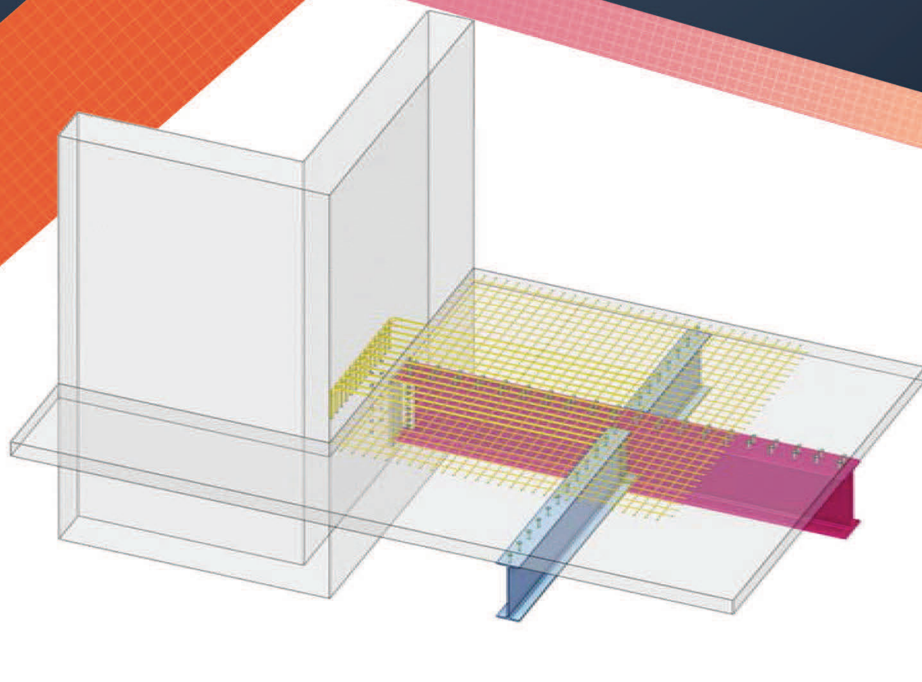
DESIGN GUIDE FOR SEMI-RIGID COMPOSITE JOINTS AND BEAMS

2nd Edition

J Y Richard Liew
Yuichi Nishida
Masaki Arita



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Design Guide for Semi-rigid Composite Joints and Beams

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DESIGN GUIDE FOR SEMI-RIGID COMPOSITE JOINTS AND BEAMS (2ND EDN.)
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Foreword

Multi-storey steel-concrete composite buildings that are braced against sidesway but not subjected to significant lateral and earthquake loads often use simplified joint details, such as fin plate bolted connections, in beam-to-column and beam-to-beam joints to speed up construction. While these joints are relatively easy to install on-site and are preferred for their simplicity, modern commercial buildings with long span and open space floor beam layouts may require a structural framework with semi-rigid connections for greater economy, without the need for complicated rigid joint detailing.

According to EN 1993-1-8[1], steel joints can be classified as pinned, semi-rigid, or rigid based on their initial rotational stiffness and moment resistance, depending on the analysis methods used in the design. Composite joints, as defined in EN 1994-1-1[2], are joints in which slab reinforcements are considered to calculate the rotational stiffness and moment resistance if the reinforcements are continuous or anchored at the joints. Thus, some of the simple steel joints defined in EN 1993-1-8 can be classified as semi-rigid composite joints, in accordance with EN 1994-1-1, if the reinforcing bars in the floor slab are continuous or anchored at the joints and some degree of rotational restraint can be provided.

The previous edition of the design guide[3] only applied to beam-to-beam composite joints, where both primary and secondary beams are designed to act compositely with the floor slab. It proposed a contact type of semi-rigid composite joint that can develop higher rotational stiffness and moment resistance. A mechanical model assumed that the tension force is transferred by reinforcing bars, and the compression force is transferred by contact plates inserted at the bottom flange of the steel beam to enhance rotational stiffness and moment resistance.

This new design guide extends its scope of application to include beam-to-beam joints, steel beams connecting to reinforced concrete primary beams, beam-to-column joints, and beam-to-reinforced concrete wall joints with contact plates to enhance semi-rigidity in the connections. The design methods for contact-type semi-rigid joints have been validated based on finite element analyses and full-scale tests conducted at the Steel Structures Research Laboratory at Nippon Steel Corporation in Japan.

The design guide aims to give structural engineers the confidence to use semi-rigid composite joints safely and economically in the design and construction of steel-concrete composite buildings. With this updated guide, designers can now expand their use of semi-rigid joints, potentially reducing construction time and costs while maintaining the necessary level of structural performance.

J Y Richard Liew
National University of Singapore
May, 2023

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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)

$b_{0,h}$ 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 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

$z_{st,eq\text{-}na}$ Equivalentvertical 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

ϕ_{sl1} Diameter of anti-crack longitudinal reinforcing bars (row 1)

ϕ_{sl2} Diameter of additional longitudinal reinforcing bars (row 2)

ϕ^*_{sl} Diameter of longitudinal reinforcing bars; Maximum diameter of longitudinal reinforcing bars

ϕ_{st1} Diameter of anti-crack transverse reinforcing bars (row 1)

ϕ_{st2} 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 One

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.

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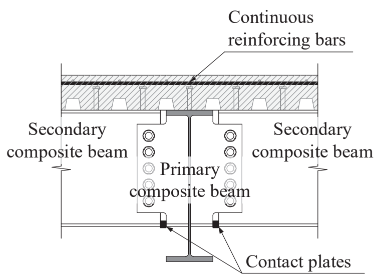
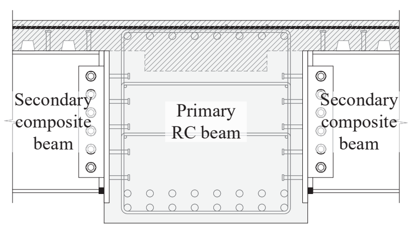
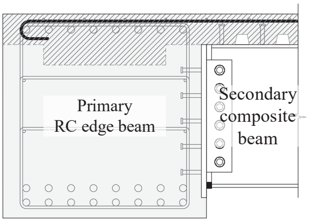
Table 1.1 Composite joint details with contact plates

<table>
<tr><th rowspan="2">Supporting members</th><th colspan="2">Composite beams with</th></tr>
<tr><th>Continuous reinforcing bars</th><th>Anchored reinforcing bars</th></tr>
<tr><td>Primary Composite beam</td><td>
</td><td></td></tr>
<tr><td>Primary RC beam</td><td>
</td><td>
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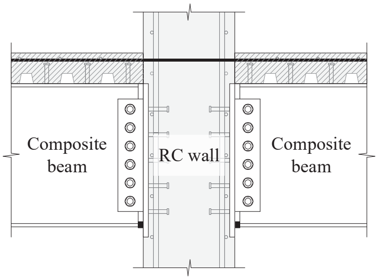
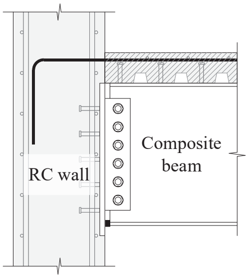
<tr><td>RC wall</td><td>
</td><td>
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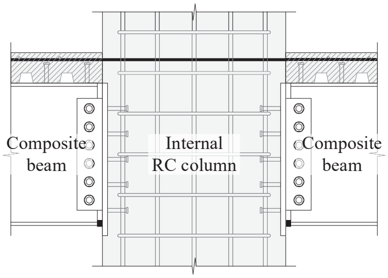
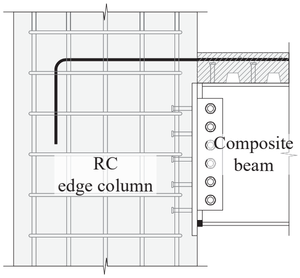
<tr><td>RC column</td><td>
</td><td>
</td></tr>
</table>

Chapter Two

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

	Nominal values of yield strength/ultimate tensile strength [N/mm^2] with thickness [mm] less than or equal to					
Steel grade	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.

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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_{ck} [N/mm^2]	20	25	30	35	40	45	50	55	60
Mean value of tensile strength f_{ctm} [N/mm^2]	$f_{ctm}\left(0.40+\frac{0.60\rho_{lc}}{2200}\right)$								
Secant modulus of elasticity E_{cm} [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:

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

α_{hs} is given by:

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

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

where

f_{hsu} is the ultimate strength of headed stud
d_{hs} is the diameter of the shank of headed stud, 16 [mm] $\leq d_{hs} \leq$ 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, γ_{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.

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

(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.

(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 Three

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.

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.

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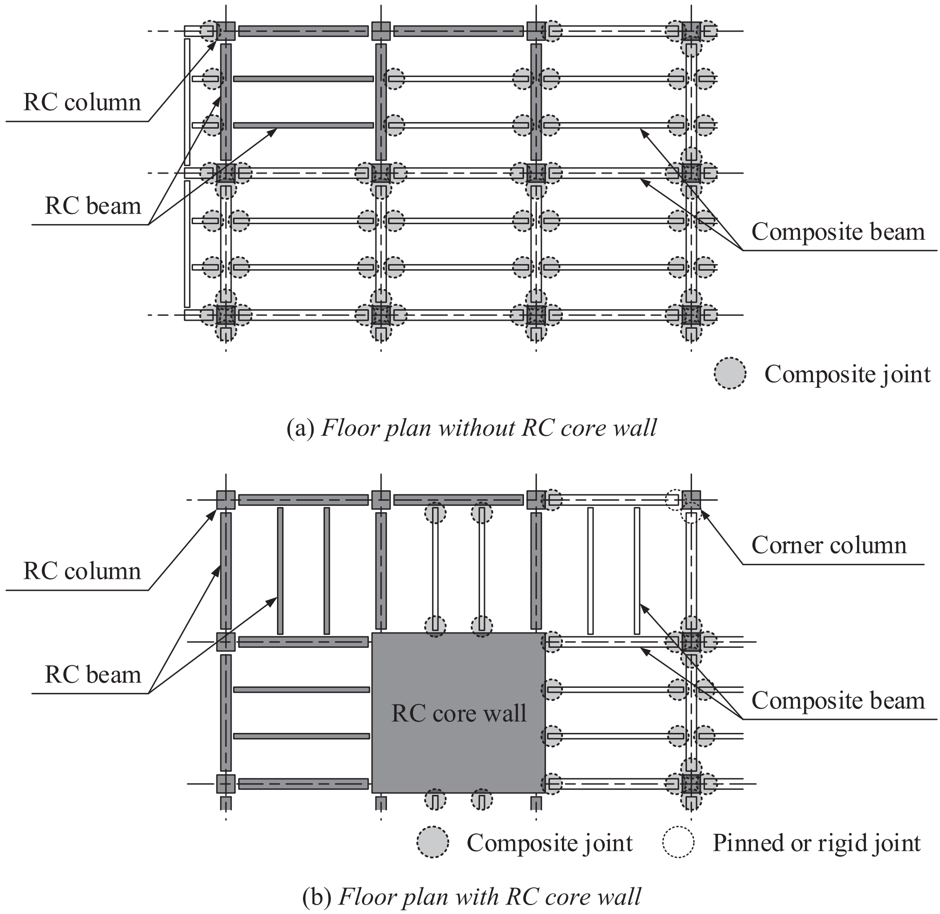


Figure 3.1 Floor plan showing composite joints

(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

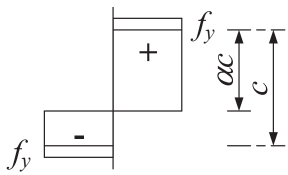
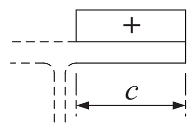
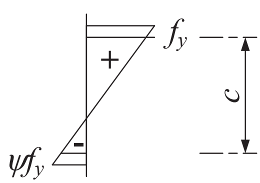
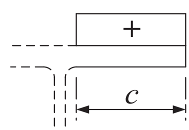
(1) Steel section

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

$\varepsilon = \sqrt{\frac{235}{f_{ay}}}$			
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

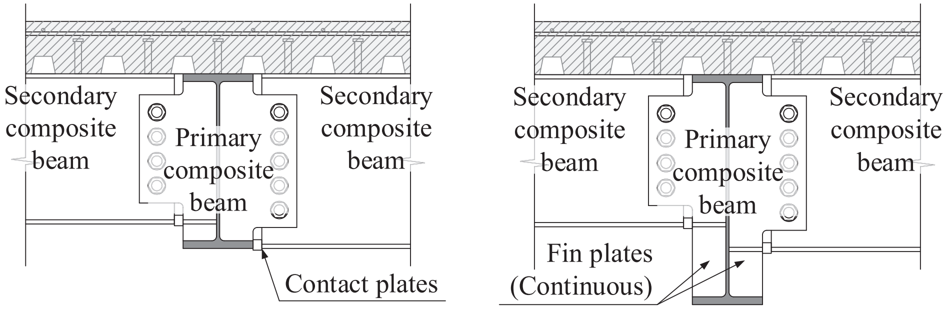
Beam-end joints should be designed as nominally pinned joint when the longitudinal reinforcing bars in the slab are not continuous or not properly anchored at beam-to-beam, beam-to-wall, and beam-to-column joints, in which case sufficient rotational restraint may not be developed at the joints.

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.



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

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

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.

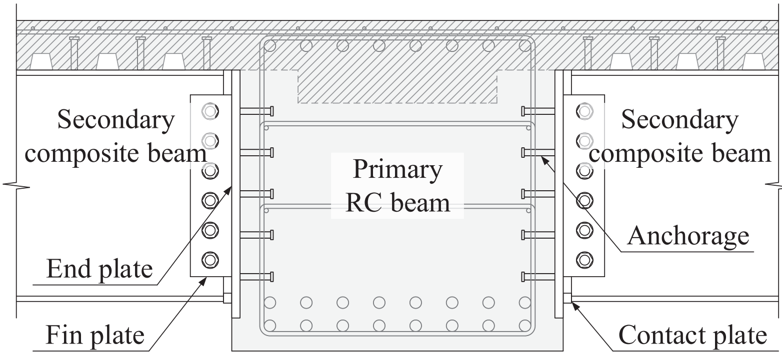


Figure 3.3 Beam-to-beam joint with primary reinforced concrete beam

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.

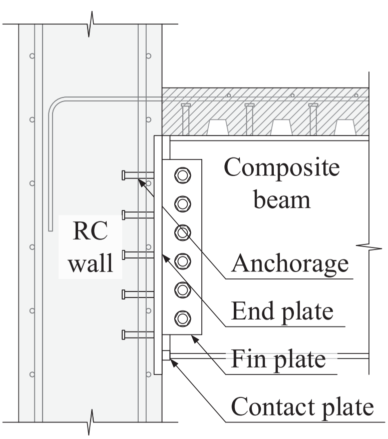


Figure 3.4 Beam-to-wall joint with reinforced concrete wall

d) *Beam-to-column joint with reinforced concrete column*
The joints may be either double-side or single-side connected but should not be a beam-to-corner column joint in which a reinforced concrete column is biaxially connected by steel beams. For single-side connected joint,

the moment due to eccentricity from the beam reaction force should be taken into consideration for the column design. As shown in Figure 3.5, fin plates should be welded to the end plates fixed to the reinforced concrete column by appropriate anchorages to transfer the shear force from the beams.

