Appendix II**.

Design distributed load due to "dead loads, superimposed dead loads, and 10% of live loads":

$$w_{com,P+0.1V} = B_b\left(g_{k,2}+g_{k,3}+0.1q_{k,2}\right) = 22.7 \text{ [kN/m]}$$

Deflection due to "dead loads, superimposed dead loads, and 10% of live loads":

$$\delta_{P+0.1V} = 10.2 \text{ [mm]}$$

Natural frequency due to "dead loads, superimposed dead loads, and 10% of live loads":

$$f_{P+0.1V} = \frac{18}{\sqrt{\delta_{P+0.1V}}} = 5.6 \text{ [Hz]}$$

Check vibration

Required minimum natural frequency:

$$f_{req} = 4.0 \text{ [Hz]}$$

$$\therefore f_{P+0.1V} > f_{req} \quad \text{OK} \quad \left(\frac{f_{req}}{f_{P+0.1V}} = 0.71\right)$$

Control of crack width

Maximum diameter of longitudinal rebars:

$$\phi_{sl}^{\ *} = \phi_{sl,max}\left(\frac{2.9}{f_{ctm}}\right) = 14.5 \text{ [mm]}$$

Limit of stress permitted in longitudinal rebars immediately after cracking:

$$\sigma_{sl,lim} = 240 \text{ [N/mm}^2\text{]} \left(\text{for } w_k = 0.3 \text{ [mm] and } 12 \text{ [mm]} < \phi^* \leq 16 \text{ [mm]}\right)$$

< Load-case 2 for maximizing hogging moment >

Effective width in hogging moment region:

$$b_{effh} = 1229.9 \text{ [mm]}$$

Cross-sectional area of longitudinal rebars within b_{effh}:

$$A_{sl} = \pi\left(\frac{\phi_{sl,1}}{2}\right)^2 \left\lfloor\frac{b_{effh}}{p_{sl,1}}\right\rfloor + \pi\left(\frac{\phi_{sl,2}}{2}\right)^2 \min\left(\left\lfloor\frac{b_{effh}}{p_{sl,2}}\right\rfloor ; \left\lfloor\frac{b_{sl,2}}{p_{sl,2}}\right\rfloor\right) = 21.97 \text{ [cm}^2\text{]}$$

Vertical distance between centre of un-cracked concrete flange and un-cracked composite section:

$$z_0 = \frac{(A_a+A_{sl})\left(0.5D_a+D_{ps}+0.5h_{cs}\right)}{A_a+A_{sl}+\left(\frac{h_{cs}b_{effh}}{n_0}\right)} = 194.7 \text{ [mm]}$$

Coefficient taking into account of stress distribution within section immediately prior to cracking:

$$k_c = \min\left\{\frac{1}{1+\left(\frac{h_{cs}}{2z_0}\right)}+0.3\ ;\ 1.0\right\} = 1.00$$

Cross-sectional area of composite slab within b_{effh} above profiled steel sheeting:

$$A_{cs} = b_{effh}h_{cs} = 1325.4\ [\text{cm}^2]$$

Required minimum cross-sectional area of longitudinal rebars within b_{effh}:

$$A_{sl,req} = \frac{0.72k_c f_{ctm} A_{cs}}{\sigma_{sl,lim}} = 10.34\ [\text{cm}^2]$$

$$\therefore\ A_{sl} > A_{sl,req}\quad \text{OK}\ \left(\frac{A_{sl,req}}{A_{sl}} = 0.47\right)$$

Equivalent vertical distance between longitudinal rebars and neutral axis:

$$z_{sl,eq\text{-}na} = \frac{A_{sl}z_{tcs\text{-}sl,eq}+A_a\left(h_{cs}+D_{ps}+0.5D_a\right)}{A_{sl}+A_a} - z_{tcs\text{-}sl,eq} = 391.5\ [\text{mm}]$$

Second moment of area in hogging moment region:

$$I_h = I_{ay}+\frac{A_aA_{sl}\left\{D_a+2\left(D_{ps}+h_{cs}-z_{tcs\text{-}sl,eq}\right)\right\}^2}{4(A_a+A_{sl})} = 139749\ [\text{cm}^4]$$

Stress in longitudinal rebars caused by M_{Edh}:

$$\sigma_{sl,0} = \frac{M_{Edh}}{I_h}z_{sl,eq\text{-}na} = 112\ [\text{N/mm}^2]$$

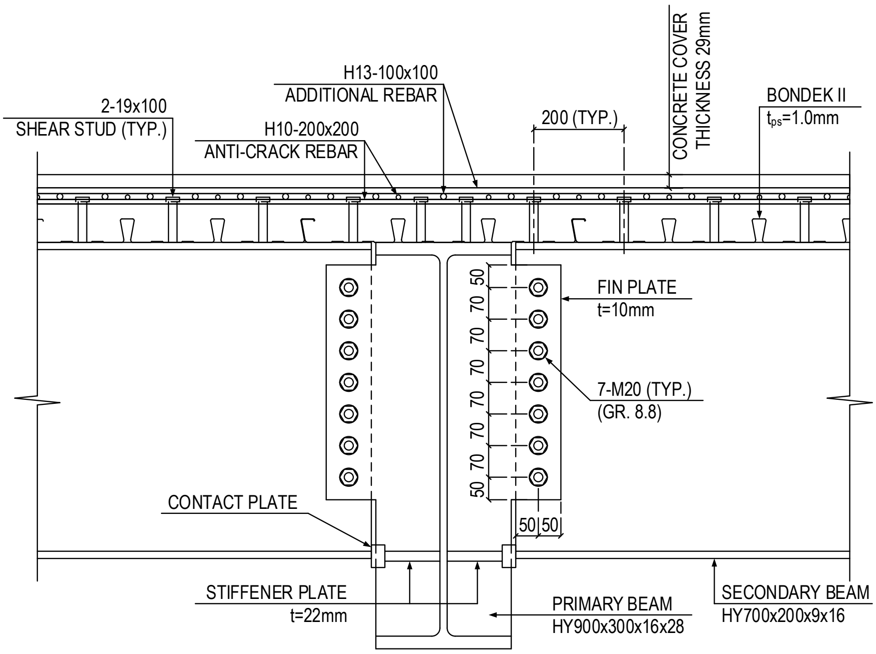
Correction of stress in longitudinal rebars for tension stiffening:

$$\Delta\sigma_{sl} = \frac{0.4f_{ctm}}{\frac{(A_a+A_{sl})I_h}{A_aI_{ay}}\left(\frac{A_{sl}}{A_{cs}}\right)} = 38\ [\text{N/mm}^2]$$

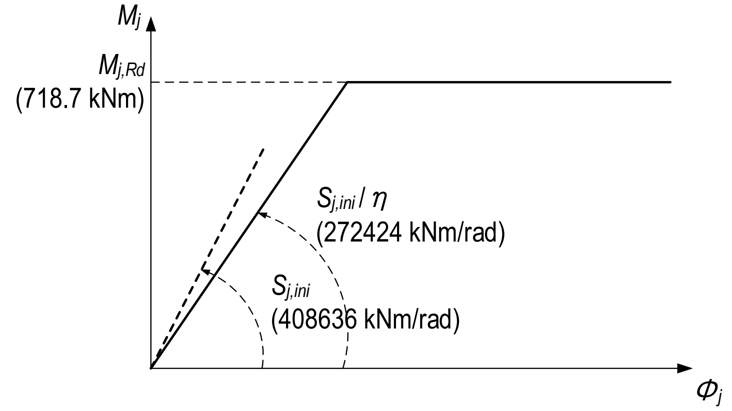
Tensile stress in longitudinal rebars due to direct loading:

$$\sigma_{sl} = \sigma_{sl,0}+\Delta\sigma_{sl} = 151\ [\text{N/mm}^2]$$

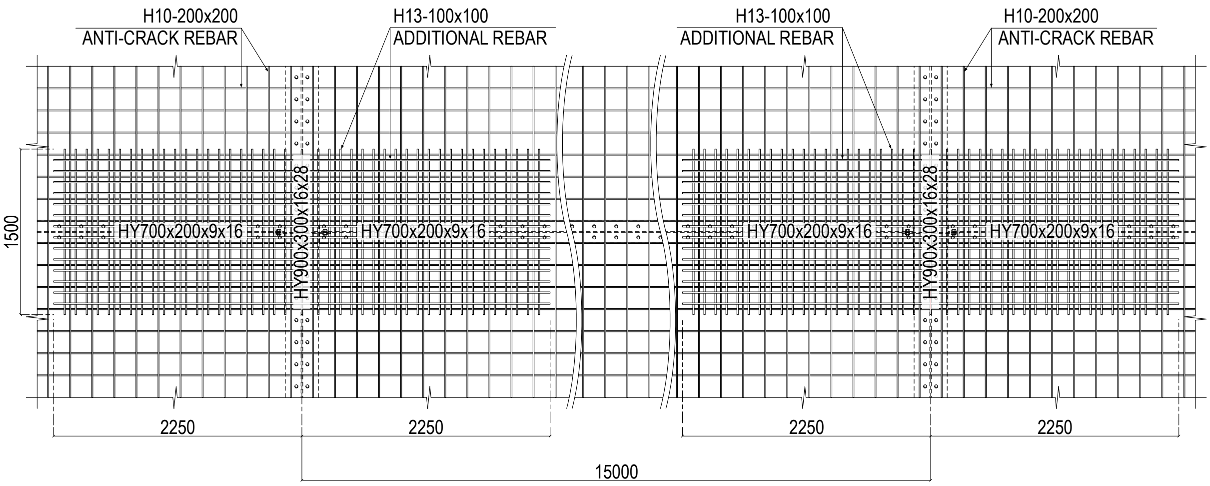
$$\therefore\ \sigma_{sl} < \sigma_{sl,lim}\quad \text{OK}\ \left(\frac{\sigma_{sl}}{\sigma_{sl,lim}} = 0.63\right)$$



(a) Joint details



(b) Joint moment-rotation characteristics (M_j-ϕ_j curves)



(c) Distribution of shear studs and arrangement of reinforcing bars in concrete slab

Figure AIII.11: Designed semi-rigid composite joint and composite beam

Appendix IV Design Example 2

This appendix presents the design example for a composite beam with beam-to-wall composite joint and pinned joint in accordance with Eurocode approach.

Figure AIV.1 shows the composite floor plan in this design example. All the beams are composite beams and the designed beam is a composite beam supported by contact-type beam-to-wall composite joint and pinned joint. This composite beam is subjected to only the gravity load, and its design moment and deflection are analysed by the elastic-plastic global analysis in which the joints are allowed to behave as plastic hinges in inelastic region. Therefore, the cross-section of the composite beam should be at least in Class 2, and the joint details of the beam-to-wall composite joint with bent reinforcing bars should comply with the pre-qualified specification shown in subsection 4.2.1.

In this design example, structural resistance checks at ultimate limit state and serviceability checks are carried out. As with the serviceability checks in design example 1, the limit of the deflection due to variable actions is $L_b/360$, the limit of the deflection due to permanent and variable actions is $L_b/250$, and the minimum natural frequency is taken as 4 [Hz]. Besides, the limit of the crack width recommended in Eurocode 2, is taken as 0.3 [mm], for the other serviceability criteria.

Other design conditions such as the sequence of construction (un-propped or propped), the direction of the ribs of the profiled steel sheeting, and the uniformity of steel section and floor slab are same as those in design example 1 in **Appendix III**.