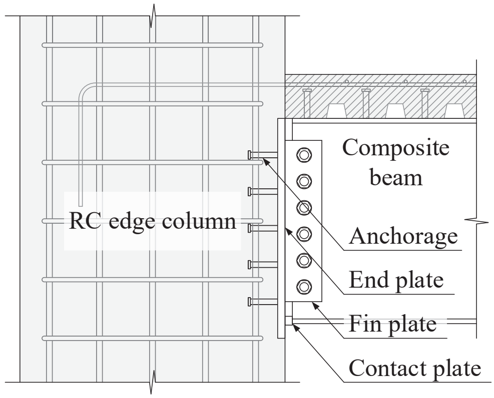


Figure 3.5 Beam-to-column joint with reinforced concrete column

(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.

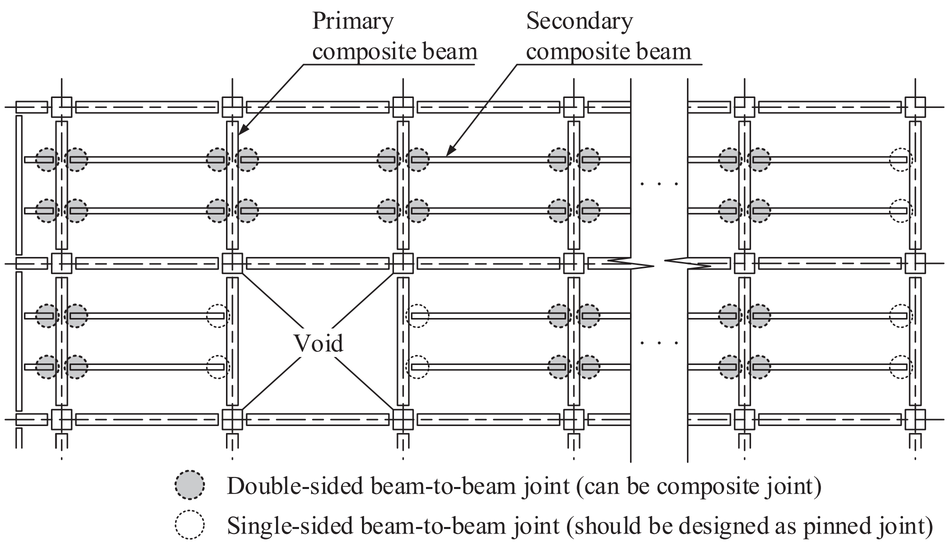


Figure 3.6 Floor beam layout with composite beams

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.

Chapter Four

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.

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.

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Authors: J Y Richard Liew, Yuichi Nishida & Masaki Arita
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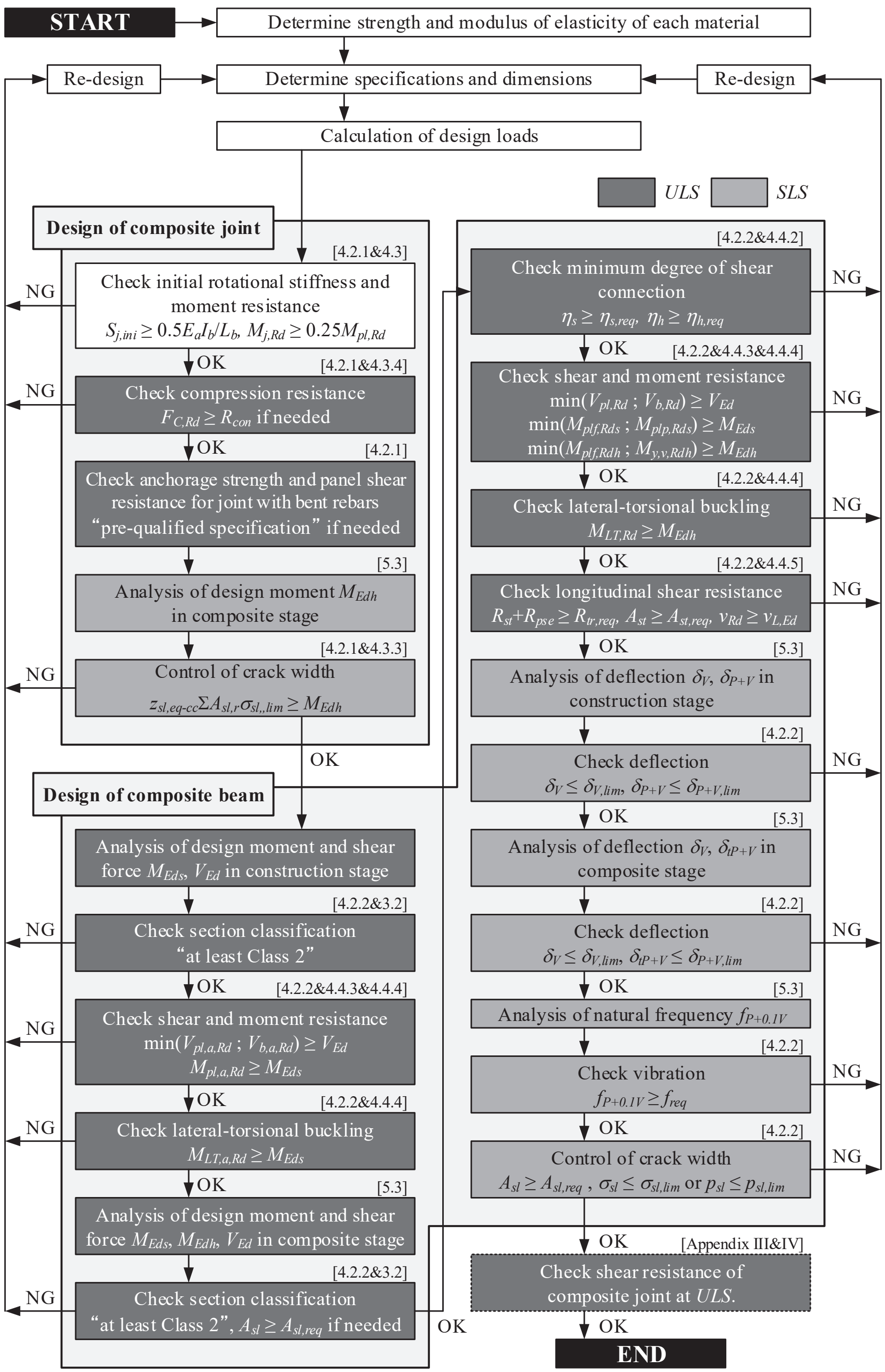

Figure 4.1 Design flowchart for composite joint and composite beam

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

The following conditions for initial rotational stiffness and moment resistance should be satisfied so that composite joints can be classified as semi-rigid.

a) *Initial rotational stiffness*

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

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*

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

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 prequalified specifications to prevent the anchorage failure and panel shear failure of supporting members. The prequalified specifications are shown in Tables 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 4.1 Pre-qualified specifications for **beam-to-wall composite joints** with bent reinforcing bars

Characteristic cylinder strength of reinforced concrete wall 40 [N/mm^2] $\leq f_{ck,cw} <$ 45 [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 wall thickness t_{cw} [mm]		
	$250 \leq t_{cw}$ (l_{dh}= 200 [mm])	$300 \leq t_{cw}$ (l_{dh}= 250 [mm])	$400 \leq t_{cw}$ (l_{dh}= 350 [mm])
Additional rebars only			10 - 100 or more 13 - 150 or more 16 - 200 or more
10 - 200 or more			Anti-crack rebars only 10 - 100 or more 13 - 200 or more
10 - 150 or more			Anti-crack rebars only 10 - 150 or more
10 - 100 or more			Anti-crack rebars only 10 - 200 or more
13 - 200 or more			
13 - 150 or more			Anti-crack rebars only
13 - 100 or more			
16 - 200 or more			Anti-crack rebars only
16 - 150 or more			
16 - 100 or more			

Table 4.1 (Continued)

<table>
<tr><th colspan="4">Characteristic cylinder strength of reinforced concrete wall
45 [N/mm^2] $\le f_{ck,cw} <$ 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] for wall thickness t_{cw} [mm]</th></tr>
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Table 4.1 (Continued)

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50 [N/mm^2] $\leq f_{ck,cw} <$ 55 [N/mm^2]</th></tr>
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Table 4.1 (Continued)

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<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>
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60 [N/mm^2] $\leq f_{ck,cw}$</th></tr>
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Table 4.2 Pre-qualified specifications for **beam-to-column composite joints** with bent reinforcing bars

<table>
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Table 4.2 (Continued)

Characteristic cylinder strength of reinforced concrete column 60 [N/mm^2] $\leq f_{ck,cc}$			
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]		
	$300 \leq D_{cc}$ (l_{dh}= $2/3D_{cc}$)	$375 \leq D_{cc}$ (l_{dh}= $2/3D_{cc}$)	$525 \leq D_{cc}$ (l_{dh}= $2/3D_{cc}$)
Additional rebars only	10 - 100 or more 13 - 150 or more	10 - 100 or more 13 - 150 or more 16 - 200 or more	10 - 100 or more 13 - 100 or more 16 - 150 or more
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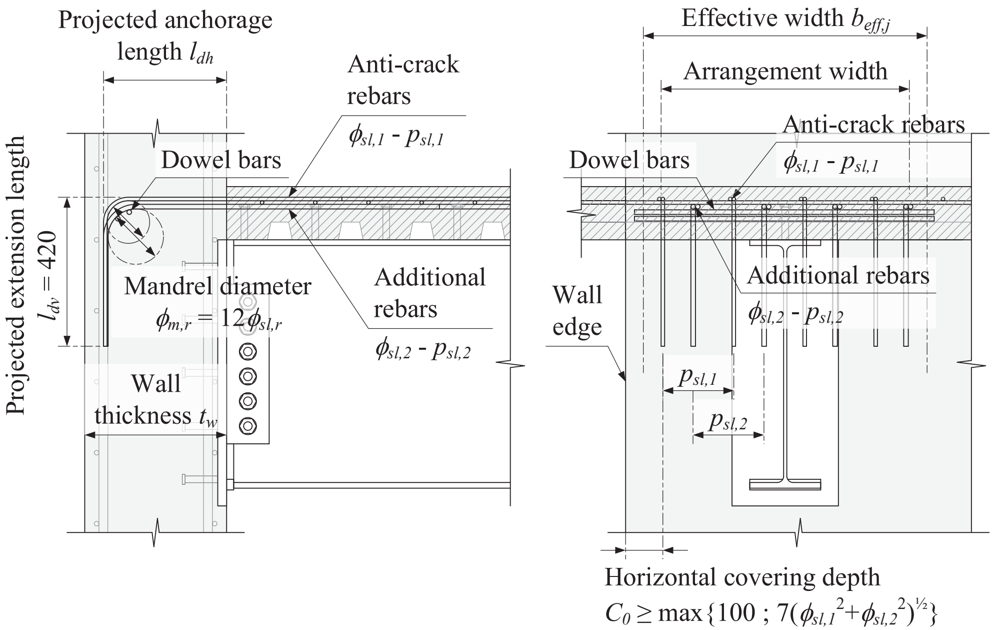
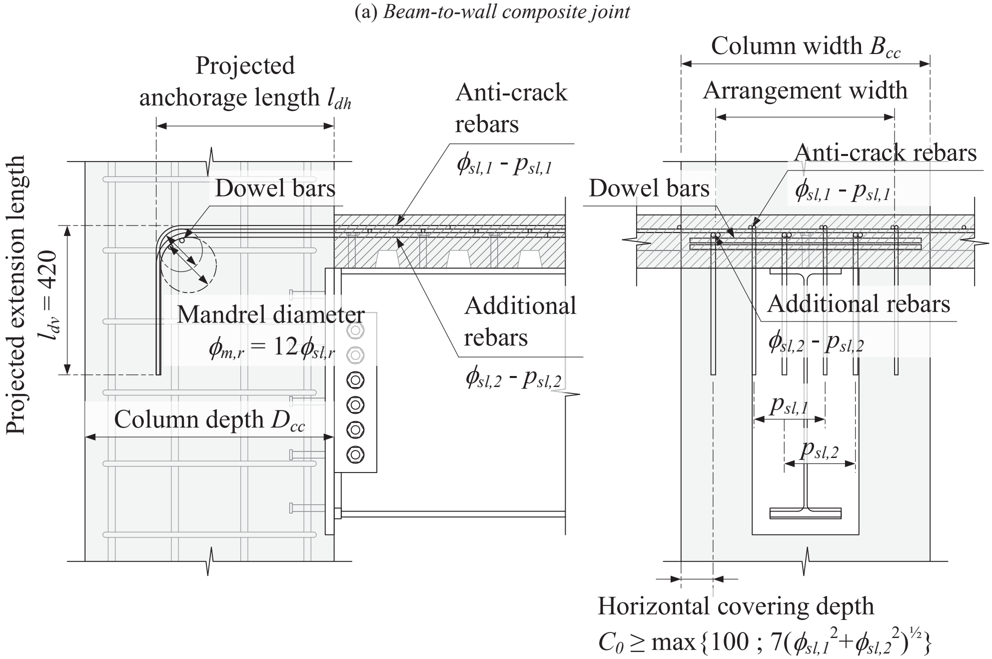


(a) *Beam-to-wall composite joint*



(b) *Beam-to-column composite joint*

Figure 4.2 Dimensions specified in pre-qualified specifications

(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 Tables 4.3 and 4.4

M_{Edh} is the design hogging moment

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	–

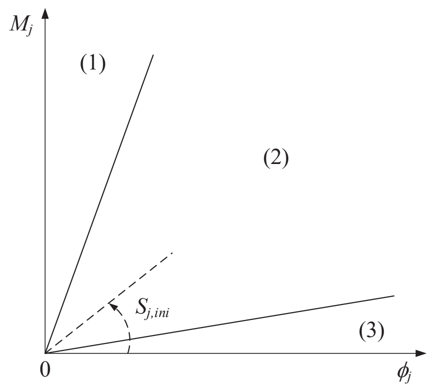
Table 4.4 Limit of stress permitted in longitudinal reinforcing bars based on spacing

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

There are various definitions of semi-rigid joints for the structural analysis, and one of the representative joint classification schemes is described in EN 1993-1-8. According to EN 1993-1-8, joints are classified as rigid joints, nominally pinned joints, or semi-rigid joints by comparing the initial rotational stiffness of the joints $S_{j,ini}$ with the boundaries shown in Figure 4.3. Here, $S_{j,ini}$ is taken as the bending moment per unit rotation, and the classification boundaries are related to the flexural stiffness of the beam member $(EI/L)_b$ adjacent to the joint. In this figure, the joints not classified as either rigid joints or nominally pinned joints, that is, the joints which have the initial rotational stiffness lower than the boundary of rigid joints but higher than that of nominally pinned joints are defined as semi-rigid joints.



(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)

In addition, joints are also classified as full-strength joints, nominally pinned joints, or partial-strength joints by comparing the moment resistance of the joint $M_{j,Rd}$ with the plastic moment resistance of the beam members $M_{pl,Rd}$ adjacent to the joints. If $M_{j,Rd} \geq M_{pl,Rd}$, the joint is classified as full-strength joints, and if $M_{j,Rd} < 0.25M_{pl,Rd}$, the joint is classified as nominally pinned joints, and otherwise, the joint is classified as partial-strength joints.

In this design guide, it is assumed that composite beams are designed with semi-rigid ends. Therefore, composite joints should satisfy the following conditions for the initial rotational stiffness and the moment resistance based on the above joint classification scheme.