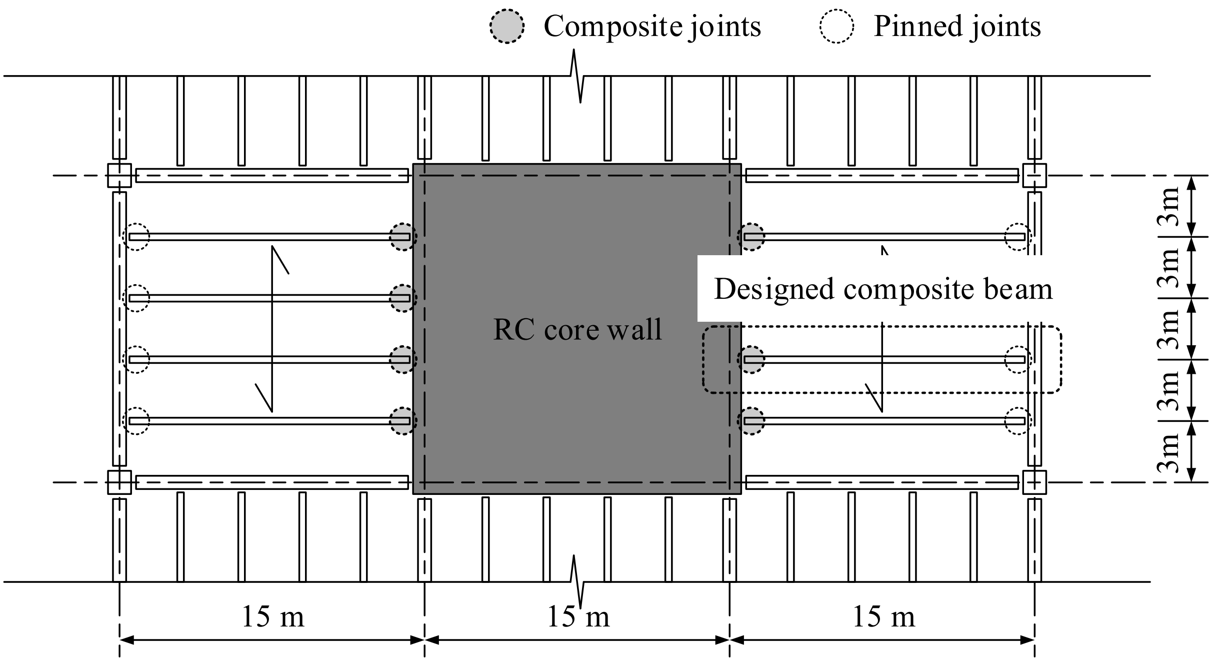


Figure AIV.1: Composite floor plan

Design Conditions

[Span and spacing]

Beam span: L_b = 15.0 [m]
Beam spacing: B_b = 3.0 [m]

[Steel beam]

Cross-section: H700x200x9x16 (JIS cross-section)
Mass per metre: g_a = 99.6 [kg/m]
Depth: D_a = 700 [mm]
Width: B_a = 200 [mm]
Web thickness: t_w = 9 [mm]
Flange thickness: t_f = 16 [mm]
Root radius: r = 18 [mm]
Cross-sectional area: A_a = 126.9 [cm^2]
Second moment of area about major axis (y-y axis): I_{ay} = 100255 [cm^4]
Second moment of area about minor axis (z-z axis): I_{az} = 2140 [cm^4]
Plastic section modulus: $W_{pl,a}$ = 3285 [cm^3]
Warping constant: $I_{w,a}$ = 2.50 [dm^6]
Torsion constant: $I_{T,a}$ = 81.8 [dm^4]
Steel grade: S355
Nominal value of yield strength: f_{ay} = 355 [N/mm^2] (for t_f = 16 [mm])
Ultimate tensile strength: f_u = 470 [N/mm^2] (for t_f = 16 [mm])
Modulus of elasticity: E_a = 210000 [N/mm^2]
Partial factor of resistance of members and cross-sections: γ_a = 1.00
Partial factor of resistance of plates in bearing: $\gamma_{a,2}$ = 1.25
Design yield strength: $f_{ayd} = f_{ay}/\gamma_a$ = 355 [N/mm^2]

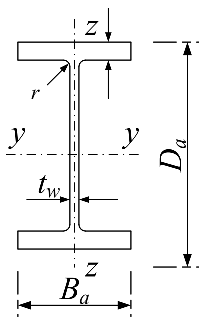


Figure AIV.2: Cross-section of steel beam

[Profiled steel sheeting]

Steel sheeting type: BONDEK 1.0
Mass per metre: g_{ps} = 13.79 [kg/m]
Overall depth: D_{ps} = 51 [mm]
Pitch of ribs: p_{ps} = 200 [mm]
Minimum width for re-entrant: $b_{0,min}$ = 168 [mm]

Maximum width for re-entrant: $b_{0,max}$ = 187 [mm]
Thickness: t_{ps} = 1.00 [mm]
Characteristic yield strength: f_{psk} = 550 [N/mm^2]
Partial factor: γ_{ps} = 1.00
Design yield strength: $f_{psd} = f_{psk}/\gamma_{ps}$ = 550 [N/mm^2]

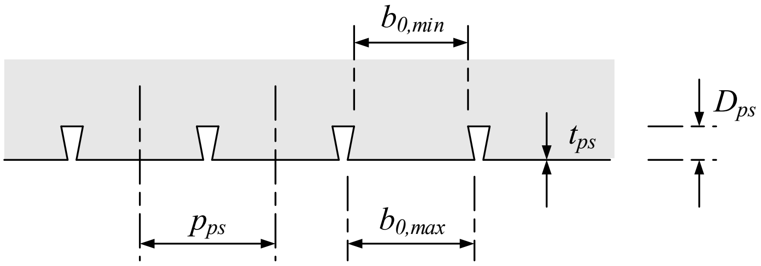


Figure AIV.3: Cross-section of profiled steel sheeting

[Concrete slab]

Overall depth: D_{cs} = 150 [mm]
Thickness above profiled steel sheeting: $h_{cs} = D_{cs}$-D_{ps} = 99 [mm]
Strength class of concrete: C30/37
Characteristic cylinder strength: $f_{ck,cs}$ = 30.0 [N/mm^2]
Mean value of tensile strength: f_{ctm} = 2.9 [N/mm^2]
Secant modulus of elasticity: $E_{cm,cs}$ = 33000 [N/mm^2]
Partial factor: $\gamma_{c,cs}$ = 1.50
Design strength: $f_{cd,cs} = f_{ck,cs} / \gamma_{c,cs}$ = 20.0 [N/mm^2]
Dry density: ρ_c = 2400 [kg/m^3]

[Reinforcing bar]

Diameter of anti-crack longitudinal rebars (row 1): $\phi_{sl,1}$ = 10 [mm]
Diameter of anti-crack transverse rebars (row 1): $\phi_{st,1}$ = 10 [mm]
Diameter of additional longitudinal rebars at semi-rigid end (row 2): $\phi_{sl,2}$ = 13 [mm]
Pitch of anti-crack longitudinal rebars (row 1): $p_{sl,1}$ = 200 [mm]
Pitch of anti-crack transverse rebars (row 1): $p_{st,1}$ = 200 [mm]
Pitch of additional longitudinal rebars at semi-rigid end (row 2): $p_{sl,2}$ = 100 [mm]
Covering depth of anti-crack longitudinal rebars (row 1): $z_{tcs\text{-}csl,1}$ = 35 [mm]
Covering depth of anti-crack transverse rebars (row 1): $z_{tcs\text{-}cst,1}$ = 45 [mm]
Covering depth of additional longitudinal rebars at semi-rigid end (row 2):
$z_{tcs\text{-}csl,2}$ = 57 [mm]
Arrangement width of additional longitudinal rebars at semi-rigid end (row 2):
$b_{sl,2}$ = 1500 [mm]
Projected anchorage length from surface of RC core wall: l_{dh} = 350 [mm]
Strength class: B500C
Characteristic yield strength: f_{sk} = 500 [N/mm^2]
Modulus of elasticity: E_s = 210000 [N/mm^2]

Partial factor: $\gamma_s = 1.15$
Design yield strength: $f_{sd} = f_{sk}/\gamma_s = 435$ [N/mm^2]

[Headed stud]

Diameter of shank: $d_{hs} = 19$ [mm]
Overall height: $h_{hs} = 100$ [mm]
Ultimate strength: $f_{hsu} = 450$ [N/mm^2]
Partial factor: $\gamma_V = 1.25$
Number per sheeting rib in sagging moment region: $n_{hss} = 2$
Number per sheeting rib in hogging moment region: $n_{hsh} = 2$
Distance between centres of outstand headed studs in sagging moment region:
$b_{0s} = 100$ [mm]
Distance between centres of outstand headed studs in hogging moment region:
$b_{0h} = 100$ [mm]
Distance between surface of RC core wall and first headed stud: $h_{cw\text{-}fhs} = 50$ [mm]
Distance between centre of joint and first headed stud at semi-rigid end: $h_{cj\text{-}fhs} = 250$ [mm]
Distance between surface of primary beam and first headed stud: $h_{pb\text{-}fhs} = 50$ [mm]
Distance between centre of joint and first headed stud at pinned end: $h_{cj\text{-}fhs} = 200$ [mm]

[Reinforced concrete core wall]

Effective width (Beam spacing): $B_{cw} = 3.0$ [m]
Effective height (Floor height): $H_{cw} = 4000$ [mm]
Thickness: $t_{cw} = 400$ [mm]
Strength class of concrete: C50/60
Characteristic cylinder strength: $f_{ck,cw} = 50.0$ [N/mm^2]
Secant modulus of elasticity: $E_{cm,cw} = 37000$ [N/mm^2]
Partial factor: $\gamma_{c,cw} = 1.50$
Design strength: $f_{cd,cw} = f_{ck,cw}/\gamma_{c,cw} = 33.3$ [N/mm^2]
End plate depth: $D_{ep} = 750$ [mm]
End plate width: $B_{ep} = 300$ [mm]
End plate thickness: $t_{ep} = 22$ [mm]
Steel grade: S355
Nominal value of yield strength: $f_{epy} = 345$ [N/mm^2] (for $t_{ep} = 22$ [mm])
Partial factor of resistance of members and cross-sections: $\gamma_{ep} = 1.00$

[Fin plate]

Depth: $D_{fp} = 450$ [mm]
Thickness: $t_{fp} = 10$ [mm]
Radius of gyration of area about minor axis (z-z axis): $i_{fpz} = 2.89$ [mm]
Leg length of fillet weld: $s_{fp} = 10$ [mm]
Steel grade: S355
Nominal value of yield strength: $f_{fpy} = 355$ [N/mm^2] (for $t_{fp} = 10$ [mm])
Ultimate tensile strength: $f_{fpu} = 470$ [N/mm^2] (for $t_{fp} = 10$ [mm])

Modulus of elasticity: E_{fp} = 210000 [N/mm^2]
Partial factor of resistance of members and cross-sections: γ_{fp} = 1.00
Partial factor of resistance of plates in bearing: $\gamma_{fp,2}$ = 1.25

[Contact plate]

Nominal value of yield strength: f_{cpy} = 345 [N/mm^2]
Partial factor of resistance of members and cross-sections: γ_{cp} = 1.00
Partial factor of resistance of plates in bearing: $\gamma_{cp,2}$ = 1.25
Design yield strength: $f_{cpyd} = f_{cpy}/\gamma_{cp}$ = 345 [N/mm^2]

[Bolt]

Size: M20 (d_b = 20 [mm])
Tensile stress area: A_b = 2.45 [cm^2]
Hole diameter: d_0 = 22 [mm]
Strength class: 8.8
Ultimate tensile strength: f_{bu} = 800 [N/mm^2]
Partial factor: γ_b = 1.25
Number on vertical line: $n_{b,v}$ = 6
Number on horizontal line: $n_{b,h}$ = 1
Pitch on vertical line: $p_{b,v}$ = 70 [mm]
Pitch on horizontal line: $p_{b,h}$ = 60 [mm]
Edge distance for fin plate on vertical line: $e_{b\text{-}fp,v}$ = 50 [mm]
Edge distance for fin plate on horizontal line: $e_{b\text{-}fp,h}$ = 50 [mm]
Edge distance for web of steel beam on vertical line: $e_{b\text{-}bw,v}$ = 90 [mm]
Edge distance for web of steel beam on horizontal line: $e_{b\text{-}bw,h}$ = 50 [mm]

Design Loads

[Permanent actions (dead loads and superimposed dead loads)]

Area per unit length of concrete slab:

$$A_c = 1000h_{cs} + \frac{(b_{0,min}+b_{0,max})D_{ps}}{2}\frac{1000}{p_{ps}} = 1443 \text{ [cm}^2\text{/m]}$$

Weight per unit area of concrete slab and rebars:

$$A_c\left(\frac{\rho_c}{100}+2\right) = 3.75 \text{ [kN/m}^2\text{]} \quad \text{(wet concrete)}$$

$$A_c\left(\frac{\rho_c}{100}+1\right) = 3.61 \text{ [kN/m}^2\text{]} \quad \text{(dry concrete)}$$

Weight per unit area of profiled steel sheeting:

$$9.8g_{ps} = 0.14 \text{ [kN/m}^2\text{]}$$

Weight per unit area of steel beam:

$$\frac{9.8g_a}{B_b} = 0.33 \text{ [kN/m}^2\text{]}$$

Dead load per unit area in construction stage:
$g_{k,1}$ = 3.75+0.14+0.33 = 4.21 [kN/m^2]

Dead load per unit area in composite stage:
$g_{k,2}$ = 3.61+0.14+0.33 = 4.07 [kN/m^2]

Superimposed dead load per unit area in composite stage:
$g_{k,3}$ = 2.00 [kN/m^2]

[Variable actions (live loads)]

Construction load per unit area in construction stage:
$q_{k,1}$ = 0.50 [kN/m^2]

Imposed floor load per unit area in composite stage:
$q_{k,2}$ = 6.00 [kN/m^2]

[Partial factors]

Partial factor for permanent actions (unfavourable): $\gamma_{G,sup}$ = 1.35
Partial factor for permanent actions (favourable): $\gamma_{G,inf}$ = 1.00
Partial factor for variable actions (unfavourable): γ_Q = 1.50
Partial factor for variable actions (favourable): γ_{Qi} = 0.00

Design of Semi-rigid Composite Joint

[Verifications of joint classification]

Check initial rotational stiffness and moment resistance

Vertical distance between centre of longitudinal rebars and centre of contact part for row 1:

$$z_{csl,1\text{-}cc} = D_a + D_{cs} - z_{tcs\text{-}csl,1} - \frac{t_f}{2} = 807 \text{ [mm]}$$

Vertical distance between centre of longitudinal rebars and centre of contact part for row 2:

$$z_{csl,2\text{-}cc} = D_a + D_{cs} - z_{tcs\text{-}csl,2} - \frac{t_f}{2} = 786 \text{ [mm]}$$

Effective width:

$$b_{eff,j} = b_{0h} + \min\left\{\frac{2(0.15L_b)}{4}\ ;\ B_b - b_{0h}\right\} = 1225.0 \text{ [mm]}$$