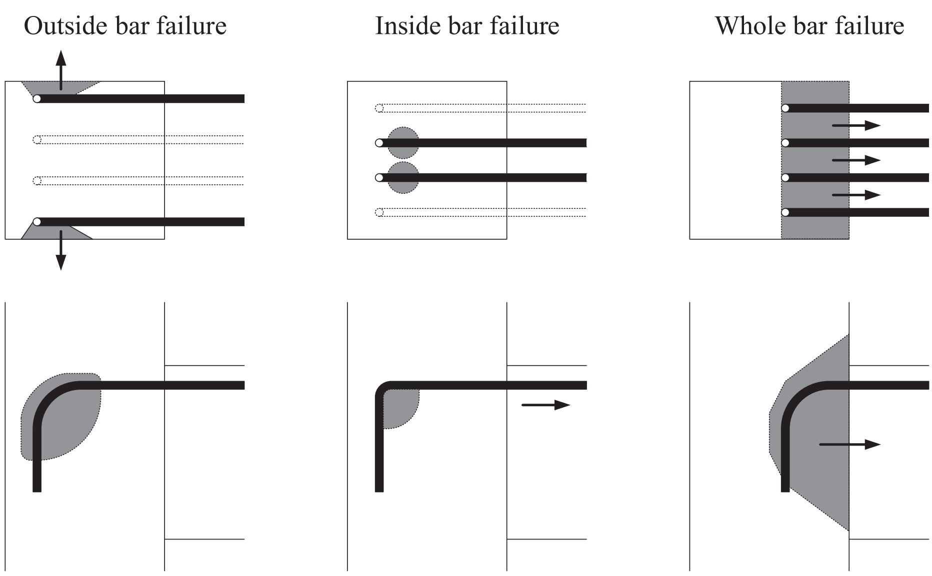
$$S_{j,ini} \geq \frac{0.5E_a I_b}{L_b} \tag{4.6}$$

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

(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 prequalified



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

Figure 4.4 Anchorage failure mode using bent reinforcing bars 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.

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 prequalified specifications in Tables 4.1 and 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 ofsteel 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 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:

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

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

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

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 ofcomposite 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*

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

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

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

$R_{trr,eq}$ 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 incomposite 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

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

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

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

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

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	–

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

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 3.3.

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.

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

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

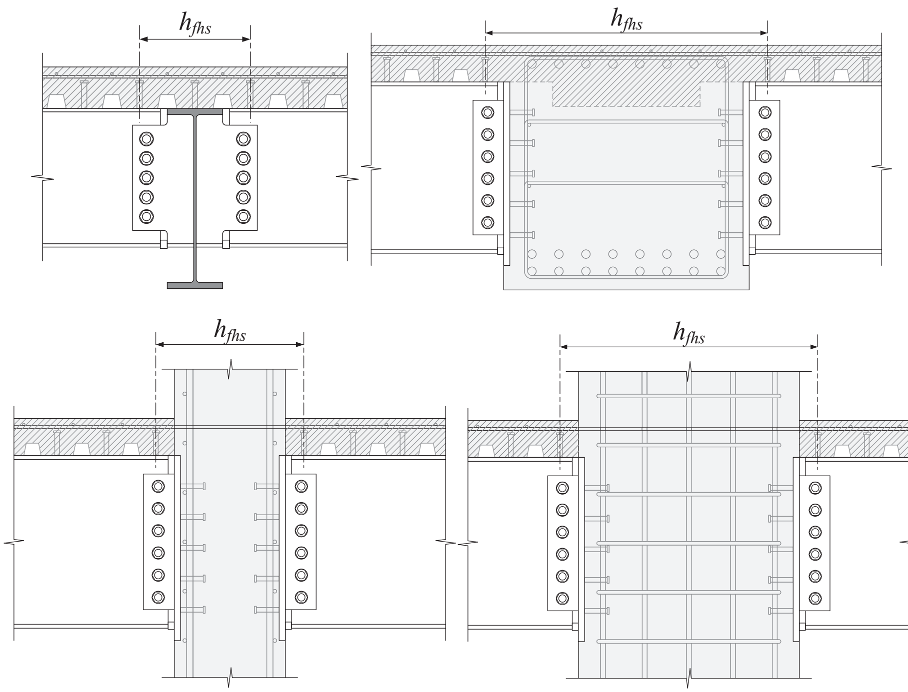
where

h_{fhs} is the distance between first headed studs on opposite composite beams, see Figure 4.5

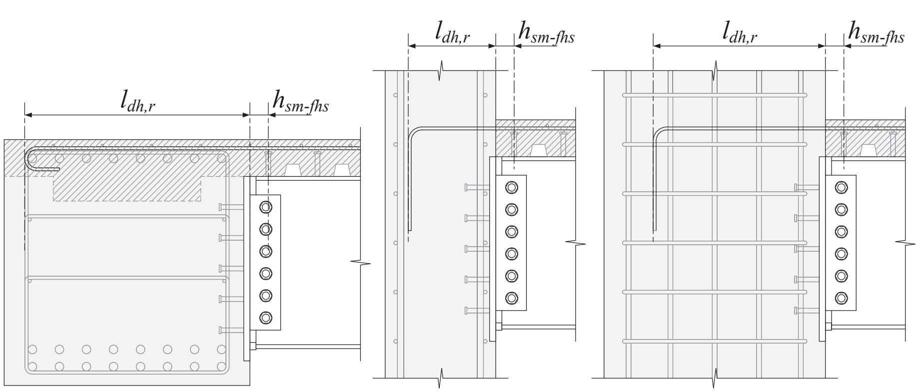
$\phi_{sl,r}$ is the diameter of longitudinal reinforcing bars for a row r

$l_{dh,r}$ 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:

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

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

where

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

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

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

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

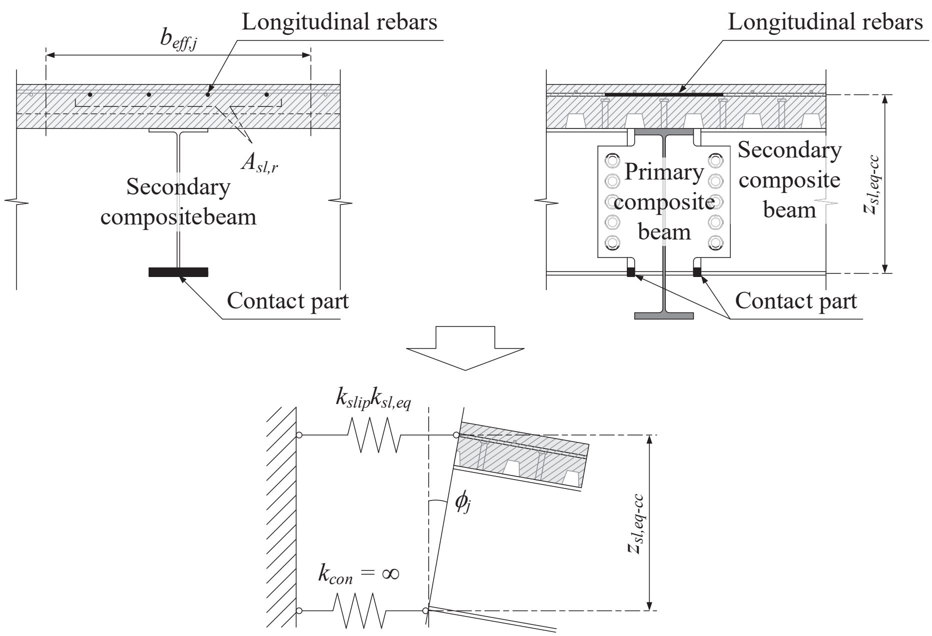
$l_{eff,r}$ is the effective length of reinforcement, see 4.3.1

K_{sc} is the stiffness related to headed studs

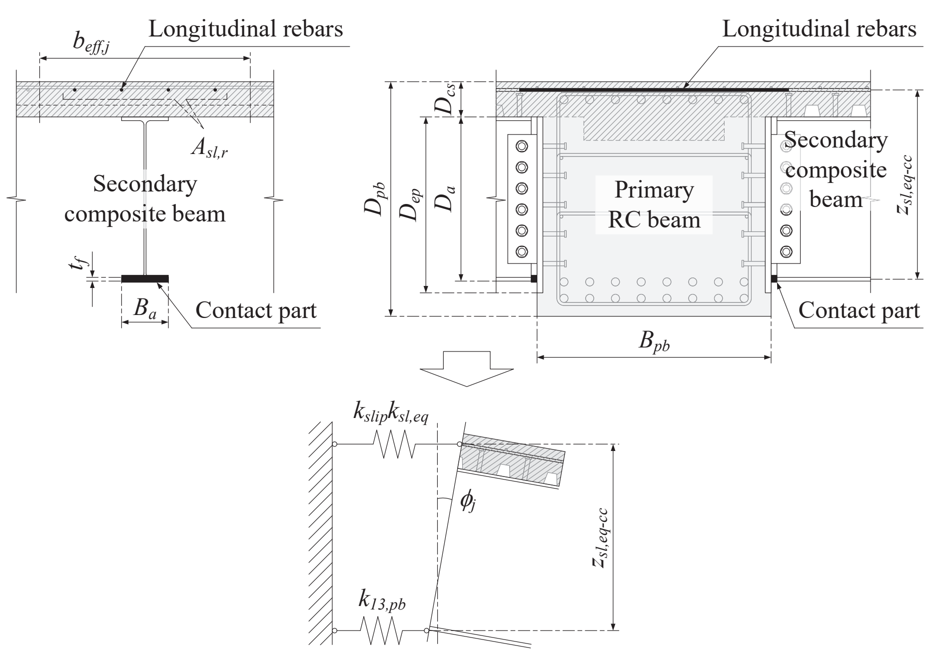
N is the number of headed studs distributed over length l

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]

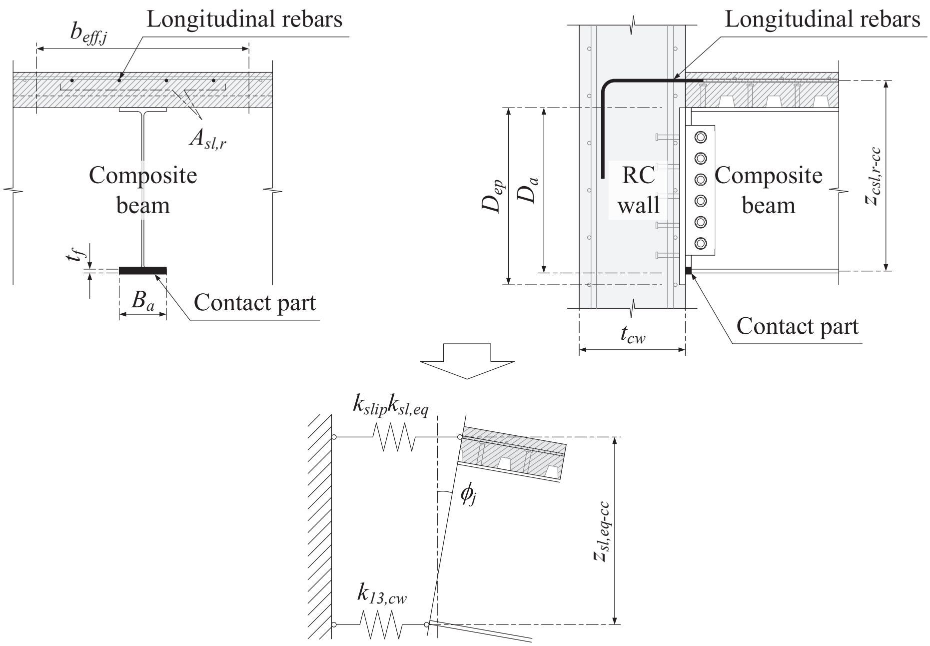


(a) *Beam-to-beam composite joint with primary composite beam*

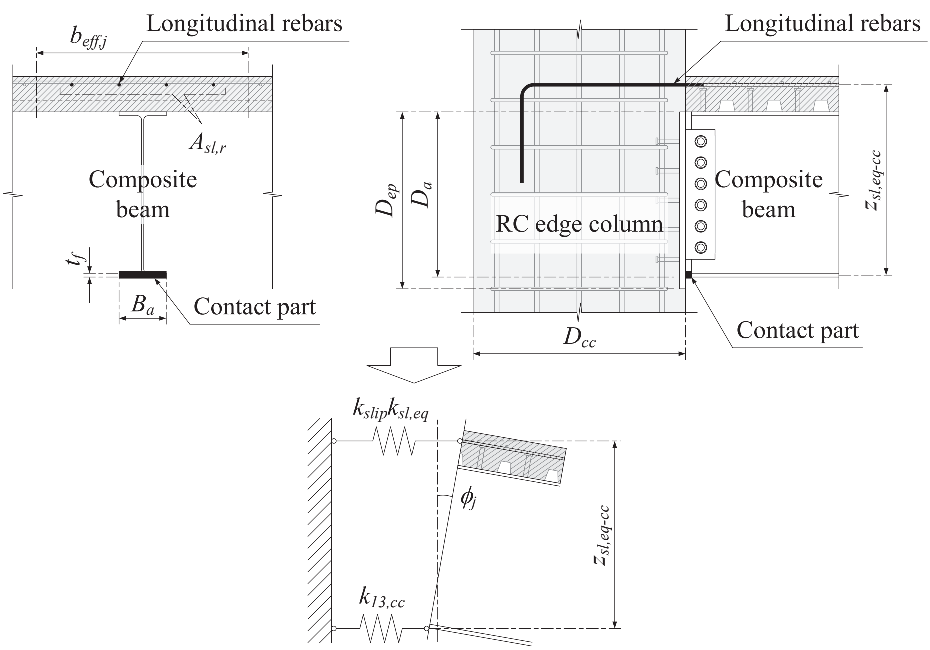


(b) *Beam-to-beam composite joint with primary reinforced concrete beam*

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 *Continued*

$z_{sl,eq\text{-}ca}$ 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.

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

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

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

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

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

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

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

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

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

α is given by Eq. (4.9) using convergence calculation, but it should be 3.0 if the calculated value is more than 3.0.

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

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.

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

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

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

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

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

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

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

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

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

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

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.

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

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

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

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

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

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

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

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

α is given by Eq. (4.16) using convergence calculation, but it should be 3.0 if the calculated value is more than 3.0.

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

where

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

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

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

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

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.

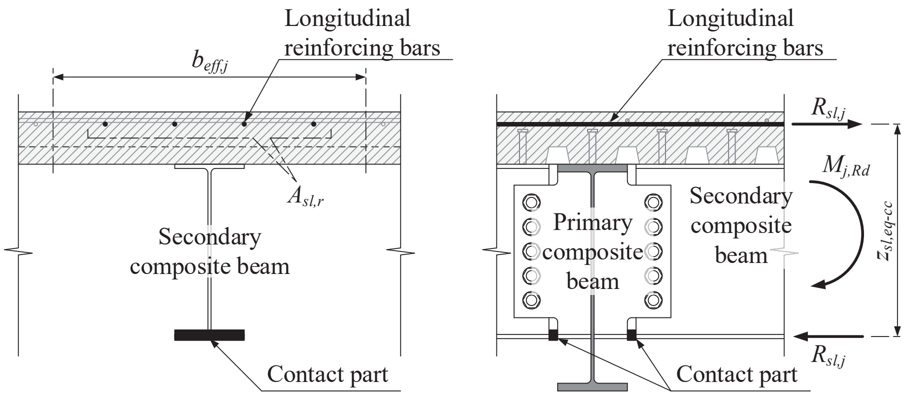
4.3.3 Moment resistance

(1) Concept

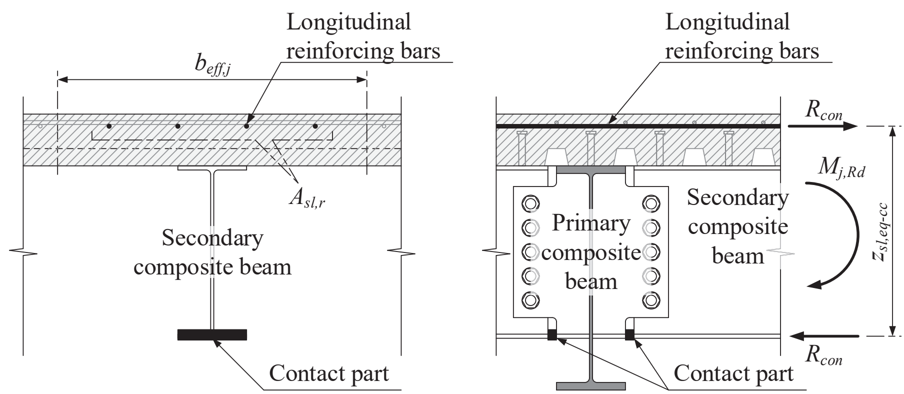
The moment resistance of composite joints should be evaluated considering the longitudinal reinforcing bars in tension and the contact parts in compression as the basic joint components. Concrete in tension shall be neglected.