Cross-sectional area of longitudinal rebars within $b_{eff,j}$ for row 1:

$$A_{sl,1} = \pi\left(\frac{\phi_{sl,1}}{2}\right)^2 \left\lfloor\frac{b_{eff,j}}{p_{sl,1}}\right\rfloor = 4.7 \text{ [cm}^2\text{]}$$

Cross-sectional area of longitudinal rebars within $b_{eff,j}$ for row 2:

$$A_{sl,2} = \pi\left(\frac{\phi_{sl,2}}{2}\right)^2 \left\lfloor\frac{b_{eff,j}}{p_{sl,2}}\right\rfloor = 15.9 \text{ [cm}^2\text{]}$$

Effective length for row 1:

$$l_{eff,1} = \min\left(\frac{l_{dh,1}+h_{cw\text{-}fhs}}{2}\ ;\ 20\phi_{sl,1}\right) = 200\ [\text{mm}]$$

Effective length for row 2:

$$l_{eff,2} = \min\left(\frac{l_{dh,2}+h_{cw\text{-}fhs}}{2}\ ;\ 20\phi_{sl,2}\right) = 200\ [\text{mm}]$$

Equivalent stiffness coefficient of longitudinal rebars:

$$k_{sl,eq} = \frac{A_{sl,1}}{l_{eff,1}}+\frac{A_{sl,2}}{l_{eff,2}} = 10.32\ [\text{mm}]$$

Length of composite beam in hogging moment region adjacent to joint:

$$l = 0.15L_b = 2250\ [\text{mm}]$$

Number of headed studs distributed over length l:

$$N = \left\lceil\frac{\left(l\text{-}h_{cj\text{-}fhs}\right)}{p_{ps}}\right\rceil n_{hsh} = 20$$

Stiffness of one headed stud with 19 [mm] diameter of shank:

$$k_{sc} = 100\ [\text{kN/mm}]$$

Equivalent vertical distance between longitudinal rebars and centre of contact part:

$$z_{sl,eq\text{-}cc} = D_a+D_{cs}\text{-}z_{tcs\text{-}sl,eq}\text{-}\frac{t_f}{2} = 790\ [\text{mm}]$$

Equivalent vertical distance between longitudinal rebars and centre of steel beam:

$$z_{sl,eq\text{-}ca} = \frac{D_a}{2}+D_{cs}\text{-}z_{tcs\text{-}sl,eq} = 448\ [\text{mm}]$$

Parameter related to deformation of headed studs:

$$\xi = \frac{E_a I_{ay}}{z_{sl,eq\text{-}ca}{}^2 E_s\left(A_{sl,1}+A_{sl,2}\right)} = 2.42$$

$$\nu = \sqrt{\frac{(1+\xi)Nk_{sc}l z_{sl,eq\text{-}ca}{}^2}{E_a I_{ay}}} = 3.83$$

Stiffness related to headed studs:

$$K_{sc} = \frac{Nk_{sc}}{\nu\text{-}\left(\frac{\nu\text{-}1}{1+\xi}\right)\left(\frac{z_{sl,eq\text{-}cc}}{z_{sl,eq\text{-}ca}}\right)} = 843798\ [\text{N/mm}]$$

Stiffness reduction factor due to deformation of headed studs:

$$k_{slip} = \frac{1}{1+\left(\frac{E_s k_{sl,eq}}{K_{sc}}\right)} = 0.28$$

Joint material coefficient of RC core wall:

$$\beta_j = 0.67$$

Amplification factor from loaded area to maximum design distribution area:

$\alpha = 2.52$ (determined by convergence calculation)

Design bearing strength of concrete for RC core wall:

$f_{jd,cw} = \beta_j \alpha f_{cd,cw} = 56.0$ [N/mm^2]

Additional bearing width of equivalent T-stub flange in compression:

$$c = t_f \sqrt{\frac{f_{epy}}{3 f_{jd,cw} \gamma_{ep}}} = 31.5 \text{ [mm]}$$

Effective depth of equivalent T-stub flange in compression:

$d_{eff} = t_f + c + \min\left(D_{ep} - D_a \ ; \ c\right) = 79.0$ [mm]

Effective width of equivalent T-stub flange in compression:

$b_{eff} = \min\left(B_a + 2c \ ; \ B_{ep}\right) = 263.0$ [mm]

Stiffness coefficient of concrete for RC core wall:

$$k_{13,cw} = \frac{E_{cm,cw}\sqrt{d_{eff} b_{eff}}}{1.275 E_a} = 19.93 \text{ [mm]}$$

Initial rotational stiffness:

$$S_{j,ini} = E_s \frac{k_{slip} k_{sl,eq} k_{13,cw}}{\left(k_{slip} k_{sl,eq} + k_{13,cw}\right)} z_{sl,eq\text{-}cc}^{\ 2} = 331340 \text{ [kNm/rad]}$$

Effective width of composite beam assuming simply supported condition:

$$b_{eff,b} = b_{0s} + \min\left(\frac{L_b}{4} \ ; \ B_b - b_{0s}\right) = 3000.0 \text{ [mm]}$$

Second moment of area of composite beam assuming simply supported condition:

$$I_b = \frac{A_a\left(h_{cs} + 2D_{ps} + D_a\right)^2}{4\left(1 + \frac{2E_a}{E_{cm,cs}} \frac{A_a}{b_{eff,b} h_{cs}}\right)} + \frac{b_{eff,b} h_{cs}^{\ 3}}{12\left(\frac{2E_a}{E_{cm,cs}}\right)} + I_{ay} = 268987 \text{ [cm}^4\text{]}$$

Upper boundary of rotational stiffness for nominally pinned joint:

$$\frac{0.5 E_a I_b}{L_b} = 18829 \text{ [kNm/rad]}$$

Requirement to calculate initial rotational stiffness:

$t_{cw} \geq \max\left\{(\alpha - 1) d_{eff} \ ; \ (\alpha - 1) b_{eff}\right\} \ \rightarrow$ can be calculated

$$\therefore \ S_{j,ini} > \frac{0.5 E_a I_b}{L_b} \quad \text{OK} \left(\frac{\left(\frac{0.5 E_a I_b}{L_b}\right)}{S_{j,ini}} = 0.06\right)$$

Cross-sectional area of longitudinal rebars within $b_{eff,j}$:

$A_{sl,j} = A_{sl,1} + A_{sl,2} = 20.6$ [cm^2]

Tension resistance of longitudinal rebars within $b_{eff,j}$:

$R_{sl,j} = A_{sl,j} f_{sd} = 897.4$ [kN]

Cross-sectional area of bottom flange of steel beam:

$A_{bf} = B_a t_f = 32.0$ [cm^2]

Cross-sectional area of contact plate (depends on contact plate detailing):

$A_{cp} = 144.0$ [cm^2]

Bearing area of contact plate (depends on contact plate detailing):

$A_{bea} = 25.6$ [cm^2]

Compression resistance of contact part:

$$R_{con} = \min\left\{A_{bf} f_{ayd}\ ;\ A_{cp} f_{cpyd}\ ;\ 1.5 A_{bea} \min\left(\frac{f_{ay}}{\gamma_{a,2}}\ ;\ \frac{f_{cpy}}{\gamma_{cp,2}}\right)\right\} = 1059.8 \text{ [kN]}$$

Moment resistance:

$$M_{j,Rd} = z_{sl,eq\text{-}cc} \min\left(R_{sl,j}\ ;\ R_{con}\right) = 709.3 \text{ [kNm]}$$

Plastic moment resistance of composite beam (This value will be calculated in the design of composite beam):

$M_{pl,Rd} = 1263.7$ [kNm]

Upper boundary of moment resistance for nominally pinned joint:

$0.25 M_{pl,Rd} = 315.9$ [kNm]

$$\therefore\ M_{j,Rd} > 0.25 M_{pl,Rd} \quad \text{OK} \quad \left(\frac{0.25 M_{pl,Rd}}{M_{j,Rd}} = 0.45\right)$$

Check compression resistance

Compression resistance of equivalent T-stub flange:

$F_{C,Rd} = f_{jd,cw} d_{eff} b_{eff} = 1164.7$ [kNm]

$$\therefore\ F_{C,Rd} > R_{con} \quad \text{OK} \quad \left(\frac{R_{con}}{F_{C,Rd}} = 0.91\right)$$

Check anchorage strength and panel shear resistance

The conditions to use the pre-qualified specifications are satisfied and the joint details comply with the pre-qualified specification in Section 4.2.

∴ Joint details → Pre-qualified specification OK

[Verifications of structural resistance in composite stage]

Check bolt group resistance

Correction factor for bolt shear resistance:

$\alpha_{bV} = 0.60$ (for strength class 8.8)

Shear resistance of a single bolt:

$$F_{bV,Rd} = \frac{\alpha_{bV} f_{bu} A_b}{\gamma_b} = 94.1 \text{ [kN]}$$

Distance between face of support and assumed line of shear transfer:

$z_{fs\text{-}b} = 60$ [mm]

α factor:

$\alpha = 0.00 \left(\text{for } n_{b,h} = 1\right)$

β factor:

$$\beta = \frac{6z_{fs\text{-}b}}{n_{b,v}\left(n_{b,v}+1\right)p_{b,v}} = 0.12 \left(\text{for } n_{b,h} = 1\right)$$

Bolt shear resistance:

$$V_{b,Rd} = \frac{n_{b,v}n_{b,h}F_{bV,Rd}}{\sqrt{\left(1+\alpha n_{b,v}n_{b,h}\right)^2+\left(\beta n_{b,v}n_{b,h}\right)^2}} = 454.9 \text{ [kN]}$$

Design shear force (This value will be calculated in the design of composite beam):

$V_{Ed} = 427.3$ [kN]

$$\therefore\ V_{b,Rd} > V_{Ed} \quad \text{OK} \left(\frac{V_{Ed}}{V_{b,Rd}} = 0.95\right)$$

< In fin plate >

k_1 factor for vertical bolt bearing resistance:

$$k_{1,vbb} = \min\left(2.8\frac{e_{b\text{-}fp,h}}{d_0}\text{-}1.7\ ;\ 2.5\right) = 2.50 \left(\text{for } n_{b,h} = 1\right)$$

Correction factor for vertical bolt bearing resistance:

$$\alpha_{vbb} = \min\left(\frac{e_{b\text{-}fp,v}}{3d_0}\ ;\ \frac{p_{b,v}}{3d_0}\text{-}\frac{1}{4}\ ;\ \frac{f_{bu}}{f_{fpu}}\ ;\ 1.0\right) = 0.76$$

Vertical bearing resistance of a single bolt:

$$F_{vbb,Rd} = \frac{k_{1,vbb}\,\alpha_{vbb}f_{fpu}d_bt_{fp}}{\gamma_b} = 142.4 \text{ [kN]}$$

k_1 factor for horizontal bolt bearing resistance:

$$k_{1,hbb} = \min\left(2.8\frac{e_{b\text{-}fp,v}}{d_0}\text{-}1.7\ ;\ 1.4\frac{p_{b,v}}{d_0}\text{-}1.7\ ;\ 2.5\right) = 2.50$$

Correction factor for horizontal bolt bearing resistance:

$$\alpha_{hbb} = \min\left(\frac{e_{b\text{-}fp,h}}{3d_0}\ ;\ \frac{f_{bu}}{f_{fpu}}\ ;\ 1.0\right) = 0.76 \left(\text{for } n_{b,h} = 1\right)$$

Horizontal bearing resistance of a single bolt:

$$F_{hbb,Rd} = \frac{k_{1,hbb}\,\alpha_{hbb}f_{fpu}d_bt_{fp}}{\gamma_b} = 142.4 \text{ [kN]}$$

Bolt bearing resistance:

$$V_{bb,Rd} = \frac{n_{b,v} n_{b,h}}{\sqrt{\left(\frac{1+\alpha n_{b,v} n_{b,h}}{F_{vbb,Rd}}\right)^2 + \left(\frac{\beta n_{b,v} n_{b,h}}{F_{hbb,Rd}}\right)^2}} = 688.7 \text{ [kN]}$$

< In web of steel beam >

Nominal value of yield strength of web of steel beam:

$f_{wy} = 355$ [N/mm^2] (for $t_w = 9$ [mm])

Ultimate tensile strength of web of steel beam:

$f_{wu} = 470$ [N/mm^2] (for $t_w = 9$ [mm])

k_1 factor for vertical bolt bearing resistance:

$$k_{1,vbb} = \min\left(2.8\frac{e_{b\text{-}bw,h}}{d_0} - 1.7\ ;\ 2.5\right) = 2.50\ \left(\text{for } n_{b,h} = 1\right)$$

Correction factor for vertical bolt bearing resistance:

$$\alpha_{vbb} = \min\left(\frac{e_{b\text{-}bw,v}}{3d_0}\ ;\ \frac{p_{b,v}}{3d_0} - \frac{1}{4}\ ;\ \frac{f_{bu}}{f_{wu}}\ ;\ 1.0\right) = 0.81$$

Vertical bearing resistance of a single bolt:

$$F_{vbb,Rd} = \frac{k_{1,vbb}\,\alpha_{vbb} f_{wu} d_b t_w}{\gamma_b} = 137.2 \text{ [kN]}$$

k_1 factor for horizontal bolt bearing resistance:

$$k_{1,hbb} = \min\left(2.8\frac{e_{b\text{-}bw,v}}{d_0} - 1.7\ ;\ 1.4\frac{p_{b,v}}{d_0} - 1.7\ ;\ 2.5\right) = 2.50$$