(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



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



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

Figure 4.7 Modelling of composite joint for moment resistance

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

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 ofcomposite 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

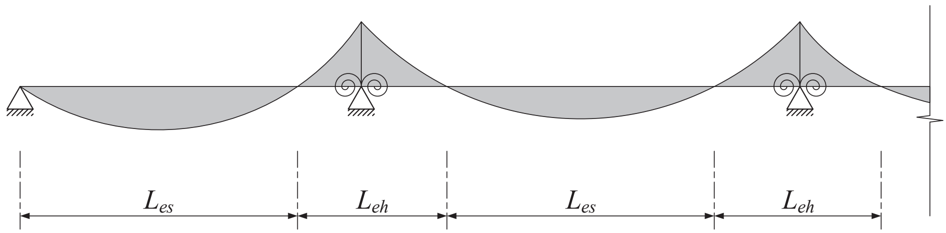


Figure 4.8 Distance between inflection points

(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.

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

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

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

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 ofsteel 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

(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

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

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

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

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

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:

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

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

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

$$V_{b,a,Rd} = \min\left\{\frac{\chi_w f_{wy}\left(D_a - 2t_f\right) t_w}{\sqrt{3}\gamma_a}; \frac{1.2 f_{wy}\left(D_a - 2t_f\right) t_w}{\sqrt{3}\gamma_a}\right\}$$

$$\text{for } \frac{\left(D_a - 2t_f\right)}{t_w} > \frac{72}{1.2}\sqrt{\frac{235}{f_{ay}}} \tag{4.130}$$

χ_w is given by:

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

$$\chi_w = \frac{0.83}{\lambda_w} \quad \text{for} \quad \frac{0.83}{1.2} \leq \lambda_w \tag{4.132}$$

where

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

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

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

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

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

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

$\chi_{LT,a}$ 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**>

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

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

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

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

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

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

<**Effective Class 2 cross-sections**>

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

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

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

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

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

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

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

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

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

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

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

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>

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

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

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

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

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

<**Effective Class 2 cross-sections**>

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

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

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)

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

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

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

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

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

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

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 ofcomposite 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

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

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

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

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

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

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

where

$$R_f = B_a t_f f_{ayd} \tag{4.171}$$

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).

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

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

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

$I_{cs2}, \chi_{LT}, \lambda_{LT}$, and k_s are given by:

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

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

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

where

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

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

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

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

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

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

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

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

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

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

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

$z_{\text{na-ccs},c}$ 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

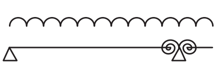
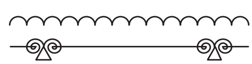
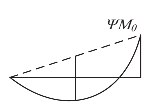
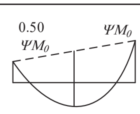
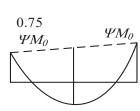
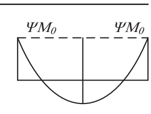
L_{cr} is the length of composite beam between points at which bottom flange is laterally restrained

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

Table 4.9 Values of factor C_4 for spans with transverse loading

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

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

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 εt_w adjacent to the compression flange measured from the base of the root radius, with another part of 20 εt_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 εt_w adjacent to the compression flange measured from the base of the weld.

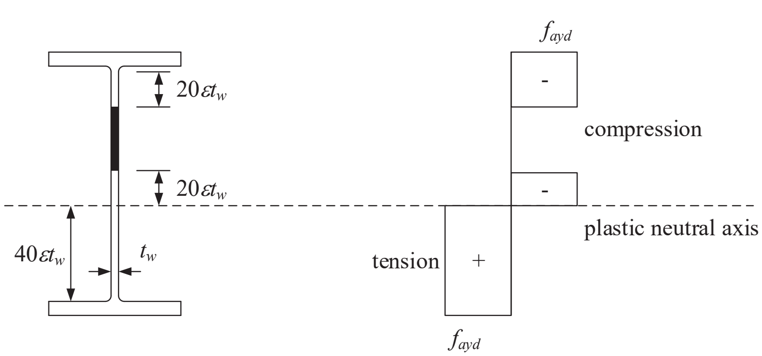


Figure 4.9 Effective Class 2 web

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, $26.5^\circ \leq \theta \leq 45^\circ$ for concrete flange in compression $38.6^\circ \leq \theta \leq 45^\circ$ for concrete flange in tension

Chapter Five

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 service ability limit states.

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(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).

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

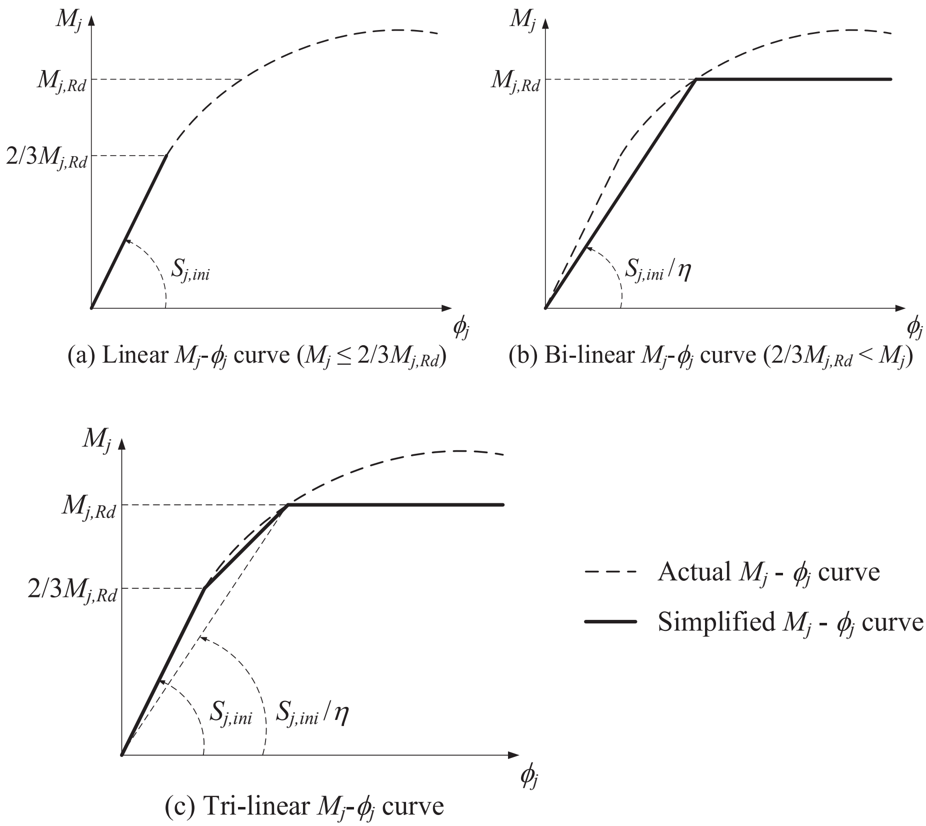


Figure 5.1 Simplified moment-rotation characteristics (M_j-ϕ_j curves)

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*

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

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

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

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

where

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

$$(EI)_s = E_a\left\{\frac{A_a\left(D_{cs} + D_{ps} + D_a\right)^2}{4\left(1 + \frac{n_0 A_a}{b_{effs}h_{cs}}\right)} + \frac{b_{effs}h_{cs}^3}{12\left(\frac{2E_a}{E_{cm,cs}}\right)} + I_{ay}\right\} \tag{5.5}$$

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

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

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

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

$M(x)$ 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*

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

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

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

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

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

where

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

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

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

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

$$\begin{aligned}\theta_{pin} = & -\frac{1}{(EI)_h L_b}\left(E(L_b) - D(x_0)L_b + F(x_0)\right) \\ & - \frac{1}{(EI)_s L_b}\left(D(x_0)L_b - F(x_0)\right)\end{aligned} \tag{5.18}$$

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*

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

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

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

$$\begin{aligned}&\frac{1}{(EI)_{h,l}}G(x_0) + \frac{1}{(EI)_s}\left(G(x_0') - G(x_0)\right) + \frac{1}{(EI)_{h,r}}\left(G(L_b) - G(x_0')\right) \\ &\qquad + \frac{M_{j,l}}{S_{j,l}} + \frac{M_{j,r}}{S_{j,r}} = 0\end{aligned} \tag{5.21}$$

$$\left\{\begin{array}{l}\dfrac{1}{(EI)_{h,l}}\left(G\left(x_0\right)L_b - I\left(x_0\right)\right)+\dfrac{1}{(EI)_s}\left(G\left(x_0'\right)L_b - G\left(x_0\right)L_b - I\left(x_0'\right)+I\left(x_0\right)\right)\\ \qquad +\dfrac{1}{(EI)_{h,r}}\left(H\left(L_b\right)-G\left(x_0'\right)L_b+I\left(x_0'\right)\right)+\dfrac{M_{j,l}}{S_{j,l}}L_b\end{array}\right\}$$

$$= 0 \tag{5.22}$$

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

where

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

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

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

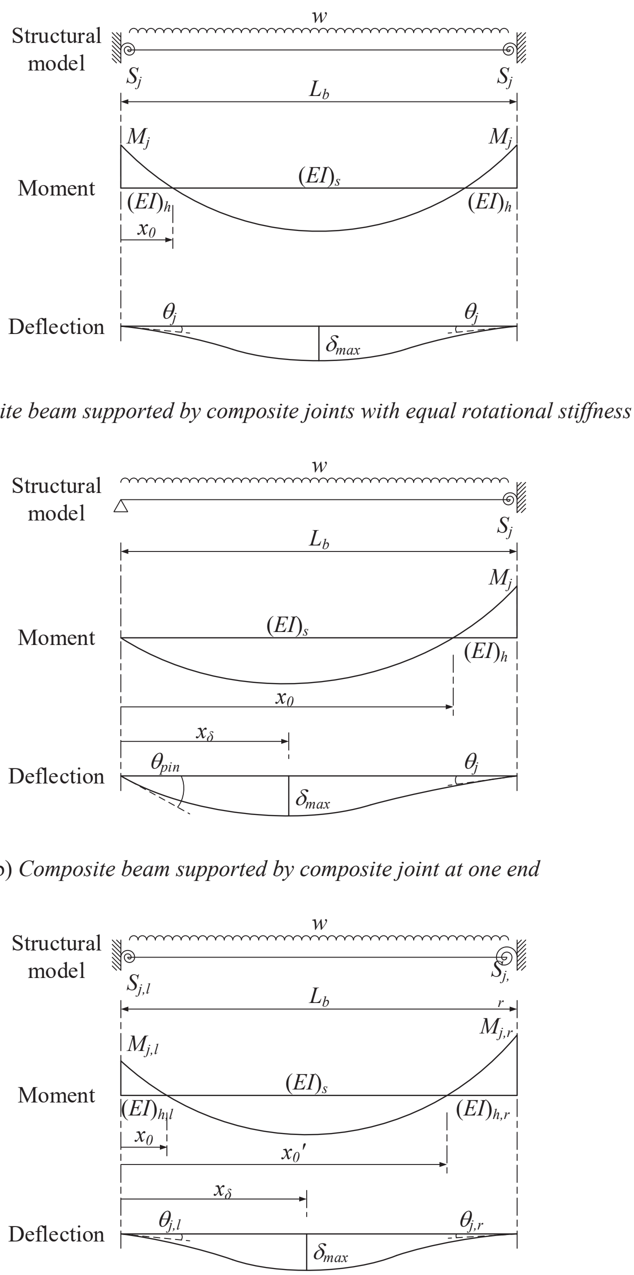
$$x_0=\frac{1}{w}\left\{\left(\frac{wL_b}{2}+\frac{M_{j,l}-M_{j,r}}{L_b}\right)-\sqrt{\left(\frac{wL_b}{2}+\frac{M_{j,l}-M_{j,r}}{L_b}\right)^2-2wM_{j,l}}\right\} \tag{5.27}$$

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

$S_{j,l}$ 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

(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



(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 join with unequal rotational stiffness*

Figure 5.2 Design moment and deflection

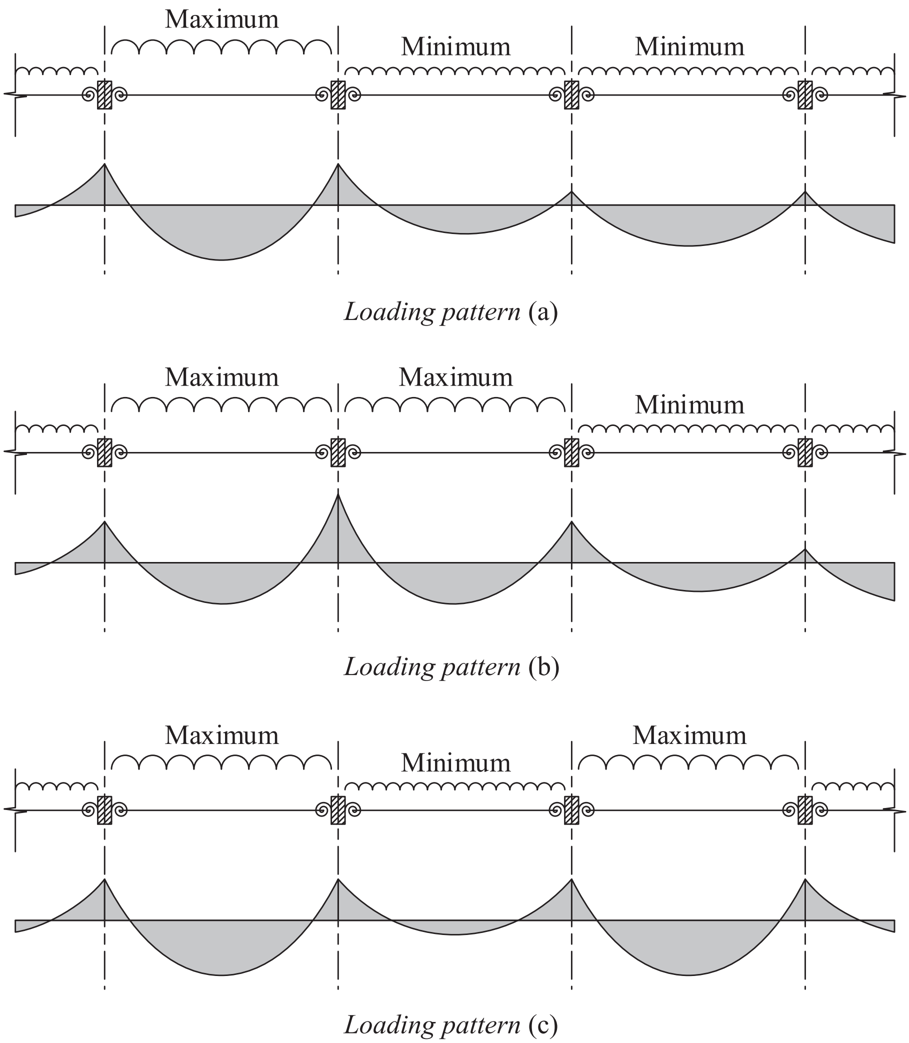
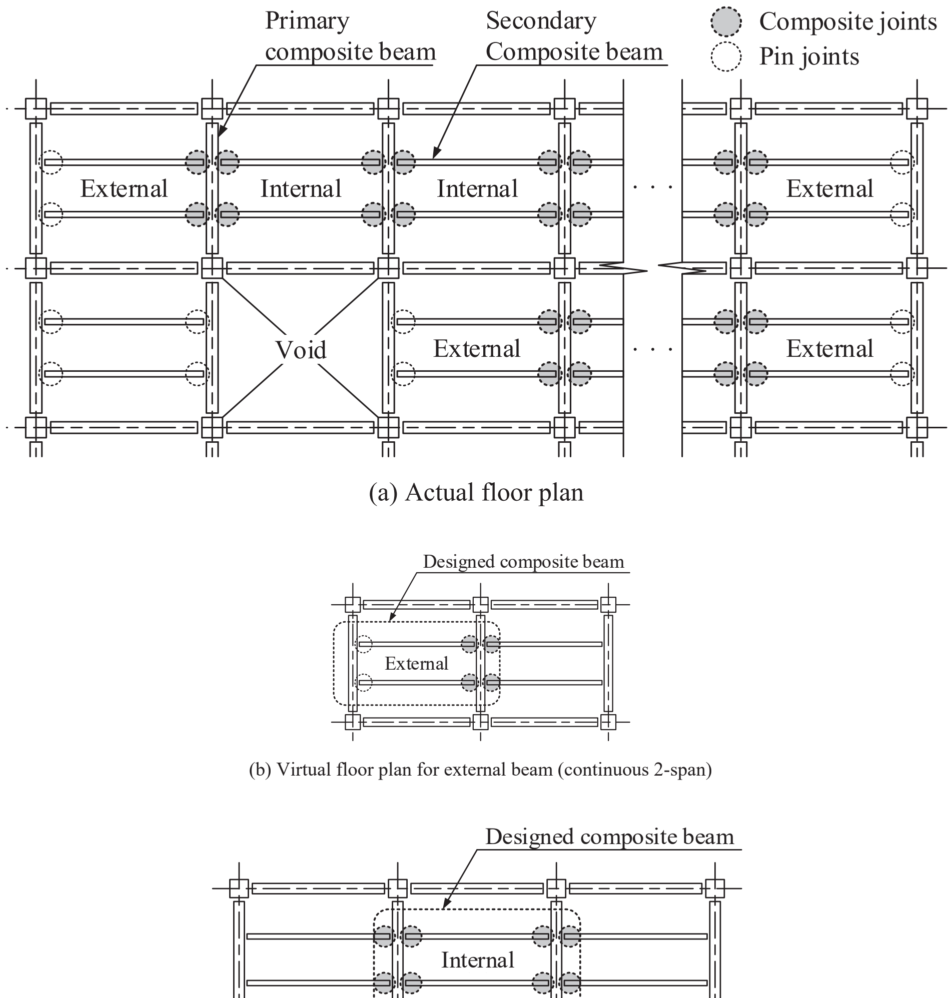


Figure 5.3 Moment distribution in response to loading patterns



(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

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**.

Chapter Six

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

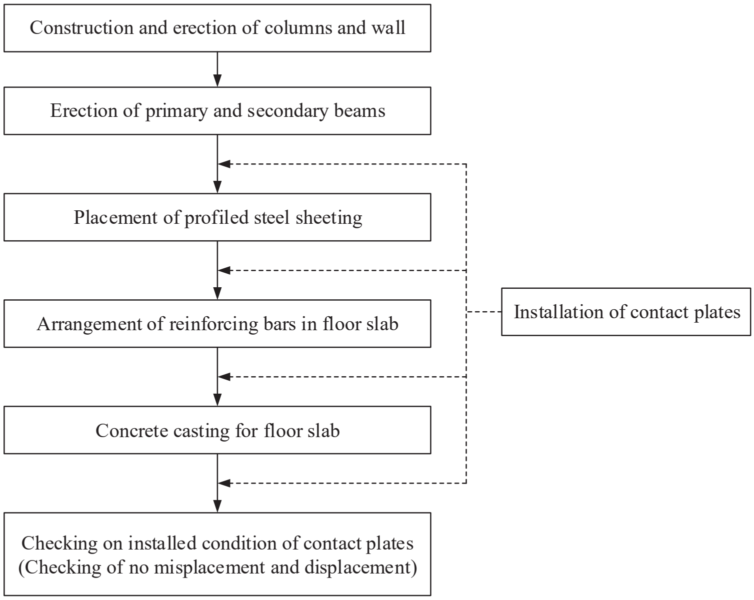


Figure 6.1 Installation of contact plate during construction

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plates can be installed at anytime 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.

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 are liable 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

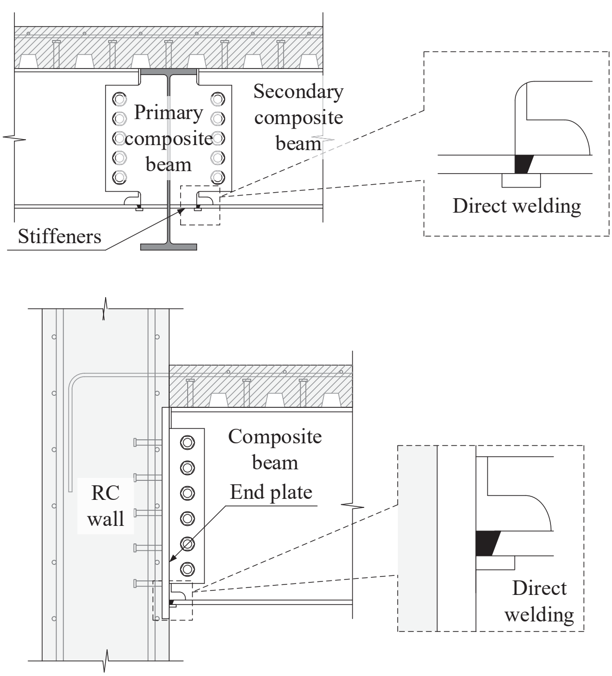
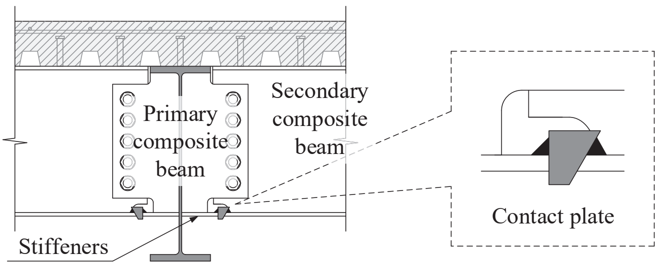
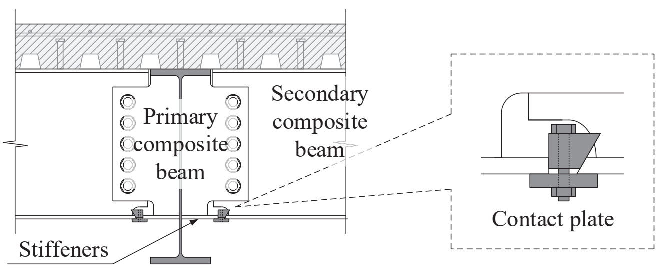


Figure 6.2 Direct welding through gap

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



(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.

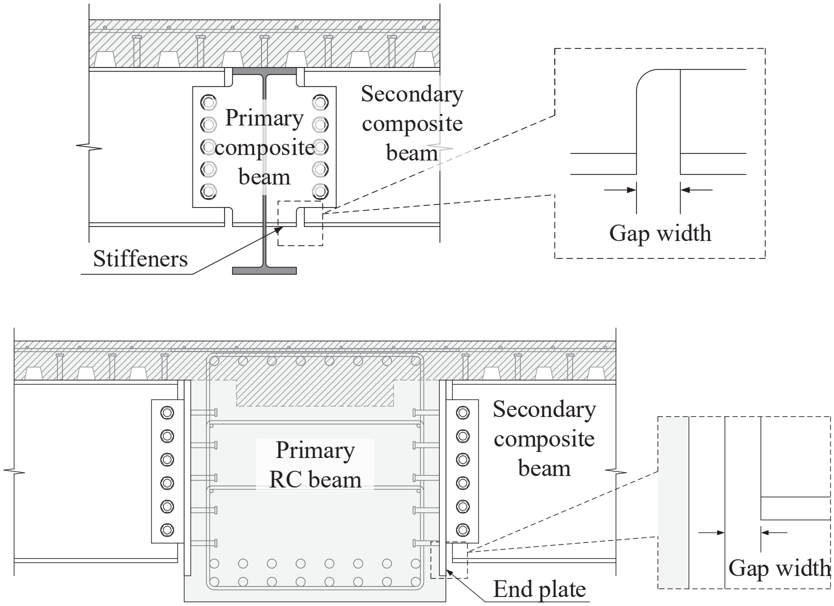


Figure 6.4 Gap width

(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.

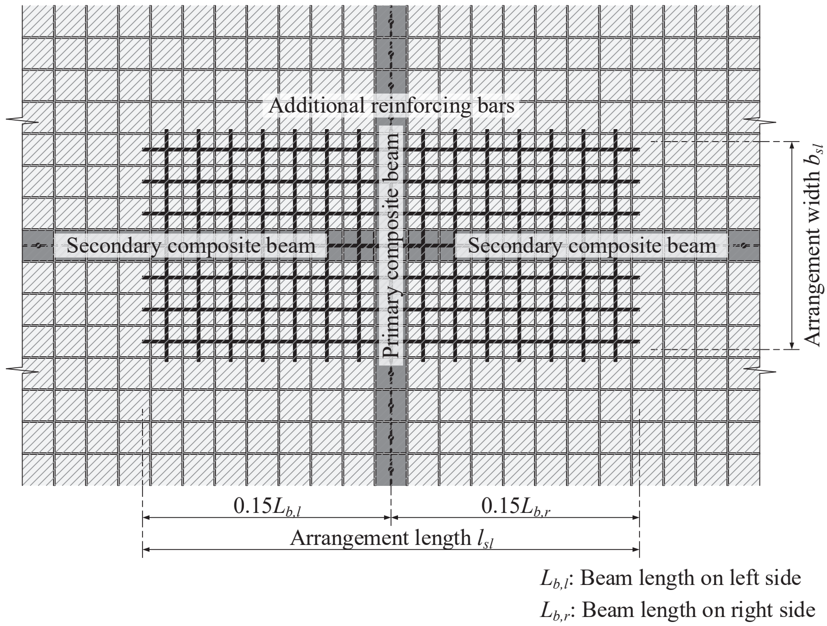
6.2.2 Reinforcing Bars in Concrete Slab

(1) Concrete cover

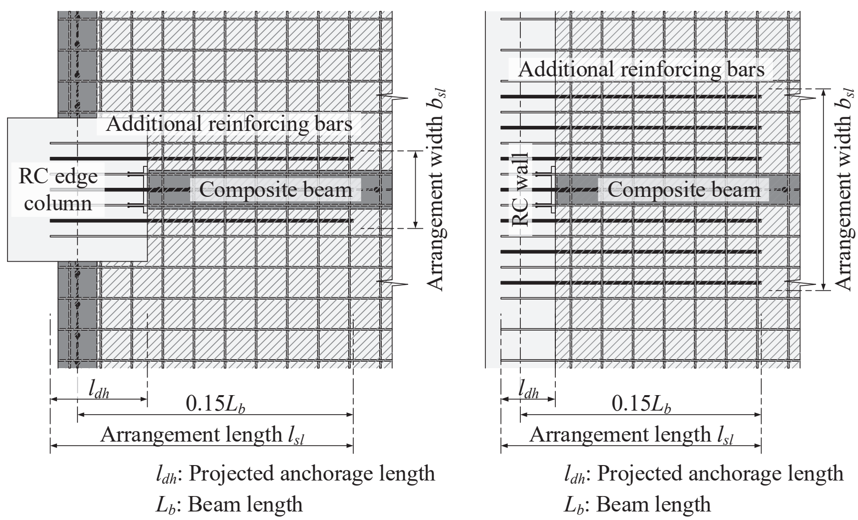
The concrete cover for the reinforcing bars in concrete slab should be more than or equal to the nominal cover c_{nom} specified in EN 1992-1-1. This value takes into account the acceptable negative deviation from the required minimum cover c_{min}.

(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



(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

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, $\alpha_1, \alpha_2, \alpha_3, \alpha_5, \alpha_6$, and l_b should be referred to EN1992-1-1.

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

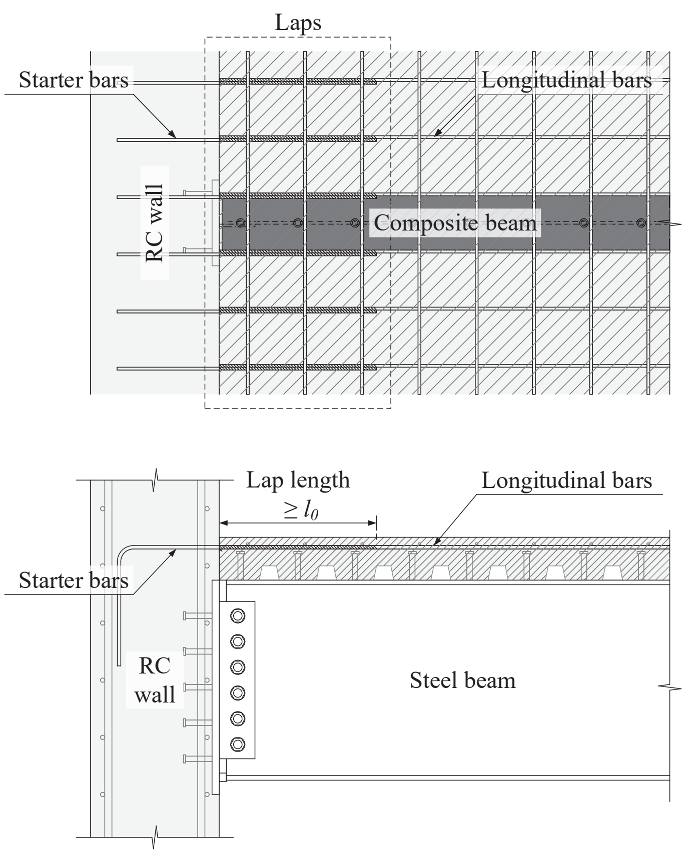


Figure 6.6 Laps of starter bars and longitudinal bars

$l_{0,min}$ 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

(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 Tables 4.1 and 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.

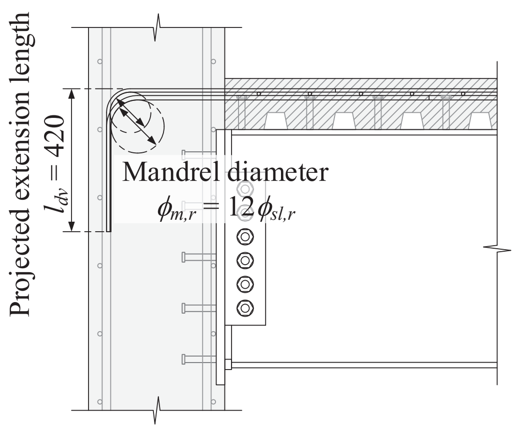


Figure 6.7 Mandrel diameter and projected extension length for pre-qualified specifications

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

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°	SD295	$\phi_{sl} \leq 16$ [mm]	$3\ \phi_{sl}$
135°	SD345	19 [mm] $\leq \phi_{sl} \leq$ 41 [mm]	$4\ \phi_{sl}$
90°	SD390	$\phi_{sl} \leq 41$ [mm]	$5\ \phi_{sl}$
90°	SD490	$\phi_{sl} \leq 25$ [mm]	
		29 [mm] $\leq \phi_{sl} \leq$ 41[mm]	$6\ \phi_{sl}$

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.