Correction factor for horizontal bolt bearing resistance:

$$\alpha_{hbb} = \min\left(\frac{e_{b\text{-}bw,h}}{3d_0}\ ;\ \frac{f_{bu}}{f_{wu}}\ ;\ 1.0\right) = 0.76\ \left(\text{for } n_{b,h} = 1\right)$$

Horizontal bearing resistance of a single bolt:

$$F_{hbb,Rd} = \frac{k_{1,hbb}\,\alpha_{hbb} f_{wu} d_b t_w}{\gamma_b} = 128.2 \text{ [kN]}$$

Bolt bearing resistance:

$$V_{bb,Rd} = \frac{n_{b,v} n_{b,h}}{\sqrt{\left(\frac{1+\alpha n_{b,v} n_{b,h}}{F_{vbb,Rd}}\right)^2 + \left(\frac{\beta n_{b,v} n_{b,h}}{F_{hbb,Rd}}\right)^2}} = 647.0 \text{ [kN]}$$

$$\therefore\ V_{bb,Rd} > V_{Ed}\quad \text{OK}\ \left(\frac{V_{Ed}}{V_{bb,Rd}} = 0.67\right)$$

Check fin plate resistance

Shear resistance for gross section:

$$V_{fp,Rd,g} = \frac{D_{fp} t_{fp}}{1.27} \frac{f_{fpy}}{\sqrt{3}\gamma_{fp}} = 726.2 \text{ [kN]}$$

Shear area for net section:

$$A_{fpV,n} = t_{fp}\left(D_{fp}\text{-}n_{b,v}d_0\right) = 31.8 \text{ [cm}^2\text{]}$$

Shear resistance for net section:

$$V_{fp,Rd,n} = A_{fpV,n}\frac{f_{fpu}}{\sqrt{3}\gamma_{fp,2}} = 690.3 \text{ [kN]}$$

Net area subjected to tension:

$$A_{fp,nt} = t_{fp}\left(e_{b\text{-}fp,h}\text{-}\frac{d_0}{2}\right) = 3.9 \text{ [cm}^2\text{]} \left(\text{for } n_{b,h} = 1\right)$$

Net area subjected to shear:

$$A_{fp,nV} = t_{fp}\left\{D_{fp}\text{-}e_{b\text{-}fp,v}\text{-}\left(n_{b,v}\text{-}0.5\right)d_0\right\} = 27.9 \text{ [cm}^2\text{]}$$

Block shear resistance:

$$V_{fp,Rd,b} = \frac{0.5 f_{fpu} A_{fp,nt}}{\gamma_{fp,2}} + \frac{f_{fpy} A_{fp,nV}}{\sqrt{3}\gamma_{fp}} = 645.2 \text{ [kN]}$$

$$\therefore \min\left(V_{fp,Rd,g} \,;\, V_{fp,Rd,n} \,;\, V_{fp,Rd,b}\right) > V_{Ed} \quad \text{OK} \quad \left(\frac{V_{Ed}}{\min\left(V_{fp,Rd,g} \,;\, V_{fp,Rd,n} \,;\, V_{fp,Rd,b}\right)} = 0.67\right)$$

Elastic moment resistance:

$$M_{el,fp,Rd} = \frac{t_{fp} {D_{fp}}^2}{6}\frac{f_{fpy}}{\gamma_{fp}} = 119.8 \text{ [kNm]}$$

$D_{fp} > 2.73 z_{fs\text{-}b}$ → no need to check

Fin plate type:

$$e_{b\text{-}bw,h}+10 > \frac{t_{fp}}{0.15} \quad \rightarrow \text{short fin plate}$$

Non-dimensional slenderness for lateral torsional buckling:

$$\lambda_{LT,fp} = \frac{e_{b\text{-}bw,h}+10}{\pi i_{fpz}}\sqrt{\frac{f_{fpy}}{E_{fp}}} = 0.27$$

Value to determine reduction factor for lateral-torsional buckling:

$$\Phi_{LT,fp} = 0.5\left\{1+0.49\left(\lambda_{LT,fp}\text{-}0.2\right)+{\lambda_{LT,fp}}^2\right\} = 0.55$$

Reduction factor for lateral-torsional buckling:

$$\chi_{LT,fp} = \min\left(\frac{1}{\Phi_{LT,fp}+\sqrt{{\Phi_{LT,fp}}^2\text{-}{\lambda_{LT,fp}}^2}} \,;\, 1.0\right) = 0.96$$

Lateral torsional buckling moment resistance:

$M_{LT,fp,Rd} = M_{el,fp,Rd} = 119.8$ [kNm] (for short fin plate)

$$\therefore\ M_{LT,fp,Rd} > V_{Ed} z_{fs\text{-}b} \quad \text{OK} \ \left(\frac{V_{Ed} z_{fs\text{-}b}}{M_{LT,fp,Rd}} = 0.22 \right)$$

Check resistance of web of steel beam

Shear area for gross section:

$A_{bwV,g} = A_a\text{-}2B_a t_f + (t_w + 2r) t_f = 70.1$ [cm^2]

Shear resistance for gross section:

$$V_{bw,Rd,g} = A_{bwV,g} \frac{f_{wy}}{\sqrt{3}\gamma_a} = 1436.8 \text{ [kN]}$$

Shear area for net section:

$A_{bwV,n} = A_{bwV,g}\text{-}n_{b,v} d_0 t_w = 58.2$ [cm^2]

Shear resistance for net section:

$$V_{bw,Rd,n} = A_{bwV,n} \frac{f_{wu}}{\sqrt{3}\gamma_{a,2}} = 1263.9 \text{ [kN]}$$

$$\therefore\ \min\left(V_{bw,Rd,g}\ ;\ V_{bw,Rd,n}\right) > V_{Ed} \quad \text{OK} \ \left(\frac{V_{Ed}}{\min\left(V_{bw,Rd,g}\ ;\ V_{bw,Rd,n}\right)} = 0.34 \right)$$

Elastic moment resistance on vertical line of bolts:

$$M_{el,vbw,Rd} = \frac{t_w \left\{ \left(n_{b,v}\text{-}1\right) p_{b,v} \right\}^2}{6} \frac{f_{wy}}{\gamma_a} = 65.2 \text{ [kNm]}$$

Plastic shear resistance on top and bottom horizontal line of bolts:

$$V_{pl,hbw,Rd} = t_w e_{b\text{-}bw,h} \frac{f_{wy}}{\sqrt{3}\gamma_a} = 92.2 \text{ [kN]} \ \left(\text{for } n_{b,h} = 1\right)$$

Design shear force on vertical line of bolts:

$$V_{vbw,Ed} = V_{Ed} \frac{\left(n_{b,v}\text{-}1\right) p_{b,v}}{D_a} = 217.0 \text{ [kN]}$$

Plastic shear resistance on vertical line of bolts:

$$V_{pl,vbw,Rd} = t_w \left(n_{b,v}\text{-}1\right) p_{b,v} \frac{f_{wy}}{\sqrt{3}\gamma_a} = 645.6 \text{ [kN]}$$

Reduced moment resistance on vertical line of bolts making allowance for presence of shear force:

$$M_{y,v,vbw,Rd} = M_{el,vbw,Rd} = 65.2 \text{ [kNm]} \ \left(\text{for } V_{vbw,Ed} \le \frac{V_{pl,vbw,Rd}}{2} \right)$$

$$\therefore \min\left(M_{el,vbw,Rd}\,;\,M_{y,v,vbw,Rd}\right)+V_{pl,hbw,Rd}\left(n_{b,v}\text{-}1\right)p_{b,v} > V_{Ed}z_{fs\text{-}b} \quad \text{OK}$$

$$\left(\frac{V_{Ed}z_{fs\text{-}b}}{\min\left(M_{el,vbw,Rd}\,;\,M_{y,v,vbw,Rd}\right)+V_{pl,hbw,Rd}\left(n_{b,v}\text{-}1\right)p_{b,v}} = 0.27\right)$$

Check fillet weld of fin plate

Effective throat thickness of fillet weld:

$a_{fp} = 0.7s_{fp} = 7.0$ [mm]

Required minimum throat thickness of fillet weld:

$a_{fp,req} = 0.6t_{fp} = 6.0$ [mm] ((for S355 steel grade))

$$\therefore a_{fp} > a_{fp,req} \quad \text{OK} \quad \left(\frac{a_{fp,req}}{a_{fp}} = 0.86\right)$$

[Verification of serviceability in composite stage]

Analysis of design moment

Design moment in composite stage can be calculated by the equations in Section 5.3. The equivalent initial rotational stiffness $S_{j,ini,eq}$ incorporating the flexural rigidity of the core wall is used as the rotational stiffness S_j in structural analysis.

Maximum design distributed load:

$w_{com,max} = B_b\left(g_{k,3}+q_{k,2}\right) = 24.0$ [kN/m]

Design hogging moment:

$M_{Edh} = 412.0$ [kNm]

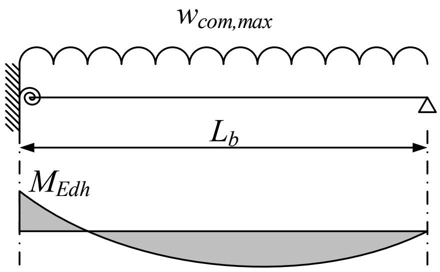


Figure AIV.4: Design load with corresponding moment

Control of crack width

Maximum diameter of longitudinal rebars:

$$\phi_{sl}^{\ *} = \phi_{sl,max}\left(\frac{2.9}{f_{ctm}}\right) = 13.0 \text{ [mm]}$$

Limit of stress permitted in longitudinal rebars immediately after cracking:

$\sigma_{sl,lim} = 320$ [N/mm^2] (for $w_k = 0.3$ [mm] and $p_{sl} = 100$ [mm])

$$\therefore\ z_{sl,eq\text{-}cc} \sum A_{sl,r}\ \sigma_{sl,lim} > M_{Edh} \quad \text{OK}$$

$$\left(\frac{M_{Edh}}{z_{sl,eq\text{-}cc} \sum A_{sl,r}\ \sigma_{sl,lim}} = 0.79 \right)$$

Design of Composite Beam

[Verifications of structural resistance in construction stage]

Analysis of design moment and shear force

Maximum design distributed load:

$$w_{con,max} = B_b \left(g_{k,1} \gamma_{G,sup} + q_{k,1} \gamma_Q \right) = 19.3 \text{ [kN/m]}$$

Design sagging moment:

$$M_{Eds} = \frac{w_{con,max} {L_b}^2}{8} = 543.0 \text{ [kNm]}$$

Design shear force:

$$V_{Ed} = \frac{w_{con,max} L_b}{2} = 144.8 \text{ [kN]}$$

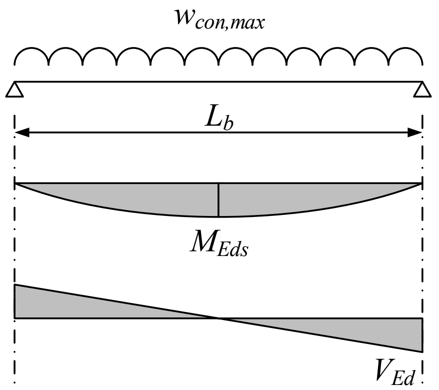


Figure AIV.5: Design load with corresponding moment and shear force

Check section classification

Classification of steel flange:

$$\frac{B_a\text{-}t_w\text{-}2r}{2t_f} = 4.84 < 9 \sqrt{\frac{235}{f_{ay}}} = 7.32 \quad \rightarrow \text{Class 1}$$

Classification of steel web:

$$\frac{D_a\text{-}2t_f\text{-}2r}{t_w} = 70.2 < 124 \sqrt{\frac{235}{f_{ay}}} = 100.9 \quad \rightarrow \text{Class 3}$$

$\therefore$ Class 1 steel flange & Class 3 steel web $\rightarrow$ Effective Class 2 OK

Check shear resistance and moment resistance

Shear area:

$$A_V = \max\left\{A_a\text{-}2B_a t_f\text{+}(t_w\text{+}2r)t_f\ ;\ 1.2D_a t_w\right\} = 72.1\ [\text{cm}^2]$$

Plastic shear resistance:

$$V_{pl,a,Rd} = \frac{A_V f_{ayd}}{\sqrt{3}} = 1478.7\ [\text{kN}]$$

Nominal value of yield strength of web:

$f_{wy} = 355$ [N/mm^2] (for $t_w = 9$ [mm])

Minimum shear buckling coefficient:

$k_{\tau,min} = 5.34$ (without rigid transverse and longitudinal stiffeners)

Modified slenderness of web:

$$\lambda_w = 0.76\sqrt{\frac{f_{wy}}{k_{\tau,min}\left\{190000\left(\frac{t_w}{B_a}\right)^2\right\}}} = 1.06$$

Factor for contribution of web to the shear buckling resistance:

$$\chi_w = \frac{0.83}{\lambda_w} = 0.79\ \left(\text{for } \frac{0.83}{1.2} \le \lambda_w\right)$$