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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:

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

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

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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

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

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

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

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

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

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

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.

$$V_{jp} = \kappa \phi F_{jp} b_{jp} l_{dh} \tag{AI.9}$$

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

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

$$b_{jp} = B_{ep} + \frac{l_{dh}}{2} \tag{AI.11}$$

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.

(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

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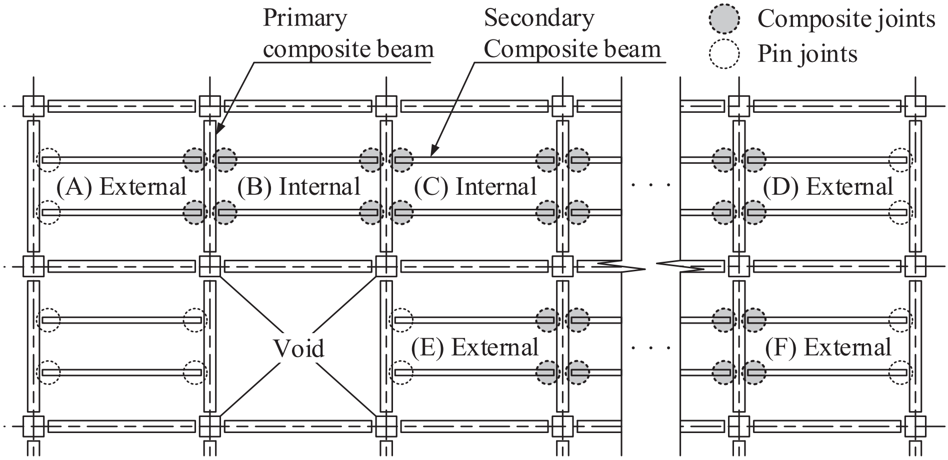
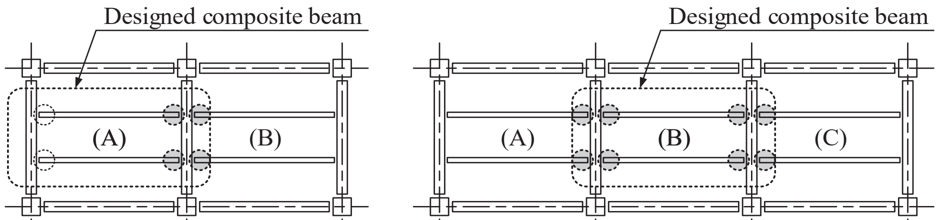


Figure AII.1 Typical composite floor plan

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).



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

Figure AII.2 Virtual floor plans in simplified analysis method

(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

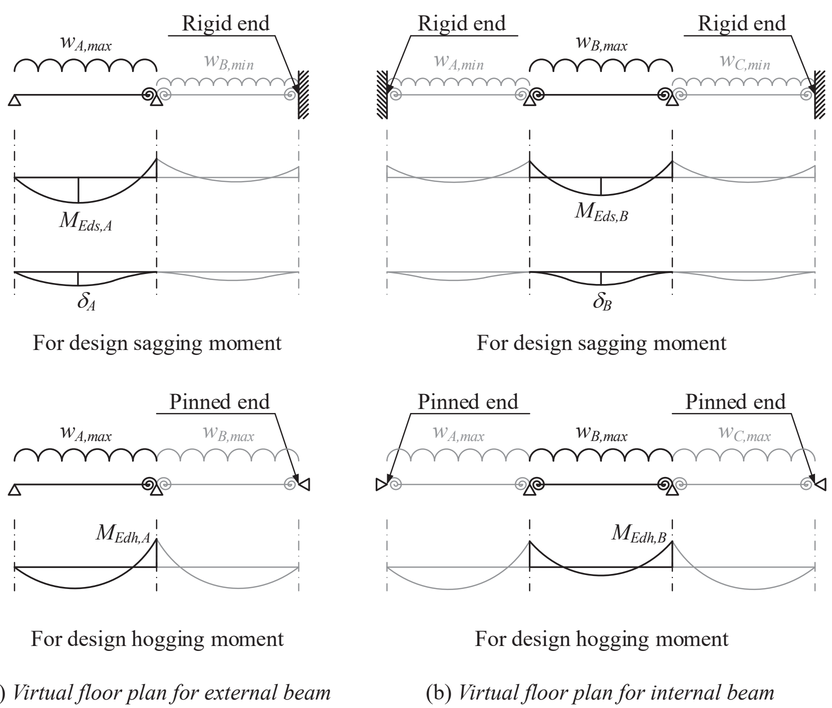


Figure AII.3 End-restraint conditions and critical loading patterns beams are maximized and minimized respectively. Also, the hogging moment may be maximized when both ends are assumed to be pinned ends and the distributed loads on all the composite beams are maximized. Incidentally, in both the virtual 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.

(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.

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

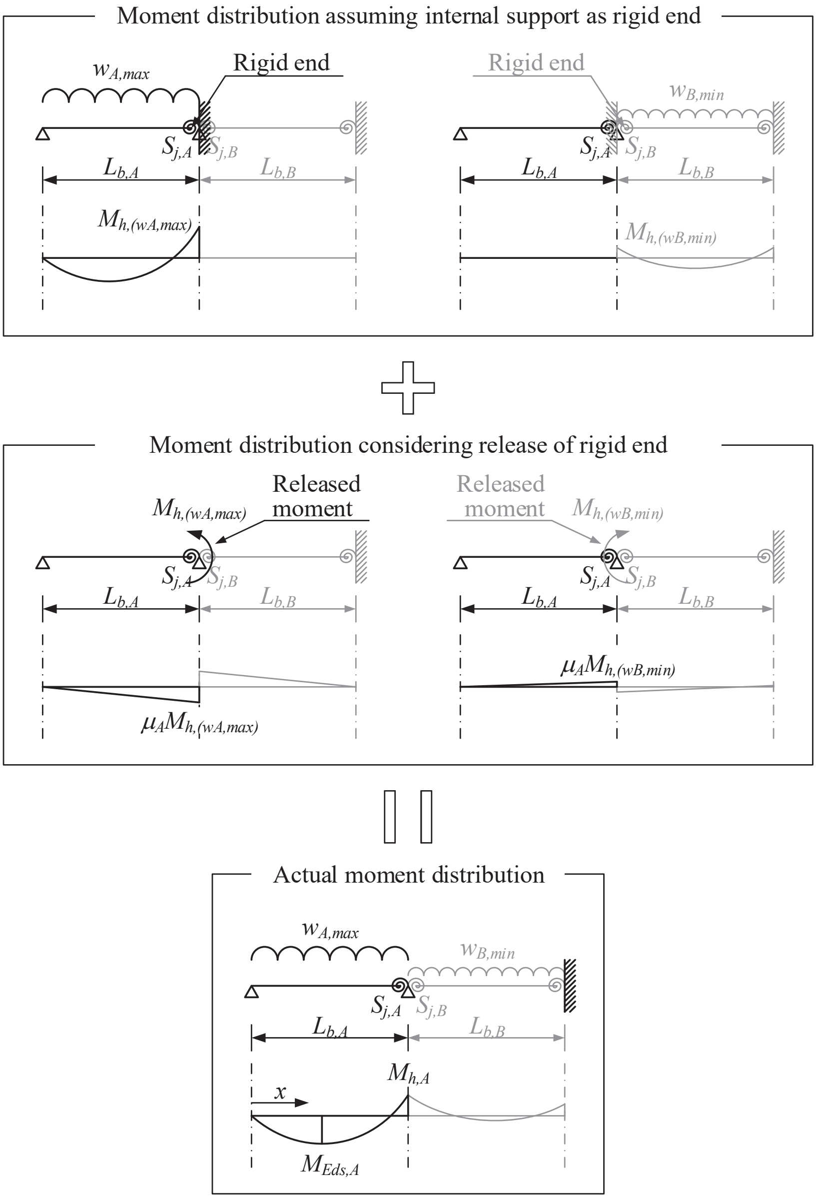


Figure AII.4 Principle of moment distribution method

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.

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 service ability criteria.

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.

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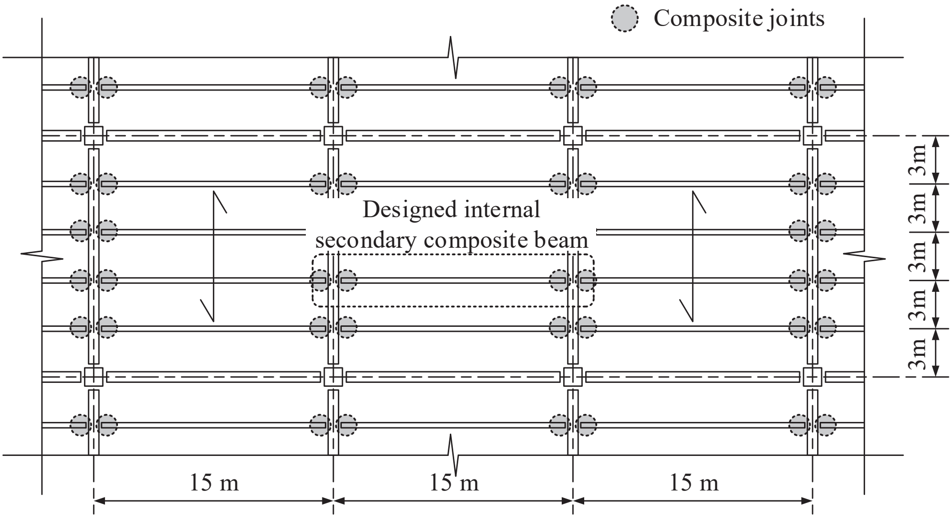


Figure AIII.1 Composite floor plan

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])

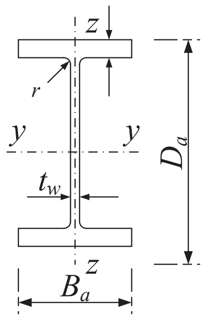


Figure AIII.2 Cross-section of steel beam

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: $\gamma_a = 1.00$

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]

[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: $\gamma_{ps} = 1.00$

Design yield strength: $f_{psd} = f_{psk}/\gamma_{ps} = 550$ [N/mm^2]

[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: $\gamma_{c,cs} = 1.50$

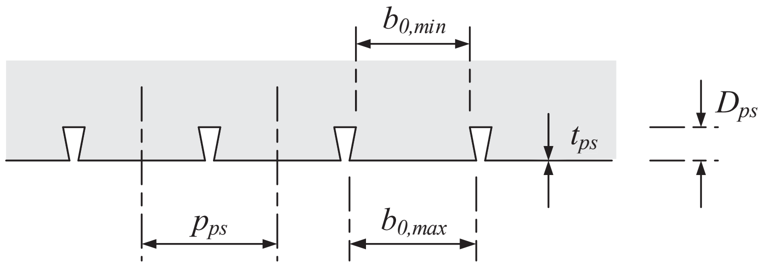


Figure AIII.3 Cross-section of profiled steel sheeting

Design strength: $f_{cd,cs} = f_{ck,cs}/\gamma_{c,cs} = 16.7$ [N/mm^2]
Dry density: $\rho_c = 2400$[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: $\gamma_s = 1.15$
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 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: $\gamma_{fp} = 1.00$

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: $\gamma_{cp} = 1.00$

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: $\gamma_b = 1.25$

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:

$$q_{k,2} = 5.00 \text{ [kN/m}^2\text{]}$$

[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): $\gamma_Q = 1.50$
Partial factor for variable actions (favourable): $\gamma_{Qi} = 0.00$

Design of Semi-rigid Composite Joints

[Verifications of joint classification]

Check initial rotational stiffness and moment resistance

Vertical distance between centre of longitudinal rebars and centre of contact partfor 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 partfor 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:

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

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

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

Effective length for row 1:

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

Effective length for row 2:

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

Equivalent stiffness coefficient of longitudinal rebars:

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

Length of composite beam in hogging moment region adjacent to joint:

$$l = 0.15L_b = 2250\ [\mathrm{mm}]$$

Number of headed studs distributed over length l:

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

Stiffness of one headed stud with19 [mm] diameter of shank:

$$k_{sc} = 100\ [\mathrm{kN/mm}]$$

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

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

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

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

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:

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

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

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

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

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

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

Cross-sectional area of bottom flange of steel beam:

$$A_{bf} = B_a t_f = 32.0 \text{ [cm}^2\text{]}$$

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

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

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

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

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):

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

Moment resistance:

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

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

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

Upper boundary of moment resistance for nominally pinned joint:

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

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

[Verifications of structural resistance in composite stage]

Check bolt group resistance

Correction factor for bolt shear resistance:

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

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:

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

α factor:

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

β factor:

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

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):

$$V_{Ed} = 383.4 \text{ [kN]}$$

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

<**In fin plate**>

k_1 factor for vertical bolt bearing resistance:

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

Correction factor for vertical bolt bearing resistance:

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

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:

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

Correction factor for horizontal bolt bearing resistance:

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

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:

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

Ultimate tensile strength of web of steel beam:

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

k_1 factor for vertical bolt bearing resistance:

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

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:

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

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:

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

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

Check fin plate resistance

Shear resistance for gross section:

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

Shear area for net section:

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

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:

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

Net area subjected to shear:

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

Block shear resistance:

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

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

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

Elastic moment resistance:

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

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

Fin plate type:

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

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:

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

Reduction factor for lateral-torsional buckling:

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

Lateral torsional buckling moment resistance:

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

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

Check resistance of web of steel beam

Shear area for gross section:

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

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:

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

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:

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

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

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

Design shear force on vertical line of bolts:

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

Plastic shear resistance on vertical line of bolts:

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

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

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

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

<u>*Check fillet weld of fin plate*</u>

Effective throat thickness of fillet weld:

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

Required minimum throat thickness of fillet weld:

$$a_{fp,req} = 0.6t_{fp} = 6.0\ [\text{mm}]\ ((\text{for S355 steel grade}))$$

$$\therefore\ a_{fp} > a_{fp,req}\ \text{OK}\ \left(\frac{a_{fp,req}}{a_{fp}} = 0.86\right)$$

[Verification of serviceability in composite stage]

<u>*Analysis of design moment*</u>

Design moment in composite stage can be calculated by the simplified analysis method described in **Appendix II**.