Shear buckling resistance:

$$V_{b,a,Rd} = \min\left\{\frac{\chi_w f_{wy}\left(D_a\text{-}2t_f\text{-}2r\right)t_w}{\sqrt{3}\gamma_a}\ ;\ \frac{1.2 f_{wy}\left(D_a\text{-}2t_f\text{-}2r\right)t_w}{\sqrt{3}\gamma_a}\right\} = 969.3\ [\text{kN}]$$

$$\left(\text{for } \frac{D_a\text{-}2t_f}{t_w} > \frac{72}{1.2}\sqrt{\frac{235}{f_{ay}}}\right)$$

$$\therefore\ \min\left(V_{pl,a,Rd}\ ;\ V_{b,a,Rd}\right) > V_{Ed}\quad \text{OK}\ \left(\frac{V_{Ed}}{\min\left(V_{pl,a,Rd}\ ;\ V_{b,a,Rd}\right)} = 0.15\right)$$

Effective plastic section modulus:

$$W_{eff,pl,a} = \left\{\begin{array}{c} B_a t_f\left(D_a\text{-}t_f\right)\text{+}0.4292r^2\left(D_a\text{-}2t_f\text{-}0.4467r\right)\text{+}t_w r\left(D_a\text{-}2t_f\text{-}r\right) \\ +\frac{t_w}{2}\left(40t_w\sqrt{\frac{235}{f_{ay}}}\right)^2 + \frac{t_w}{2}\left(20t_w\sqrt{\frac{235}{f_{ay}}}\right)^2 \\ +t_w\left(20t_w\sqrt{\frac{235}{f_{ay}}}\right)\left(D_a\text{-}t_f\text{-}r\text{-}40t_w\sqrt{\frac{235}{f_{ay}}}\text{-}t_f\text{-}r\text{-}10t_w\sqrt{\frac{235}{f_{ay}}}\right)\end{array}\right\} = 3219\ [\text{cm}^3]$$

Plastic moment resistance:

$M_{pl,a,Rd} = W_{eff,pl,a} f_{ayd} = 1142.7$ [kNm] (for effective Class 2 cross-section)

$$\therefore\ M_{pl,a,Rd} > M_{Eds}\quad \text{OK}\ \left(\frac{M_{Eds}}{M_{pl,a,Rd}} = 0.48\right)$$

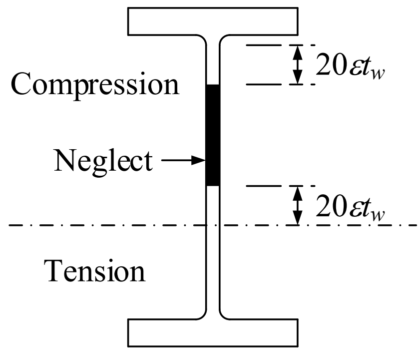


Figure AIV.6: Effective cross-section for effective Class 2

[Verifications of structural resistance in composite stage]

Analysis of design moment and shear force

Design moment and shear force in composite stage can be calculated by the equations in Section 5.3. The equivalent initial rotational stiffness $S_{j,ini,eq}$ incorporating the flexural rigidity of the core wall is used as the rotational stiffness S_j in structural analysis.

Maximum design distributed load:

$$w_{com,max} = B_b \left(g_{k,1}\gamma_{G,sup} + g_{k,3}\gamma_{G,sup} + q_{k,1}\gamma_Q \right) = 51.6 \text{ [kN/m]}$$

Design sagging moment:

M_{Eds} = 1117.5 [kNm]

Design hogging moment:

M_{Edh} = 709.3 [kNm]

Design shear force:

V_{Ed} = 434.1 [kN]

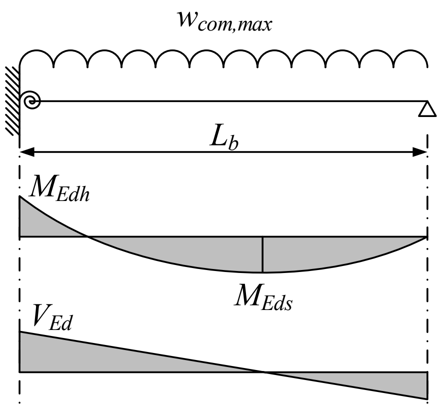


Figure AIV.7: Design load with corresponding moment and shear force

Check section classification

Classification of steel flange in sagging moment region:

Bottom flange is in tension → Class 1

Effective width in sagging moment region:

$$b_{effs} = b_{0s}+\min\left(\frac{x_0}{4}\ ;\ B_b\text{-}b_{0s}\right) = 3000.0\ [\text{mm}]$$

Compression resistance of composite slab within b_{effs}:

$$R_{cs} = b_{effs}h_{cs}\left(0.85f_{cd,cs}\right) = 5049.0\ [\text{kN}]$$

Portion of part of cross-section in compression:

$$\alpha = \frac{\left(\frac{D_a\text{-}2t_f\text{-}2r}{2}\right)\text{-}\left(\frac{R_{cs}}{2t_wf_{ayd}}\right)}{D_a\text{-}2t_f\text{-}2r} = \text{-}0.75$$

Classification of steel web in sagging moment region:

Full web is in tension $\rightarrow$ Class 1

∴ Class 1 steel flange & Class 1 steel web $\rightarrow$ Class 1 OK

Classification of steel flange in hogging moment region:

$$\frac{(B_a\text{-}t_w\text{-}2r)}{2t_f} = 4.84 < 9\sqrt{\frac{235}{f_{ay}}} = 7.32 \quad \rightarrow \text{Class 1}$$

Effective width in hogging moment region:

$$b_{effh} = b_{0h}+\min\left\{\frac{2(L_b\text{-}x_0)}{4}\ ;\ B_b\text{-}b_{0h}\right\} = 1016.9\ [\text{mm}]$$

Cross-sectional area of longitudinal rebars within b_{effh}:

$$A_{sl} = \pi\left(\frac{\phi_{sl,1}}{2}\right)^2\left\lfloor\frac{b_{effh}}{p_{sl,1}}\right\rfloor+\pi\left(\frac{\phi_{sl,2}}{2}\right)^2\min\left(\left\lfloor\frac{b_{effh}}{p_{sl,2}}\right\rfloor\ ;\ \left\lfloor\frac{b_{sl,2}}{p_{sl,2}}\right\rfloor\right) = 17.20\ [\text{cm}^2]$$

Tension resistance of longitudinal rebars within b_{effh}:

$$R_{sl} = A_{sl}f_{sd} = 747.8\ [\text{kN}]$$

Portion of part of cross-section in compression:

$$\alpha = \frac{\left(\frac{D_a\text{-}2t_f\text{-}2r}{2}\right)+\left(\frac{R_{sl}}{2t_wf_{ayd}}\right)}{D_a\text{-}2t_f\text{-}2r} = 0.69$$

Equivalent vertical distance between longitudinal rebars and bottom of flange of steel beam:

$$z_{sl,eq\text{-}bf} = D_a+D_{cs}\text{-}z_{tcs\text{-}sl,eq} = 798.4\ [\text{mm}]$$

Stress or strain ratio:

$$\psi = 1\text{-}\frac{D_a\text{-}2t_f\text{-}2r}{\frac{A_aD_a+2A_{sl}z_{sl,eq\text{-}bf}}{2(A_a+A_{sl})}\text{-}t_f\text{-}r} = \text{-}0.71$$

Classification of steel web in hogging moment region:

$$\frac{D_a\text{-}2t_f\text{-}2r}{t_w} = 70.2 < \frac{42\sqrt{\dfrac{235}{f_{ay}}}}{0.67+0.33\psi} = 78.4 \quad \rightarrow \text{Class 3}$$

∴ Class 1 steel flange & Class 3 steel web → Effective Class 2 OK

Second moment of area in hogging moment region:

$$I_h = I_{ay} + \frac{A_a A_{sl}\left\{D_a+2\left(D_{ps}+h_{cs}\text{-}z_{tcs\text{-}sl,eq}\right)\right\}^2}{4(A_a+A_{sl})} = 130712\ [\text{cm}^4]$$

Elastic hogging moment resistance:

$$M_{el,Rdh} = \min\left\{\frac{I_h f_{sd}}{z_{sl,eq\text{-}bf}\text{-}\dfrac{A_a D_a+2A_{sl}z_{sl,eq\text{-}bf}}{2(A_a+A_{sl})}}\ ;\ \frac{I_h f_{ayd}}{\dfrac{A_a D_a+2A_{sl}z_{sl,eq\text{-}bf}}{2(A_a+A_{sl})}}\right\} = 1149.9\ [\text{kNm}]$$

Modular ratio for short-term loading:

$$n_0 = \frac{E_a}{E_{cm,cs}} = 6.36$$

Vertical distance between centre of un-cracked concrete flange and un-cracked composite section:

$$z_0 = \frac{(A_a+A_{sl})\left(0.5D_a+D_{ps}+0.5h_{cs}\right)}{A_a+A_{sl}+\left(\dfrac{h_{cs}b_{effh}}{n_0}\right)} = 214.7\ [\text{mm}]$$

Coefficient taking into account of stress distribution within section immediately prior to cracking:

$$k_c = \min\left\{\frac{1}{1+\left(\dfrac{h_{cs}}{2z_0}\right)}+0.3\ ;\ 1.0\right\} = 1.00$$

Required minimum reinforcement ratio:

$$\rho_{sl,req} = \frac{f_{ay}}{235}\frac{f_{ctm}}{f_{sk}}\sqrt{k_c} = 0.88\%\ \text{(for Class 2 cross-section)}$$

Cross-sectional area of composite slab within b_{effh} above profiled steel sheeting:

$$A_{cs} = b_{effh}h_{cs} = 1006.8\ [\text{cm}^2]$$

Required minimum cross-sectional area of longitudinal rebars within b_{effh}:

$$A_{sl,req} = \rho_{sl,req}A_{cs} = 8.82\ [\text{cm}^2]$$

$M_{el,Rdh} > M_{j,Rd}$ → no need to check $A_{sl} > A_{sl,req}$

Check minimum degree of shear connection

Distance between inflection points in sagging moment region:

$$L_{es} = x_0 = 13166\ [\text{mm}]$$

Correction factor of headed stud taking into account h_{hs}/d_{hs}:

$$\alpha_{hs} = 1.00 \ \left(\text{for } \frac{h_{hs}}{d_{hs}} > 4\right)$$

Shear resistance of a headed stud:

$$P_{Rd} = \min\left(\frac{0.8 f_{hsu} \pi d_{hs}^{\ 2}}{4\gamma_V} \ ; \ \frac{0.29 \alpha d_{hs}^{\ 2}\sqrt{f_{ck,cs}E_{cm,cs}}}{\gamma_V}\right) = 81.7 \text{ [kN]}$$

Number of headed studs arranged within half of L_{es}:

$$N_{hss} = \left\lfloor \frac{\left(\frac{L_{es}}{2}\right) \text{-} h_{cj\text{-}fhs}}{p_{ps}} \right\rfloor n_{hss} = 64$$

Maximum reduction factor for shear resistance of a headed stud in sagging moment region:

$$k_{ts,max} = 0.60 \ \left(\text{for } n_{hss} = 2,\ t_{ps} \le 1,\ d_{hs} \le 20, \text{ and sheeting with holes}\right)$$

Reduction factor for shear resistance of a headed stud in sagging moment region:

$$k_{ts} = \min\left\{\frac{0.7}{\sqrt{\min(n_{hss}\ ;\ 2)}} \frac{b_{0,min}}{D_{ps}}\left(\frac{h_{hs}}{D_{ps}}\text{-}1\right)\ ;\ k_{ts,max}\right\} = 0.60$$

Longitudinal shear force transfer within half of L_{es}:

$$R_{qs} = N_{hss} k_{ts} P_{Rd} = 3135.6 \text{ [kN]}$$

Tension (Compression) resistance of steel beam:

$$R_a = A_a f_{ayd} = 4505.0 \text{ [kN]}$$

Degree of shear connection in sagging moment region:

$$\eta_s = \frac{R_{qs}}{\min(R_a\ ;\ R_{cs})} = 0.70$$

Required minimum degree of shear connection in sagging moment region:

$$\eta_{s,req} = \max\left\{1\text{-}\left(\frac{355}{f_{ayd}}\right)(0.75\text{-}0.03L_{es});\ 0.4\right\} = 0.64 \ (\text{for } L_{es} \le 25 \text{ [m]})$$

$$\therefore\ \eta_s > \eta_{s,req} \quad \text{OK} \ \left(\frac{\eta_{s,req}}{\eta_s} = 0.93\right)$$

Half of distance between inflection points in hogging moment region:

$$\frac{L_{eh}}{2} = L_b\text{-}x_0 = 1834 \text{ [mm]}$$

Number of headed studs arranged within half of L_{eh}:

$$N_{hsh} = \left\lfloor \frac{\left(\frac{L_{eh}}{2}\right) \text{-} h_{cj\text{-}fhs}}{p_{ps}} \right\rfloor n_{hsh} = 16$$

Maximum reduction factor for shear resistance of a headed stud in hogging moment region:

$$k_{th,max} = 0.60 \ \left(\text{for } n_{hsh} = 2,\ t_{ps} \le 1,\ d_{hs} \le 20, \text{ and sheeting with holes}\right)$$

Reduction factor for shear resistance of a headed stud in hogging moment region:

$$k_{th} = \min\left\{\frac{0.7}{\sqrt{\min(n_{hsh}\ ;\ 2)}}\frac{b_{0,min}}{D_{ps}}\left(\frac{h_{hs}}{D_{ps}}-1\right)\ ;\ k_{th,max}\right\} = 0.60$$