Maximum design distributed load:

$$w_{com,max} = B_b\,(g_{k,3}+q_{k,2}) = 24.0\ [\text{kN/m}]$$

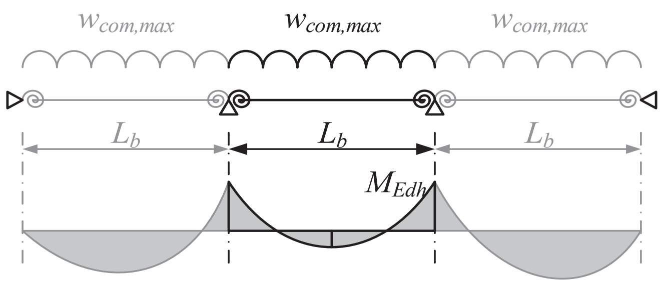


Figure AIII.4 Design load with corresponding moment (Load-case 2)

Design hogging moment (Load-case 2):

$$M_{Edh} = 401.4 \text{ [kNm]}$$

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:

$$\sigma_{sl,lim} = 320 \text{ [N/mm}^2\text{] (for } w_k = 0.3 \text{ [mm] and } p_{sl} = 100 \text{ [mm])}$$

$$\therefore\ z_{sl,eq\text{-}cc}\sum A_{sl,r}\sigma_{sl,lim} > M_{Edh} \quad \text{OK}$$

$$\left(\frac{M_{Edh}}{z_{sl,eq\text{-}cc}\sum A_{sl,r}\sigma_{sl,lim}} = 0.76\right)$$

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:

$$V_{Ed} = \frac{w_{con,max}L_b}{2} = 144.8 \text{ [kN]}$$

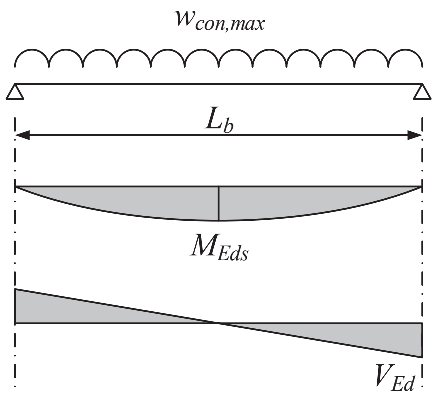


Figure AIII.5 Design load with corresponding moment and shear force

Check section classification

Classification of steel flange:

$$\frac{B_a - t_w - 2r}{2t_f} = 4.84 < 9\sqrt{\frac{235}{f_{ay}}} = 7.32 \rightarrow \text{ Class 1}$$

Classification of steel web:

$$\frac{D_a - 2t_f - 2r}{t_w} = 70.2 < 124\sqrt{\frac{235}{f_{ay}}} = 100.9 \rightarrow \text{ Class 3}$$

$\therefore$ Class 1 steel flange & Class 3 steel web $\rightarrow$ Effective Class 2 OK

Check shear resistance and moment resistance

Shear area:

$$A_V = \max\left\{A_a - 2B_a t_f + (t_w + 2r)\, t_f; 1.2D_a t_w\right\} = 72.1\ [\text{cm}^2]$$

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:

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

Minimum shear buckling coefficient:

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

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:

$$\chi_w = \frac{0.83}{\lambda_w} = 0.79\left(for\frac{0.83}{1.2} \leq \lambda_w\right)$$

Shear buckling resistance:

$$V_{b,a,Rd} = \min\left\{\frac{\chi_w f_{wy}\left(D_a - 2t_f - 2r\right)t_w}{\sqrt{3}\gamma_a}; \frac{1.2 f_{wy}\left(D_a - 2t_f - 2r\right)t_w}{\sqrt{3}\gamma_a}\right\}$$

$$= 969.3 \text{ [kN]} \qquad \left(\text{for } \frac{D_a - 2t_f}{t_w} > \frac{72}{1.2}\sqrt{\frac{235}{f_{ay}}}\right)$$

$$\therefore \min\left(V_{pl,a,Rd}; V_{b,a,Rd}\right) > V_{Ed} \text{ OK } \left(\frac{V_{Ed}}{\min\left(V_{pl,a,Rd}; V_{b,a,Rd}\right)} = 0.15\right)$$

Effective plastic section modulus:

$$W_{eff,pl,a} = \left\{\begin{array}{c} B_a t_f\left(D_a - t_f\right) + 0.4292r^2\left(D_a - 2t_f - 0.4467r\right) + t_w r\left(D_a - 2t_f - r\right) \\ +\frac{t_w}{2}\left(40t_w\sqrt{\frac{235}{f_{ay}}}\right)^2 + \frac{t_w}{2}\left(20t_w\sqrt{\frac{235}{f_{ay}}}\right)^2 \\ +t_w\left(20t_w\sqrt{\frac{235}{f_{ay}}}\right)\left(D_a - t_f - r - 40t_w\sqrt{\frac{235}{f_{ay}}} - t_f - r - 10t_w\sqrt{\frac{235}{f_{ay}}}\right) \end{array}\right\}$$

$$= 3219 \text{ [cm}^3\text{]}$$

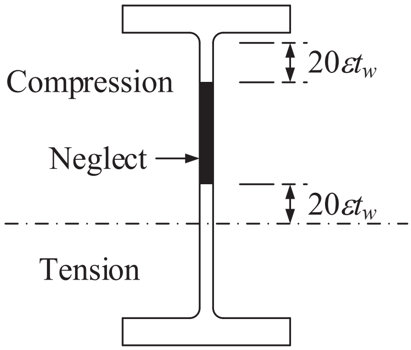


Figure AIII.6 Effective cross-section for effective Class 2

Plastic moment resistance:

$$M_{pl,a,Rd} = W_{eff,pl,a} f_{ayd} = 1142.7[\text{kNm}] \text{ (for effective Class 2 cross-section)}$$

$$\therefore\ M_{pl,a,Rd} > M_{Eds} \quad \text{OK} \left(\frac{M_{Eds}}{M_{pl,a,Rd}} = 0.48 \right)$$

[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):

$$M_{Eds} = 950.8 \text{ [kNm]}$$

Design hogging moment (Load-case 2):

$$M_{Edh} = 718.7 \text{ [kNm]}$$

Design shear force:

$$V_{Ed} = 383.4 \text{ [kN]}$$

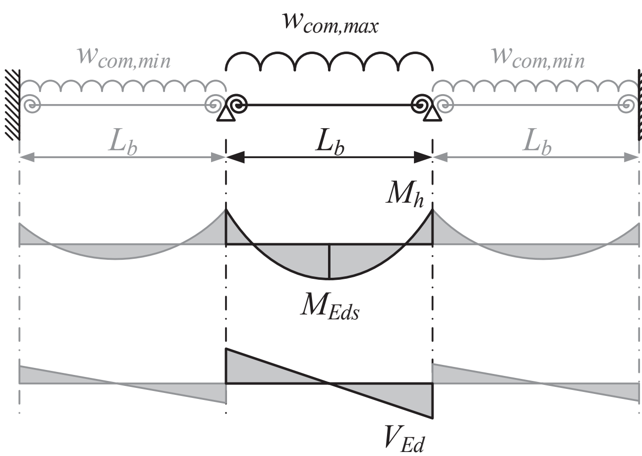


Figure AIII.7 Design load with corresponding moment and shear force (Load-case 1)

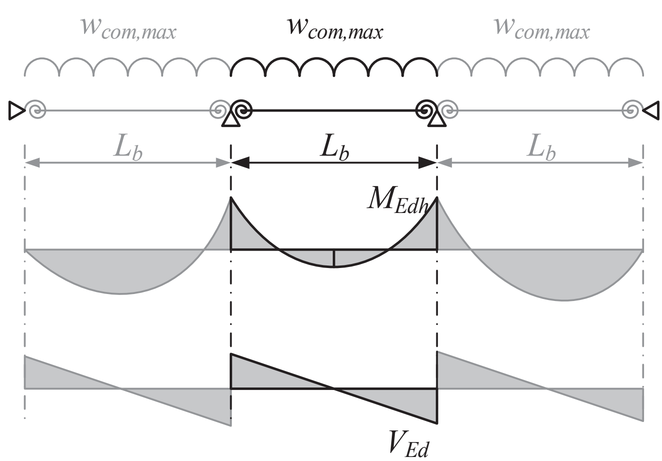


Figure AIII.8 Design load with corresponding moment and shear force (Load-case 2)

Check section classification

<**Load-case 1 for maximizing sagging moment**>

Classification of steel flange in sagging moment region:

$$\text{Bottom flange is in tension} \rightarrow \text{Class 1}$$

Effective width in sagging moment region:

$$b_{effs} = 3000.0 \text{ [mm]}$$

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:

$$\alpha = \frac{\left(\frac{D_a - 2t_f - 2r}{2}\right) - \left(\frac{R_{cs}}{2t_w f_{ayd}}\right)}{D_a - 2t_f - 2r} = -0.54$$

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:

$$\frac{(B_a - t_w - 2r)}{2t_f} = 4.84 < 9\sqrt{\frac{235}{f_{ay}}} = 7.32 \quad \rightarrow \text{ Class 1}$$

Effective width in hogging moment region:

$$b_{effh} = 1117.7 \text{ [mm]}$$

Cross-sectional area of longitudinal rebars within b_{effh}:

$$A_{sl} = \pi\left(\frac{\phi_{sl,1}}{2}\right)^2 \frac{b_{effh}}{p_{sl,1}} + \pi\left(\frac{\phi_{sl,2}}{2}\right)^2 \min\left(\frac{b_{effh}}{p_{sl,2}}; \frac{b_{sl,2}}{p_{sl,2}}\right) = 18.53 \text{ [cm}^2\text{]}$$

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:

$$\alpha = \frac{\left(\frac{D_a - 2t_f - 2r}{2}\right) + \left(\frac{R_{sl}}{2t_w f_{ayd}}\right)}{D_a - 2t_f - 2r} = 0.70$$

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:

$$\psi = 1 - \frac{D_a - 2t_f - 2r}{\frac{A_a D_a + 2A_{sl} z_{sl,eq\text{-}bf}}{2(A_a + A_{sl})} - t_f - r} = -0.69$$

Classification of steel web in hogging moment region:

$$\frac{D_a - 2t_f - 2r}{t_w} = 70.2 < \frac{42\sqrt{\frac{235}{f_{ay}}}}{0.67 + 0.33\psi} = 77.1 \quad \rightarrow \text{ Class 3}$$

$\therefore$ Class 1 steel flange & Class 3 steel web $\rightarrow$ Effective Class 2 OK

Second moment of area in hogging moment region:

$$I_h = I_{ay} + \frac{A_a A_{sl}\left\{D_a + 2\left(D_{ps} + h_{cs} - z_{tcs\text{-}sl,eq}\right)\right\}^2}{4\left(A_a + A_{sl}\right)} = 134362\ [\text{cm}^4]$$

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:

$$z_0 = \frac{\left(A_a + A_{sl}\right)\left(0.5D_a + D_{ps} + 0.5h_{cs}\right)}{A_a + A_{sl} + \left(\frac{h_{cs} b_{effh}}{n_0}\right)} = 212.2\ [\text{mm}]$$

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}:

$$A_{sl,req} = \rho_{sl,req}A_{cs} = 8.69\ [\text{cm}^2]$$

$$M_{el,Rdh} > M_{j,Rd} \rightarrow \text{ no need to check } A_{sl} > A_{sl,req}$$

Check minimum degree of shear connection

< **Load-case 1 for maximizing sagging moment**>

Distance between inflection points in sagging moment region:

$$L_{es} = L_b - 2x_0 = 12198\ [\text{mm}]$$

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}:

$$N_{hss} = \left[\frac{\left(\frac{L_{es}}{2}\right)}{p_{ps}}\right]n_{hss} = 62$$

Maximum reduction factor for shear resistance of a headed stud in sagging moment region:

$$k_{ts,max} = 0.60\left(\text{for } n_{hss} = 2, t_{ps} \leq 1, d_{hs} \leq 20, \text{ and sheeting with holes}\right)$$

Reduction factor for shear resistance of a headed stud in sagging moment region:

$$k_{ts} = \min\left\{\frac{0.7}{\sqrt{\min(n_{hss};2)}}\frac{b_{0,min}}{D_{ps}}\left(\frac{h_{hs}}{D_{ps}}-1\right);k_{ts,max}\right\} = 0.60$$

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:

$$R_a = A_a f_{ayd} = 4505.0\ [\text{kN}]$$

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:

$$\eta_{s,req} = \max\left\{1-\left(\frac{355}{f_{ayd}}\right)(0.75-0.03L_{es});0.4\right\} = 0.62\ (\text{for } L_{es} \le 25\ [\text{m}])$$

$$\therefore\ \eta_s > \eta_{s,req} \quad \text{OK}\ \left(\frac{\eta_{s,req}}{\eta_s} = 0.94\right)$$

< **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}:

$$N_{hsh} = \left[\frac{\left(\frac{L_{eh}}{2}\right) - h_{cj\text{-}fhs}}{p_{ps}}\right] n_{hsh} = 20$$

Maximum reduction factor for shear resistance of a headed stud in hogging moment region:

$$k_{th,max} = 0.60\ (\text{for } n_{hsh} = 2, t_{ps} \le 1, d_{hs} \le 20, \text{ and sheeting with holes})$$

Reduction factor for shear resistance of a headed stud in hogging moment region:

$$k_{th} = \min\left\{\frac{0.7}{\sqrt{\min\left(n_{hsh};2\right)}}\frac{b_{0,min}}{D_{ps}}\left(\frac{h_{hs}}{D_{ps}}-1\right);k_{th,max}\right\} = 0.60$$

Longitudinal shear force transfer within half of L_{eh}:

$$R_{qh} = N_{hsh}k_{th}P_{Rd} = 884.8\ [\mathrm{kN}]$$

Degree of shear connection in hogging moment region:

$$\eta_h = \frac{R_{qh}}{\min\left(R_a;R_{sl}\right)} = 1.10$$

Required minimum degree of shear connection in hogging moment region:

$$\eta_{h,req} = 1.00\ (\text{full shear connection})$$

$$\therefore\ \eta_h > \eta_{h,req} \quad \mathrm{OK}\ \left(\frac{\eta_{h,req}}{\eta_h} = 0.91\right)$$

<u>*Check shear resistance and moment resistance*</u>

Plastic shear resistance:

$$V_{pl,Rd} = 1478.7\ [\mathrm{kN}]$$

Shear buckling resistance:

$$V_{b,Rd} = 969.3\ [\mathrm{kN}]$$

$$\therefore\ \min\left(V_{pl,Rd};V_{b,Rd}\right) > V_{Ed} \quad \mathrm{OK}\ \left(\frac{V_{Ed}}{\min\left(V_{pl,Rd};V_{b,Rd}\right)} = 0.40\right)$$

< **Load-case 1 for maximizing sagging moment** >

Tension (Compression) resistance of overall web of steel beam:

$$R_w = R_a - 2B_a t_f f_{ayd} = 2233.0\ [\mathrm{kN}]$$

Tension (Compression) resistance of clear web of steel beam:

$$R_v = \left(D_a - 2t_f - 2r\right)t_w f_{ayd} = 2019.2\ [\mathrm{kN}]$$

Tension (Compression) resistance of effective clear web of steel beam:

$$R_{eff,v} = 40t_w^2 f_{ayd}\sqrt{\frac{235}{f_{ay}}} = 935.8 \text{ [kN]}$$

Location of plastic neutral axis for full shear connection:

$$R_w \leq R_{cs} < R_a \rightarrow PNA \text{ in steel flange}$$

Plastic sagging moment resistance with full shear connection:

$$M_{plf,Rds} = R_a \frac{D_a}{2} + R_{cs}\left(\frac{D_{cs} + D_{ps}}{2}\right) - \frac{(R_a - R_{cs})^2}{4B_a f_{ayd}} = 1999.3 \text{ [kNm]}$$