Longitudinal shear force transfer within half of L_{eh}:

$$R_{qh} = N_{hsh}k_{th}P_{Rd} = 783.9\ [\text{kN}]$$

Degree of shear connection in hogging moment region:

$$\eta_h = \frac{R_{qh}}{\min(R_a\ ;\ R_{sl})} = 1.05$$

Required minimum degree of shear connection in hogging moment region:

$\eta_{h,req} = 1.00$ (full shear connection)

$$\therefore\ \eta_h > \eta_{h,req}\quad \text{OK}\ \left(\frac{\eta_{h,req}}{\eta_h} = 0.95\right)$$

<u>Check shear resistance and moment resistance</u>

Plastic shear resistance:

$V_{pl,Rd} = 1478.7$ [kN]

Shear buckling resistance:

$V_{b,Rd} = 969.3$ [kN]

$$\therefore\ \min\left(V_{pl,Rd}\ ;\ V_{b,Rd}\right) > V_{Ed}\quad \text{OK}\ \left(\frac{V_{Ed}}{\min\left(V_{pl,Rd}\ ;\ V_{b,Rd}\right)} = 0.45\right)$$

Tension (Compression) resistance of overall web of steel beam:

$$R_w = R_a\text{-}2B_a t_f f_{ayd} = 2233.0\ [\text{kN}]$$

Tension (Compression) resistance of clear web of steel beam:

$$R_v = \left(D_a\text{-}2t_f\text{-}2r\right)t_w f_{ayd} = 2019.2\ [\text{kN}]$$

Tension (Compression) resistance of effective clear web of steel beam:

$$R_{eff,v} = 40{t_w}^2 f_{ayd}\sqrt{\frac{235}{f_{ay}}} = 935.8\ [\text{kN}]$$

Location of plastic neutral axis for full shear connection:

$R_a \leq R_{cs}\quad \rightarrow PNA$ in concrete flange

Plastic sagging moment resistance with full shear connection:

$$M_{plf,Rds} = R_a\left\{\frac{D_a}{2}+D_{cs}\text{-}\frac{R_a}{R_{cs}}\frac{\left(D_{cs}\text{-}D_{ps}\right)}{2}\right\} = 2053.5\ [\text{kNm}]$$

Location of plastic neutral axis for partial shear connection:

$R_w \leq R_{qs}\quad \rightarrow PNA$ in steel flange

Plastic sagging moment resistance with partial shear connection:

$$M_{plp,Rds} = R_a \frac{D_a}{2} + R_{qs}\left(D_{cs} - \frac{R_{qs}}{R_{cs}}\frac{D_{cs}-D_{ps}}{2}\right) - \frac{\left(R_a - R_{qs}\right)^2}{4B_a f_{ayd}} = 1944.1 \text{ [kNm]}$$

$$\therefore \min\left(M_{plf,Rds}\,;M_{plp,Rds}\right) > M_{Eds} \quad \text{OK} \quad \left(\frac{M_{Eds}}{\min\left(M_{plf,Rds}\,;M_{plp,Rds}\right)} = 0.57\right)$$

Equivalent vertical distance between longitudinal rebars and top of flange of steel beam:

$z_{sl,eq\text{-}tf} = D_{cs} - z_{tcs\text{-}sl,eq} = 98.4$ [mm]

Tension (Compression) resistance of effective steel beam:

$R_{eff,a} = R_a - R_v + R_{eff,v} = 3421.6$ [kN]

Location of plastic neutral axis for with full shear connection:

$R_{sl} < R_{eff,v} \quad \rightarrow PNA$ in steel web

Plastic hogging moment resistance with full shear connection:

$$M_{plf,Rdh} = \left\{\begin{array}{c} W_{pl,a} f_{ayd} + R_{sl}\left(\dfrac{D_a}{2} + z_{sl,eq\text{-}tf}\right) \\ -\dfrac{R_{sl}^2 + (R_v + R_{sl})\left(R_v + R_{sl} - 2R_{eff,v}\right)}{4 t_w f_{ayd}} \end{array}\right\} = 1263.7 \text{ [kNm]}$$

Tension (Compression) resistance of flange of steel beam:

$R_f = B_a t_f f_{ayd} = 1136.0$ [kN]

Vertical distance between centres of top and bottom flange of steel beam:

$z_{ctf\text{-}cbf} = D_a - t_f = 684.0$ [mm]

Plastic moment resistance after deducting shear area:

$$M_{pl,f,Rd} = R_f z_{ctf\text{-}cbf} + R_{sl}\left(\frac{D_a}{2} + z_{sl,eq\text{-}tf}\right) = 1112.4 \text{ [kNm]}$$

Reduced hogging moment resistance making allowance for presence of shear force:

$$M_{y,v,Rdh} = M_{plf,Rdh} = 1263.7 \text{ [kNm]} \quad \left(\text{for } V_{Ed} \le \frac{V_{pl,Rd}}{2}\right)$$

$$\therefore \min\left(M_{plf,Rdh}\,;M_{y,v,Rdh}\right) > M_{Edh} \quad \text{OK} \quad \left(\frac{M_{Edh}}{\min\left(M_{plf,Rdh}\,;M_{y,v,Rdh}\right)} = 0.56\right)$$

<u>Check lateral-torsional buckling</u>

Polar radius of gyration of area of steel beam:

$$i_{ax} = \sqrt{\frac{\left(I_{ay} + I_{az}\right)}{A_a}} = 284.1 \text{ [mm]}$$

Vertical distance between centre of composite slab and centre of steel beam:

$$z_{ccs\text{-}ca} = \frac{D_a}{2} + \frac{D_{cs}}{2} = 425.0 \text{ [mm]}$$

e value:

$$e = \frac{(A_a+A_{sl})I_{ay}}{A_a z_{ccs\text{-}ca} A_{sl}} = 1557.3 \text{ [mm]}$$

k_c factor:

$$k_c = \frac{\dfrac{z_{ctf\text{-}cbf} I_h}{I_{ay}}}{\left(\dfrac{z_{ctf\text{-}cbf}^2}{4}+i_{ax}^2\right)/e+z_{ctf\text{-}cbf}} = 1.10$$

Property of distribution of moment (conservative value):

$$C_4 = 33.9 \left(\text{for } \psi = \frac{M_{Edh}}{\left(\dfrac{w_{com,max}L_b^2}{8}\right)} < 0.50 \right)$$

Length between points at which bottom flange of steel beam is laterally restrained:

$$L_{cr} = L_b = 15000 \text{ [mm]}$$

Cross-sectional area of transverse rebars per unit length:

$$A_{st} = \left\lfloor \frac{1000}{p_{st,1}} \right\rfloor \pi \left(\frac{\phi_{st,1}}{2}\right)^2 = 3.93 \text{ [cm}^2\text{/m]}$$

Area per unit length of concrete slab in compression:

$$A_{c,c} = \frac{A_c\text{-}1000h_{cs}}{\left(\dfrac{E_a}{E_{cm,cs}}\right)} = 71.1 \text{ [cm}^2\text{/m]}$$

Equivalent vertical distance between transverse rebars and concrete slab in compression:

$$z_{st,eq\text{-}ccs,c} = D_{cs}\text{-}z_{tcs\text{-}st,eq}\text{-}\frac{D_{ps}}{2} = 79.5 \text{ [mm]}$$

Equivalent Vertical distance between transverse rebars and neutral axis of composite slab:

$$z_{st,eq\text{-}na} = \frac{z_{st,eq\text{-}ccs,c}}{\left(1+\dfrac{A_{st}}{A_{c,c}}\right)} = 75.3 \text{ [mm]}$$

Vertical distance between neutral axis of composite slab and centre of concrete slab in compression:

$$z_{na\text{-}ccs,c} = z_{st,eq\text{-}ccs,c}\text{-}z_{st,eq\text{-}na} = 4.2 \text{ [mm]}$$

Second moment of area of cracked composite slab in direction transverse to steel beam:

$$I_{cs2} = A_{st} z_{st,eq\text{-}na}^2 + A_{c,c}\left(z_{na\text{-}ccs,c}^2+\frac{D_{ps}^2}{12}\right) = 389.4 \text{ [cm}^4\text{]}$$

Cracked flexural stiffness per unit length of composite slab:

$$E_a I_{cs2} = 817.7 \text{ [kNm}^2\text{/m]}$$

Flexural stiffness of cracked composite slab in direction transverse to steel beam:

$$k_1 = \frac{4E_a I_{cs2}}{B_b} = 1090.3 \text{ [kN/rad] (for continuous slab)}$$

Flexural stiffness of web of steel beam:

$$k_2 = \frac{E_a t_w^{\ 3}}{4\left(1\text{-}0.3^2\right) z_{ctf\text{-}cbf}} = 61.5 \text{ [kN/rad]}$$

Transverse (rotational) stiffness per unit length:

$$k_s = \frac{k_1 k_2}{k_1 + k_2} = 58.2 \text{ [kN/rad]}$$

Second moment of area of bottom flange of steel beam about minor axis (z-z axis):

$$I_{bfz} = \frac{t_f B_a^{\ 3}}{12} = 1067 \text{ [cm}^4\text{]}$$

Elastic critical moment for lateral-torsional buckling:

$$M_{cr} = \frac{k_c C_4}{L_{cr}} \sqrt{\left[\left\{\frac{E_a}{2(1+0.3)}\right\} I_{T,a} + \frac{k_s L_{cr}^{\ 2}}{\pi^2}\right] E_a I_{bfz}} = 4390.4 \text{ [kNm]}$$

Location of plastic neutral axis for $M_{pl,Rkh}$:

$A_{sl} f_{sk} < R_{eff,v} \quad \rightarrow PNA$ in steel web

Characteristic value of plastic hogging moment resistance:

$$M_{pl,Rkh} = \left\{ \begin{array}{c} W_{pl,a} f_{ayd} + A_{sl} f_{sk} \left(\dfrac{D_a}{2} + z_{sl,eq\text{-}tf}\right) \\ -\dfrac{\left(A_{sl} f_{sk}\right)^2 + \left(R_v + A_{sl} f_{sk}\right)\left(R_v + A_{sl} f_{sk} \text{-} 2R_{eff,v}\right)}{4 t_w f_{ayd}} \end{array} \right\} = 1266.8 \text{ [kNm]}$$

Non-dimensional slenderness for lateral-torsional buckling:

$$\lambda_{LT} = \sqrt{\frac{M_{pl,Rkh}}{M_{cr}}} = 0.54$$

Imperfection factor corresponding to appropriate lateral-torsional bucklinγ χurve:

$$\alpha_{LT} = 0.76 \ \left(\text{for } \frac{D_a}{B_a} = 3.5 > 2.0\right)$$

Value to determine reduction factor for lateral-torsional buckling:

$$\Phi_{LT} = 0.5\left\{1 + \alpha_{LT}(\lambda_{LT}\text{-}0.4) + 0.75\lambda_{LT}^{\ 2}\right\} = 0.66$$

Reduction factor for lateral-torsional buckling:

$$\chi_{LT} = \min\left(\frac{1}{\Phi_{LT} + \sqrt{\Phi_{LT}^{\ 2}\text{-}0.75\lambda_{LT}^{\ 2}}} \ ; 1.0 \ ; \ \frac{1}{\lambda_{LT}^{\ 2}}\right) = 0.89$$

Buckling moment resistance of laterally unrestrained composite beam:

$$M_{LT,Rd} = \chi_{LT} M_{plf,Rdh} = 1119.4 \text{ [kNm]}$$

Requirements to use calculation method in DD ENV 1994-1-1:

$$E_{cm,cs}I_{cs2} \geq 0.35E_a t_w^2 \frac{B_b}{D_a} \text{ and } \frac{p_{ps}}{B_a} \leq 0.4 f_{hsu} d_{hs}^2 \frac{1\text{-}\chi_{LT}\lambda_{LT}^2}{k_s \chi_{LT}\lambda_{LT}^2} \rightarrow \text{can be used}$$

$$\therefore M_{LT,Rd} > M_{Edh} \quad \text{OK} \left(\frac{M_{Edh}}{M_{LT,Rd}} = 0.63 \right)$$

Check longitudinal shear resistance

< Beam-to-wall composite joint side >

Change of longitudinal force in composite slab:

$$\Delta N_L = \left\{ \begin{array}{c} \min(R_a \,;\, R_{cs} \,;\, N_{hss}P_{Rd}) \\ +\min\left(\left\lceil \frac{L_b\text{-}x_0\text{-}h_{cj\text{-}fhs}}{p_{ps}} \right\rceil n_{hsh}P_{Rd} \,;\, R_{sl} \right) \end{array} \right\} = 5253 \text{ [kN]}$$

x-coordinate at point of maximum sagging moment:

$$x_s = \frac{1}{w_{com,max}} \left(\frac{w_{com,max}L_b}{2} - \frac{M_{Edh}}{L_b} \right) = 6583 \text{ [mm]}$$

Design longitudinal shear stress in composite slab:

$$v_{L,Ed} = \frac{\Delta N_L}{2h_{cs}(L_b\text{-}x_s)} = 3.15 \text{ [N/mm}^2\text{]}$$

Minimum angle to minimize cross-sectional area of transverse rebars:

$$\theta_{min} = 38.6\ °$$

Required tension resistance of transverse reinforcement per unit length:

$$R_{tr,req} = \frac{1000 h_{cs} v_{L,Ed}}{\cot\theta_{min}} = 249.1 \text{ [kN/m]}$$

< Pinned joint side >

Change of longitudinal force in composite slab:

$$\Delta N_L = \min(R_a \,;\, R_{cs} \,;\, N_{hss}P_{Rd}) = 4505 \text{ [kN]}$$

Design longitudinal shear stress in composite slab:

$$v_{L,Ed} = \frac{\Delta N_L}{2h_{cs}x_s} = 3.46 \text{ [N/mm}^2\text{]}$$

Minimum angle to minimize cross-sectional area of transverse rebars:

$$\theta_{min} = 38.6\ °$$

Required tension resistance of transverse reinforcement per unit length:

$$R_{tr,req} = \frac{1000 h_{cs} v_{L,Ed}}{\cot\theta_{min}} = 273.1 \text{ [kN/m]}$$

Cross-sectional area of transverse rebars per unit length for row 1:

$$A_{st,1} = \left\lfloor \frac{1000}{p_{st,1}} \right\rfloor \pi \left(\frac{\phi_{st,1}}{2} \right)^2 = 3.93 \text{ [cm}^2\text{/m]}$$

Tension resistance of transverse reinforcement per unit length:

$R_{st}+R_{pse} = A_{st,1}f_{sd}+A_{pse}f_{psd} = 668.5$ [kN/m]

$$\therefore\ R_{st}+R_{pse} > \max\left(R_{tr,req}\right)\quad \text{OK}\ \left(\frac{\max\left(R_{tr,req}\right)}{R_{st}+R_{pse}} = 0.41\right)$$

Required minimum cross-sectional area of transverse rebars per unit length:

$$A_{st,req} = 1000h_{cs}\frac{0.08\sqrt{f_{ck,cs}}}{f_{sk}} = 0.87\ [\text{cm}^2/\text{m}]$$

$$\therefore\ A_{st,1} > A_{st,req}\quad \text{OK}\ \left(\frac{A_{st,req}}{A_{st,1}} = 0.22\right)$$

Crushing shear stress of concrete slab:

$$v_{Rd} = 0.6\left(1\text{-}\frac{f_{ck,cs}}{250}\right)f_{cd,cs}\sin\theta_{min}\cos\theta_{min} = 5.15\ [\text{N/mm}^2]$$

$$\therefore\ v_{Rd} > \max\left(v_{L,Ed}\right)\quad \text{OK}\ \left(\frac{\max\left(v_{L,Ed}\right)}{v_{Rd}} = 0.67\right)$$

[Verifications of serviceability in construction stage]

Analysis of deflection

Design distributed load due to “dead loads”:

$w_{con,P} = B_b g_{k,1} = 12.6$ [kN/m]

Design distributed load due to “live loads”:

$w_{con,V} = B_b q_{k,1} = 1.5$ [kN/m]

Design distributed load due to “dead loads and live loads”:

$w_{con,P+V} = B_b\left(g_{k,1}+q_{k,1}\right) = 14.1$ [kN/m]

Deflection due to “dead loads”:

$$\delta_P = \frac{w_{con,P}L_b^{\ 4}}{384E_aI_{ay}} = 39.6\ [\text{mm}]$$

Deflection due to “live loads”:

$$\delta_V = \frac{w_{con,V}L_b^{\ 4}}{384E_aI_{ay}} = 4.7\ [\text{mm}]$$

Deflection due to “dead loads and live loads”:

$$\delta_{P+V} = \frac{w_{con,P+V}L_b^{\ 4}}{384E_aI_{ay}} = 44.3\ [\text{mm}]$$

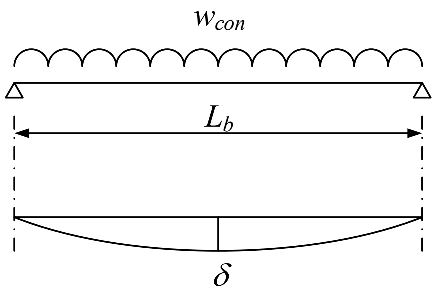


Figure AIV.8: Design load with corresponding deflection

Check deflection

Limit of deflection due to "live loads":

$$\delta_{V,lim} = \frac{L_b}{360} = 41.7 \text{ [mm]}$$

Limit of deflection due to "dead loads and live loads":

$$\delta_{P+V,lim} = \frac{L_b}{250} = 60.0 \text{ [mm]}$$

$$\therefore\ \delta_V < \delta_{V,lim} \quad \text{OK} \left(\frac{\delta_V}{\delta_{V,lim}} = 0.11 \right)$$

$$\therefore\ \delta_{P+V} < \delta_{P+V,lim} \quad \text{OK} \left(\frac{\delta_{P+V}}{\delta_{P+V,lim}} = 0.74 \right)$$

[Verifications of serviceability in composite stage]

Analysis of deflection

Deflection in composite stage can be calculated by the equations in Section 5.3. The equivalent initial rotational stiffness $S_{j,ini,eq}$ incorporating the flexural rigidity of the core wall is used as the rotational stiffness S_j in structural analysis.

Design distributed load due to "superimposed dead loads":

$w_{com,P} = B_b g_{k,3} = 6.0$ [kN/m]

Maximum design distributed load due to "live loads":

$w_{com,V,max} = B_b q_{k,2} = 18.0$ [kN/m]

Deflection due to "superimposed dead loads":

$\delta_P = 4.4$ [mm]

Deflection due to "live loads" (Load-case 1):

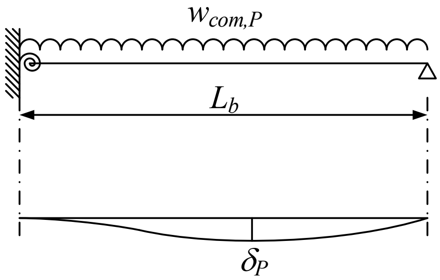
$\delta_V = 13.2$ [mm]

Deflection due to "dead loads and superimposed dead loads":

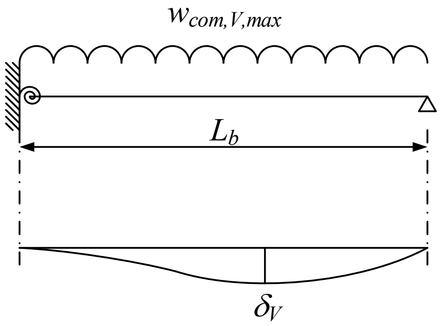
$\delta_{tP} = 42.6$ [mm]

Deflection due to "dead loads, superimposed dead loads, and live loads":

$\delta_{tP+V} = \delta_{tP} + \delta_V = 55.8$ [mm]



(a) Superimposed dead loads



(b) Live loads

Figure AIV.9: Design load with corresponding deflection

Check deflection

Limit of deflection due to “live loads”:

$$\delta_{V,lim} = \frac{L_b}{360} = 41.7 \text{ [mm]}$$

Limit of deflection due to “dead loads, superimposed dead loads, and live loads”:

$$\delta_{P+V,lim} = \frac{L_b}{250} = 60.0 \text{ [mm]}$$

$$\therefore\ \delta_V < \delta_{V,lim} \quad \text{OK} \left(\frac{\delta_V}{\delta_{V,lim}} = 0.32 \right)$$

$$\therefore\ \delta_{tP+V} < \delta_{P+V,lim} \quad \text{OK} \left(\frac{\delta_{tP+V}}{\delta_{P+V,lim}} = 0.93 \right)$$

Analysis of natural frequency

Natural frequency in composite stage can be obtained from the deflection due to “dead loads, superimposed dead loads, and 10% of live loads”. Also the deflection can be calculated by the equations in Section 5.3. The equivalent initial rotational stiffness $S_{j,ini,eq}$ incorporating the flexural rigidity of the core wall is used as the rotational stiffness S_j in structural analysis.

Design distributed load due to “dead loads, superimposed dead loads, and 10% of live loads”:

$$w_{com,P+0.1V} = B_b \left(g_{k,2} + g_{k,3} + 0.1 q_{k,2} \right) = 20.0 \text{ [kN/m]}$$

Deflection due to "dead loads, superimposed dead loads, and 10% of live loads":

$\delta_{P+0.1V} = 14.7$ [mm]

Natural frequency due to "dead loads, superimposed dead loads, and 10% of live loads":

$$f_{P+0.1V} = \frac{18}{\sqrt{\delta_{P+0.1V}}} = 4.7 \text{ [Hz]}$$

Check vibration

Required minimum natural frequency:

$f_{req} = 4.0$ [Hz]

$$\therefore f_{P+0.1V} > f_{req} \quad \text{OK} \left(\frac{f_{req}}{f_{P+0.1V}} = 0.85 \right)$$

Control of crack width

Maximum diameter of longitudinal rebars:

$$\phi_{sl}^{*} = \phi_{sl,max} \left(\frac{2.9}{f_{ctm}} \right) = 13.0 \text{ [mm]}$$

Limit of stress permitted in longitudinal rebars immediately after cracking:

$\sigma_{sl,lim} = 240$ [N/mm^2] $\left(\text{for } w_k = 0.3 \text{ [mm] and } \phi^{*} = 13 \text{ [mm]}\right)$

Effective width in hogging moment region:

$$b_{effh} = b_{0h} + \min\left\{ \frac{2(L_b\text{-}x_0)}{4} \; ; \; B_b\text{-}b_{0h} \right\} = 1244.4 \text{ [mm]}$$

Cross-sectional area of longitudinal rebars within b_{effh}:

$$A_{sl} = \pi \left(\frac{\phi_{sl,1}}{2} \right)^2 \left\lfloor \frac{b_{effh}}{p_{sl,1}} \right\rfloor + \pi \left(\frac{\phi_{sl,2}}{2} \right)^2 \min \left(\left\lfloor \frac{b_{effh}}{p_{sl,2}} \right\rfloor \; ; \; \left\lfloor \frac{b_{sl,2}}{p_{sl,2}} \right\rfloor \right) = 20.64 \text{ [cm}^2\text{]}$$

Vertical distance between centre of un-cracked concrete flange and un-cracked composite section:

$$z_0 = \frac{(A_a + A_{sl})\left(0.5D_a + D_{ps} + 0.5h_{cs}\right)}{A_a + A_{sl} + \left(\frac{h_{cs} b_{effh}}{n_0} \right)} = 194.8 \text{ [mm]}$$

Coefficient taking into account of stress distribution within section immediately prior to cracking:

$$k_c = \min\left\{ \frac{1}{1 + \left(\frac{h_{cs}}{2z_0} \right)} + 0.3 \; ; \; 1.0 \right\} = 1.00$$

Cross-sectional area of composite slab within b_{effh} above profiled steel sheeting:

$A_{cs} = b_{effh} h_{cs} = 1231.9$ [cm^2]

Required minimum cross-sectional area of longitudinal rebars within b_{effh}:

$$A_{sl,req} = \frac{0.72 k_c f_{ctm} A_{cs}}{\sigma_{sl,lim}} = 10.72 \text{ [cm}^2\text{]}$$

$$\therefore\ A_{sl} > A_{sl,req} \quad \text{OK} \left(\frac{A_{sl,req}}{A_{sl}} = 0.52 \right)$$

Equivalent vertical distance between longitudinal rebars and neutral axis:

$$z_{sl,eq\text{-}na} = \frac{A_{sl} z_{tcs\text{-}sl,eq} + A_a \left(h_{cs} + D_{ps} + 0.5 D_a \right)}{A_{sl} + A_a} - z_{tcs\text{-}sl,eq} = 385.7 \text{ [mm]}$$

Second moment of area in hogging moment region:

$$I_h = I_{ay} + \frac{A_a A_{sl} \left\{ D_a + 2\left(D_{ps} + h_{cs} - z_{tcs\text{-}sl,eq} \right) \right\}^2}{4(A_a + A_{sl})} = 135951 \text{ [cm}^4\text{]}$$

Stress in longitudinal rebars caused by M_{Edh}:

$$\sigma_{sl,0} = \frac{M_{Edh}}{I_h} z_{sl,eq\text{-}na} = 117 \text{ [N/mm}^2\text{]}$$

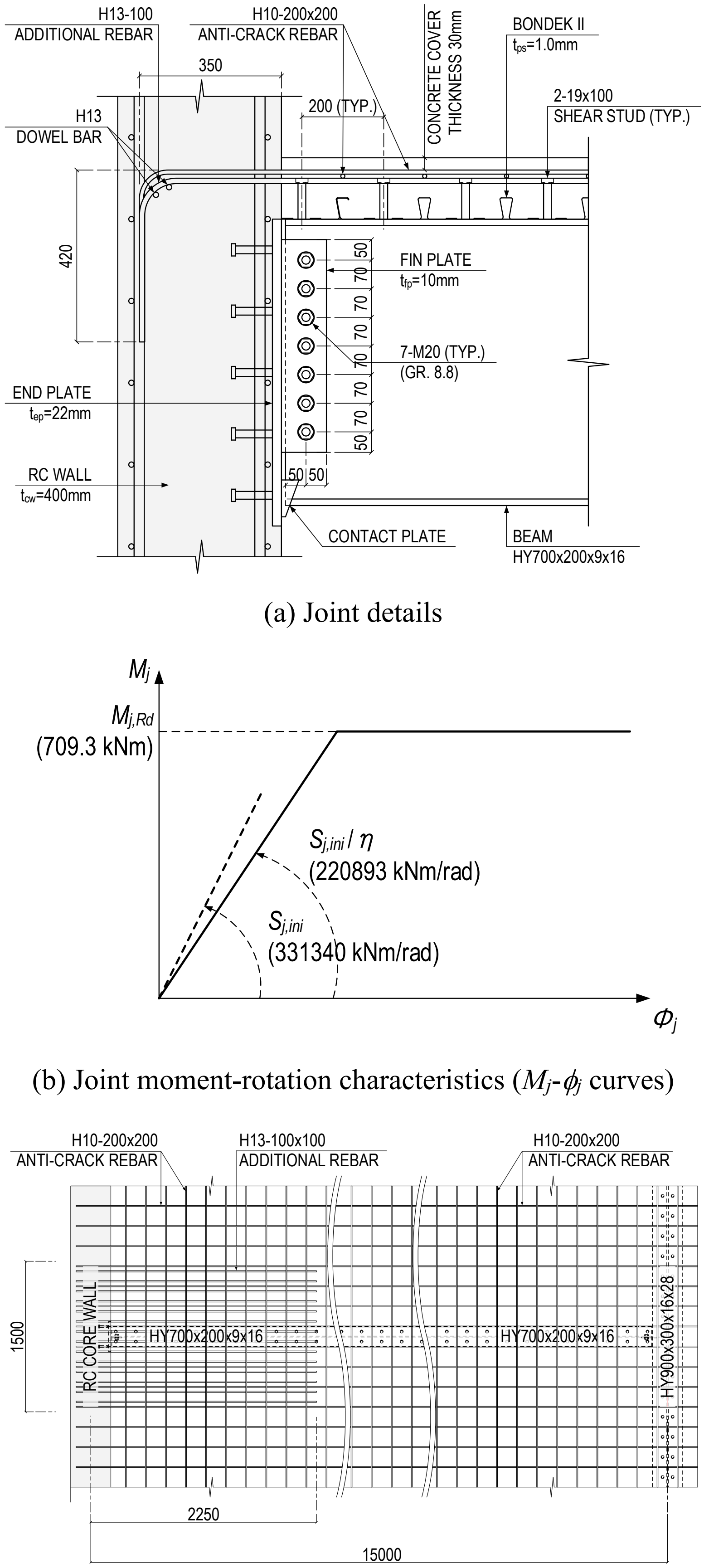
Correction of stress in longitudinal rebars for tension stiffening:

$$\Delta\sigma_{sl} = \frac{0.4 f_{ctm}}{\frac{(A_a + A_{sl}) I_h}{A_a I_{ay}} \left(\frac{A_{sl}}{A_{cs}} \right)} = 44 \text{ [N/mm}^2\text{]}$$

Tensile stress in longitudinal rebars due to direct loading:

$$\sigma_{sl} = \sigma_{sl,0} + \Delta\sigma_{sl} = 161 \text{ [N/mm}^2\text{]}$$

$$\therefore\ \sigma_{sl} < \sigma_{sl,lim} \quad \text{OK} \left(\frac{\sigma_{sl}}{\sigma_{sl,lim}} = 0.67 \right)$$



(a) Joint details

(b) Joint moment-rotation characteristics (M_j-ϕ_j curves)

(c) Distribution of shear studs and arrangement of reinforcing bars in concrete slab

Figure AIV.10: Designed semi-rigid composite joint and composite beam

Appendix V Comparison of Semi-rigid Joint and Pinned Joint

This appendix presents the comparison of semi-rigid joints and nominally pinned joints in term of weight saving for the beam sizing. As introduced in the **Foreword**, the significant advantage of using semi-rigid joints is lighter weight of steel beams compared to using pinned joints. Lighter weight of even only secondary steel beams leads to benefits because secondary beams often make up 20% to 30% of total steel tonnage. Furthermore, primary beam ends can also be semi-rigid according to the current design guide, resulting in big impact on reduction of material cost and improvement of construction productivity.

The degree of steel weight reduction depends on the floor beam layout, especially the beam span. In general, the ratio of steel weight reduction due to semi-rigid joints may be increased as beam span is longer. This is because the design of a long span beam is governed by its deflection which can be reduced effectively by semi-rigid joints. Table AV.1 and Table AV.2 show the comparison of pinned joints and semi-rigid joints on weight of secondary steel beam under 12 m beam span and 15 m beam span, respectively. Other design conditions such as beam spacing and design loads are the same. In both comparisons, pinned joints are used for Case 1 and semi-rigid joints are used for Case 2, thus the advantages of semi-rigid joints for pinned joints can be evaluated. In addition, Case 3 in which the cross-section of Case 2 is changed from UB to JIS is also provided to clarify the advantage of using JIS cross-sections. Case 3 in Table AV.2 corresponds to the design example 1 in **Appendix III**.

Comparing Case 1 and Case 2 in both tables, it can be observed that semi-rigid joints can contribute to about 15% reduction in weight of steel beam. In other words, an economical long span beam layout can be achieved without increasing the beam weight, which may lead to the efficient use of floor space. Besides, it can be observed from the comparison of Case 2 and Case 3 that the combination of JIS cross-sections and semi-rigid joints enables a greater weight reduction because JIS has a larger number of I-sections with high cross-sectional efficiency due to narrow flange width and thinner web plate.

Table AV.1: Comparison of pinned joints and semi-rigid joints on weight of secondary steel beam under 12 [m] beam span

Case		Case 1	Case 2	Case 3
Beam span L_b		12.0 [m]		
Beam spacing B_b		3.0 [m]		
Design load	SDL $g_{k,3}$	3.0 [kN/m^2]		
	IL $q_{k,2}$	5.0 [kN/m^2]		
Joint classification		Pinned	Semi-rigid	Semi-rigid
Steel beam	Cross-section	UB533x210x101	UB533x210x82	JIS550x200x9x12
	Mass per metre	101.0 [kg/m]	82.2 [kg/m] (-18.6%)	76.0 [kg/m] (-24.8%)

Table AV.2: Comparison of pinned joints and semi-rigid joints on weight of secondary steel beam under 15 [m] beam span

Case		Case 1	Case 2	Case 3
Beam span L_b		15.0 [m]		
Beam spacing B_b		3.0 [m]		
Design load	SDL $g_{k,3}$	3.0 [kN/m^2]		
	IL $q_{k,2}$	5.0 [kN/m^2]		
Joint classification		Pinned	Semi-rigid	Semi-rigid
Steel beam	Cross-section	UB762x267x147	UB610x229x125	JIS700x200x9x16
	Mass per metre	146.9 [kg/m]	125.1 [kg/m] (-14.8%)	99.6 [kg/m] (-32.2%)

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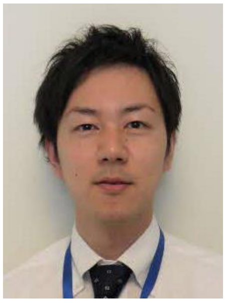
Dr J. Y. Richard Liew is an internationally recognised civil engineering leader with expertise in steel and concrete composite structures. He is Professor and Head of the Department of Civil and Environmental Engineering at the National University of Singapore (NUS), where he has made seminal contributions to the advancement of long-span structural systems and composite tall building design. Dr Liew is a Professional Engineer (Singapore), a Specialist Professional Engineer in Protective Security Engineering, and a Chartered Professional Engineer under the ASEAN framework. He is a Fellow of the Academy of Engineering Singapore, an Honorary Fellow and Past President of the Singapore Structural Steel Society, and an Honorary Fellow of the Hong Kong Institute of Steel Construction. His research and professional practice span lightweight and high-strength materials for offshore, marine, defence, building, and civil infrastructure applications. He has served as an expert and technical advisor on numerous landmark and strategically significant projects. A prolific scholar, he has co-authored more than ten books and published over 500 refereed technical papers, and currently serves on the editorial boards of several leading international journals. Beyond his academic and professional achievements, Dr Liew chairs several national committees on standards and specifications for steel, aluminium, and composite structures, and has played a pivotal role in shaping Singapore's national design standards in these domains.



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A complimentary book “Liew J Y R (2019), Design Guide for Buildable Steel Connections, Published by Singapoer Structural Steel Society, 671pp” can be downloaded from https://ssss.org.sg/~ssssorgs/images/stories/docs/Design_guide_for_buildable_steel_connections_Final_Version_20190327.pdf

Design Guide for Buildable Steel Connections

-Bolted and Welded Connection to SS EN1993-1-8

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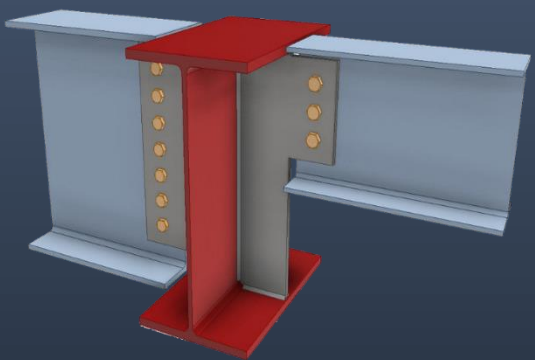

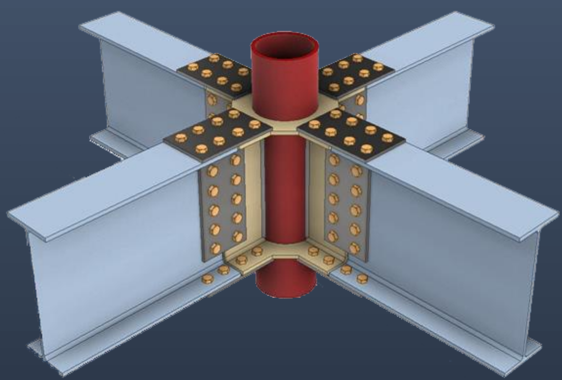

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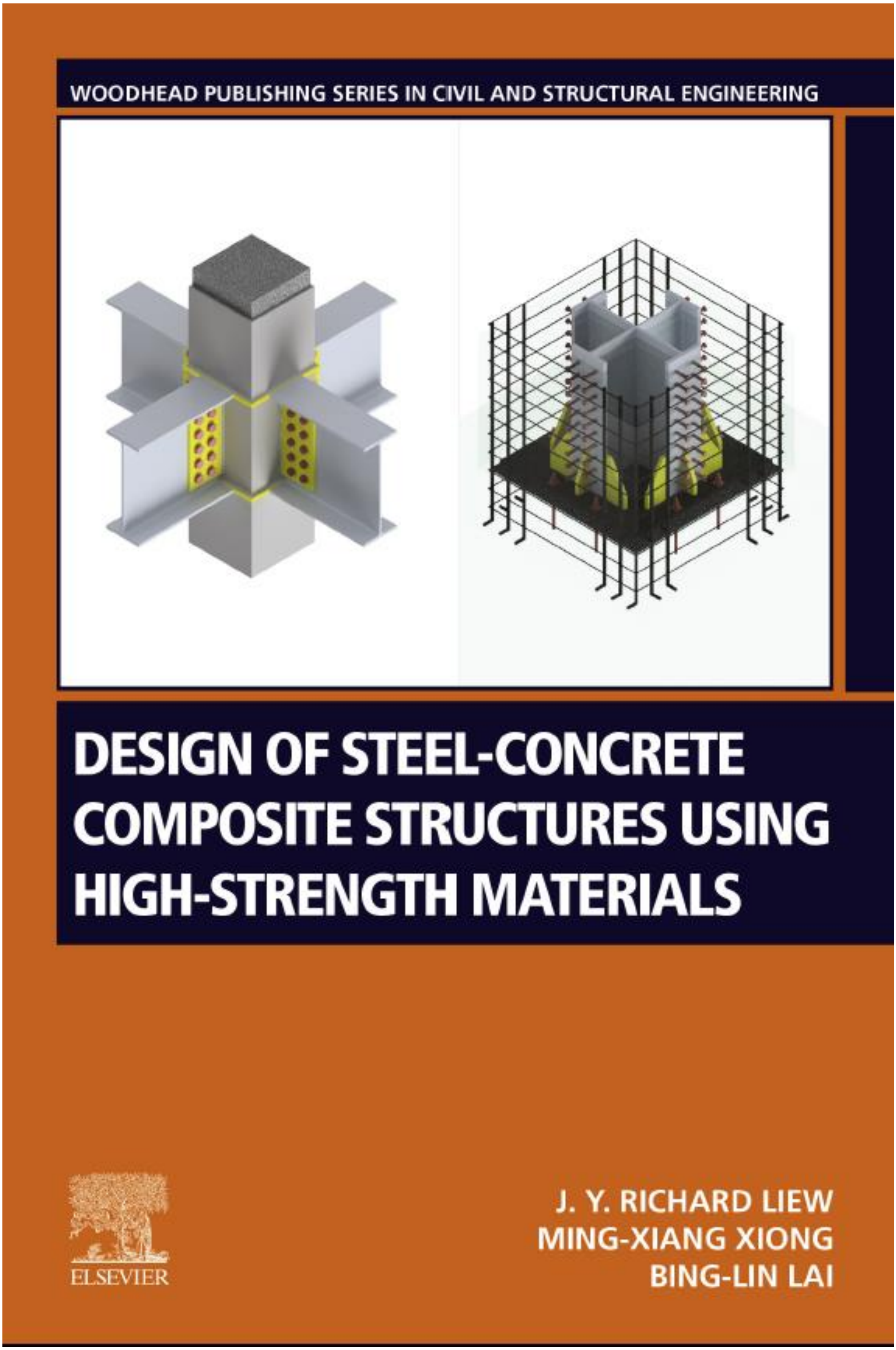
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