Location of plastic neutral axis for partial shear connection:

$$R_w \leq R_{qs} \rightarrow PNA \text{ in steel flange}$$

Plastic sagging moment resistance with partial shear connection:

$$M_{plp,Rds} = R_a \frac{D_a}{2} + R_{qs}\left(D_{cs} - \frac{R_{qs}}{R_{cs}}\frac{D_{cs} - D_{ps}}{2}\right) - \frac{(R_a - R_{qs})^2}{4B_a f_{ayd}} = 1888.7 \text{ [kNm]}$$

$$\therefore \min\left(M_{plf,Rds}; M_{plp,Rds}\right) > M_{Eds} \quad \text{OK}$$

$$\left(\frac{M_{Eds}}{min\left(M_{plf,Rds}; M_{plp,Rds}\right)} = 0.50\right)$$

< Load-case 2 for maximizing hogging moment >

Equivalent vertical distance between longitudinal rebars and top of flange of steel beam:

$$z_{sl,eq\text{-}tf} = D_{cs} - z_{tcs\text{-}sl,eq} = 109.3 \text{ [mm]}$$

Tension (Compression) resistance of effective steel beam:

$$R_{eff,a} = R_a - R_v + R_{eff,v} = 3421.6 \text{ [kN]}$$

Location of plastic neutral axis for with full shear connection:

$$R_{sl} < R_{eff,v} \rightarrow PNA \text{ in steel web}$$

Plastic hogging moment resistance with full shear connection:

$$M_{plf,Rdh} = \left\{ \begin{array}{c} W_{pl,a} f_{ayd} + R_{sl} \left(\dfrac{D_a}{2} + z_{sl,eq\text{-}tf} \right) \\ - \dfrac{R_{sl}^2 + (R_v + R_{sl}) \left(R_v + R_{sl} - 2R_{eff,v} \right)}{4 t_w f_{ayd}} \end{array} \right\} = 1274.6 \text{ [kNm]}$$

Tension (Compression) resistance of flange of steel beam:

$$R_f = B_a t_f f_{ayd} = 1136.0 \text{ [kN]}$$

Vertical distance between centres of top and bottom flange of steel beam:

$$z_{ctf\text{-}cbf} = D_a - t_f = 684.0 \text{ [mm]}$$

Plastic moment resistance after deducting shear area:

$$M_{pl,f,Rd} = R_f z_{ctf\text{-}cbf} + R_{sl} \left(\frac{D_a}{2} + z_{sl,eq\text{-}tf} \right) = 1147.0 \text{ [kNm]}$$

Reduced hogging moment resistance making allowance for presence of shear force:

$$M_{y,v,Rdh} = M_{plf,Rdh} = 1274.6 \text{ [kNm]} \left(\text{for } V_{Ed} \leq \frac{V_{pl,Rd}}{2} \right)$$

$$\therefore \ \min \left(M_{plf,Rdh}; M_{y,v,Rdh} \right) > M_{Edh} \quad \text{OK}$$

$$\left(\frac{M_{Edh}}{min \left(M_{plf,Rdh}; M_{y,v,Rdh} \right)} = 0.56 \right)$$

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:

$$e = \frac{(A_a + A_{sl})\, I_{ay}}{A_a z_{ccs\text{-}ca} A_{sl}} = 1459.1$$

k_c factor:

$$k_c = \frac{\frac{z_{ctf\text{-}cbf} I_h}{I_{ay}}}{\left(\frac{z^2_{ctf\text{-}cbf}}{4} + i^2_{ax}\right)/e + z_{ctf\text{-}cbf}} = 1.12$$

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:

$$L_{cr} = L_b = 15000 \text{ [mm]}$$

Cross-sectional area of transverse rebars per unit length:

$$A_{st} = \left\lfloor \frac{1000}{p_{st,1}} \right\rfloor \pi \left(\frac{\phi_{st,1}}{2}\right)^2 + \left\lfloor \frac{1000}{p_{st,2}} \right\rfloor \pi \left(\frac{\phi_{st,2}}{2}\right)^2 = 17.20 \text{ [cm}^2\text{/m]}$$

Area per unit length of concrete slab in compression:

$$A_{c,c} = \frac{A_c - 1000 h_{cs}}{\left(\frac{E_a}{E_{cm,cs}}\right)} = 66.8 \text{ [cm}^2\text{/m]}$$

Equivalent vertical distance between transverse rebars and concrete slab in compression:

$$z_{st,eq\text{-}ccs,c} = D_{cs} - z_{tcs\text{-}st,eq} - \frac{D_{ps}}{2} = 75.7 \text{ [mm]}$$

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:

$$z_{na\text{-}ccs,c} = z_{st,eq\text{-}ccs,c} - z_{st,eq\text{-}na} = 15.5 \text{ [mm]}$$

Second moment of area of cracked composite slab in direction transverse to steel beam:

$$I_{cs2} = A_{st} z_{st,eq\text{-}na}^2 + A_{c,c}\left(z_{na\text{-}ccs,c}^2 + \frac{D_{ps}^2}{12}\right) = 927.8\ [\text{cm}^4]$$

Cracked flexural stiffness per unit length of composite slab:

$$E_a I_{cs2} = 1948.4\ [\text{kNm}^2/\text{m}]$$

Flexural stiffness of cracked composite slab in direction transverse to steel beam:

$$k_1 = \frac{4E_a I_{cs2}}{B_b} = 2597.9\ [\text{kN/rad}]\ (\text{for continuous slab})$$

Flexural stiffness of web of steel beam:

$$k_2 = \frac{E_a t_w^3}{4\left(1-0.3^2\right) z_{ctf\text{-}cbf}} = 61.5\ [\text{kN/rad}]$$

Transverse (rotational) stiffness per unit length:

$$k_s = \frac{k_1 k_2}{k_1 + k_2} = 60.1\ [\text{kN/rad}]$$

Second moment of area of bottom flange of steel beam about minor axis (z-z axis):

$$I_{bfz} = \frac{t_f B_a^3}{12} = 1067\ [\text{cm}^4]$$

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 $M_{pl,Rkh}$:

$$R_{eff,v} \leq A_{sl} f_{sk} < R_{eff,a} \rightarrow PNA \text{ in steel flange}$$

Characteristic value of plastic hogging moment resistance:

$$M_{pl,Rkh} = R_{eff,a}\frac{D_a}{2} + A_{sl} f_{sk} z_{csl\text{-}tf} - \frac{\left(R_{eff,a} - A_{sl} f_{sk}\right)^2}{4B_a f_{ayd}} = 1276.8\ [\text{kNm}]$$

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:

$$\Phi_{LT} = 0.5 \left\{ 1 + \alpha_{LT} \left(\lambda_{LT} - 0.4 \right) + 0.75 \lambda_{LT}^2 \right\} = 0.76$$

Reduction factor for lateral-torsional buckling:

$$\chi_{LT} = \min \left(\frac{1}{\Phi_{LT} + \sqrt{\Phi_{LT}^2 - 0.75 \gamma_{LT}^2}} ; 1.0; \frac{1}{\gamma_{LT}^2} \right) = 0.79$$

Buckling moment resistance of laterally unrestrained composite beam:

$$M_{LT,Rd} = \chi_{LT} M_{plf,Rdh} = 1006.2 \text{ [kNm]}$$

Requirements to use calculation method in DD ENV 1994-1-1:

$$E_{cm,cs} I_{cs2} \geq 0.35 E_a t_w^2 \frac{B_b}{D_a} \quad \text{and} \quad \frac{p_{ps}}{B_a} \leq 0.4 f_{hsu} d_{hs}^2 \frac{1 - \chi_{LT} \lambda_{LT}^2}{k_s \chi_{LT} \lambda_{LT}^2} \rightarrow \text{ can be used}$$

$$\therefore \; M_{LT,Rd} > M_{Edh} \quad \text{OK} \left(\frac{M_{Edh}}{M_{LT,Rd}} = 0.71 \right)$$

Check longitudinal shear resistance

< **Load-case 1 for maximizing sagging moment** >

Effective width in hogging moment region:

$$b_{effh} = 1522.4 \text{ [mm]}$$

Cross-sectional area of longitudinal rebars within b_{effh}:

$$A_{sl} = \pi \left(\frac{\phi_{sl,1}}{2} \right)^2 \left\lfloor \frac{b_{effh}}{p_{sl,1}} \right\rfloor + \pi \left(\frac{\phi_{sl,2}}{2} \right)^2 \min \left(\left\lfloor \frac{b_{effh}}{p_{sl,2}} \right\rfloor ; \left\lfloor \frac{b_{sl,2}}{p_{sl,2}} \right\rfloor \right)$$

$$= 25.41 \text{ [cm}^2\text{]}$$

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:

$$\Delta N_L = \left\{ \begin{matrix} \min\left(R_a; R_{cs}; N_{hss} P_{Rd}\right) \\ + \min\left(\left\lfloor \frac{x_0 - h_{cj\text{-}fhs}}{p_{ps}} \right\rfloor n_{hsh} P_{Rd}; R_{sl}\right) \end{matrix} \right\} = 5240 \text{ [kN]}$$

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:

$$\theta_{min} = 38.6^\circ$$

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:

$$L_{es} = L_b - 2x_0 = 10608 \text{ [mm]}$$

Number of headed studs arranged within half of L_{es}:

$$N_{hss} = \left\lfloor \frac{\left(\frac{L_{es}}{2}\right)}{p_{ps}} \right\rfloor n_{hss} = 54$$

Change of longitudinal force in composite slab:

$$\Delta N_L = \left\{ \begin{array}{c} \min\left(R_a; R_{cs}; N_{hss}P_{Rd}\right) \\ +\min\left(\left\lfloor \frac{x_0 - h_{cj\text{-}fhs}}{p_{ps}} \right\rfloor n_{hsh}P_{Rd}; R_{sl}\right) \end{array} \right\} = 4665\ [\text{kN}]$$

Design longitudinal shear stress in composite slab:

$$v_{L,Ed} = \frac{\Delta N_L}{2h_{cs}\left(\frac{L_b}{2}\right)} = 3.14\ [\text{N/mm}^2]$$

Minimum angle to minimize cross-sectional area of transverse rebars:

$$\theta_{min} = 38.6^\circ$$

Required tension resistance of transverse reinforcement per unit length:

$$R_{tr,req} = \frac{1000h_{cs}v_{L,Ed}}{cot\theta_{min}} = 248.3\ [\text{kN/m}]$$

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:

$$R_{st} + R_{pse} = A_{st,1}f_{sd} + A_{pse}f_{psd} = 668.5\ [\text{kN/m}]$$

$$\therefore\ R_{st} + R_{pse} > \max\left(R_{tr,req}\right) \quad \text{OK} \left(\frac{\max\left(R_{tr,req}\right)}{R_{st} + R_{pse}} = 0.42\right)$$

Required minimum cross-sectional area of transverse rebars per unit length:

$$A_{st,req} = 1000h_{cs}\frac{0.08\sqrt{f_{ck,cs}}}{f_{sk}} = 0.79\ [\text{cm}^2/\text{m}]$$

$$\therefore\ A_{st,1} > A_{st,req} \quad \text{OK} \left(\frac{A_{st,req}}{A_{st,1}} = 0.20\right)$$

Crushing shear stress of concrete slab:

$$v_{Rd} = 0.6\left(1 - \frac{f_{ck,cs}}{250}\right) f_{cd,cs}sin\theta_{min}cos\theta_{min} = 4.39\ [\text{N/mm}^2]$$

$$\therefore\ v_{Rd} > \max\left(v_{L,Ed}\right) \quad \text{OK} \left(\frac{\max\left(v_{L,Ed}\right)}{v_{Rd}} = 0.80\right)$$

[Verifications of serviceability in construction stage]

Analysis of deflection

Design distributed load due to “dead loads”:

$$w_{con,P} = B_b g_{k,1} = 12.6 \text{ [kN/m]}$$

Design distributed load due to “live loads”:

$$w_{con,V} = B_b q_{k,1} = 1.5 \text{ [kN/m]}$$

Design distributed load due to “dead loads andlive loads”:

$$w_{con,P+V} = B_b \left(g_{k,1} + q_{k,1}\right) = 14.1 \text{ [kN/m]}$$

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 andlive loads”:

$$\delta_{P+V} = \frac{w_{con,P+V} L_b^4}{384 E_a I_{ay}} = 44.3 \text{ [mm]}$$

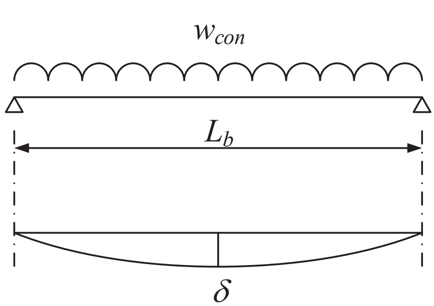


Figure AIII.9 Design load with corresponding deflection

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":

$$w_{com,P} = B_b g_{k,3} = 9.0 \text{ [kN/m]}$$

Maximum design distributed load due to "live loads":

$$w_{com,V,max} = B_b q_{k,2} = 15.0 \text{ [kN/m]}$$

Minimum design distributed load due to "live loads":

$$w_{com,V,min} = 0.0 \text{ [kN/m]}$$

Deflection due to "superimposed dead loads":

$$\delta_P = 4.0 \text{ [mm]}$$

Deflection due to "live loads" (Load-case 1):

$$\delta_V = 12.3 \text{ [mm]}$$

Deflection due to "dead loads and superimposed dead loads":

$$\delta_{tP} = 42.2 \text{ [mm]}$$

Deflection due to "dead loads, superimposed dead loads, and live loads":

$$\delta_{tP+V} = \delta_{tP} + \delta_V = 54.5 \text{ [mm]}$$

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":

$$\delta_{P+V,lim} = \frac{L_b}{250} = 60.0 \text{ [mm]}$$

$$\therefore\ \delta_V < \delta_{V,lim} \quad \text{OK} \left(\frac{\delta_V}{\delta_{V,lim}} = 0.29\right)$$

$$\therefore\ \delta_{tP+V} < \delta_{P+V,lim} \quad \text{OK} \left(\frac{\delta_{tP+V}}{\delta_{P+V,lim}} = 0.91\right)$$

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":

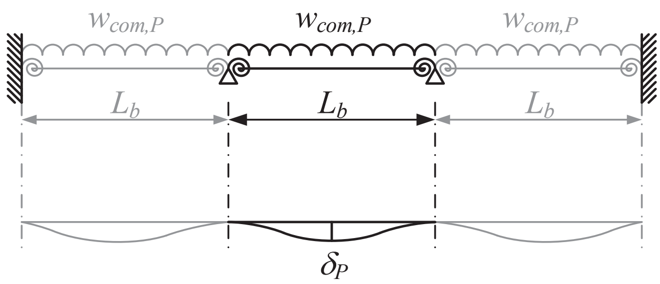
$$w_{com,P+0.1V} = B_b\left(g_{k,2} + g_{k,3} + 0.1q_{k,2}\right) = 22.7 \text{ [kN/m]}$$

Deflection due to "dead loads, superimposed dead loads, and 10% of live loads":

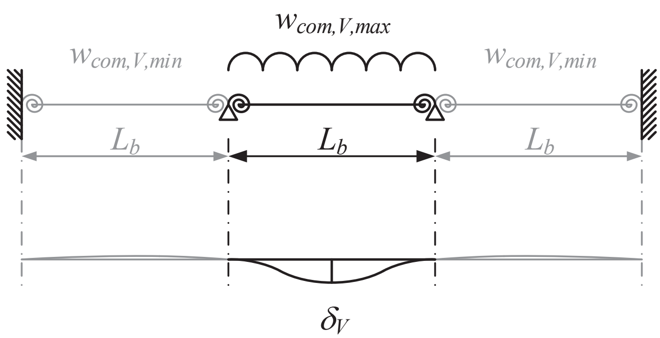
$$\delta_{P+0.1V} = 10.2 \text{ [mm]}$$

Natural frequency due to "dead loads, superimposed dead loads, and 10% of live loads":

$$f_{P+0.1V} = \frac{18}{\sqrt{\delta_{P+0.1V}}} = 5.6 \text{ [Hz]}$$



(a) *Superimposed dead loads*



(b) *Live loads (Load-case 1)*

Figure AIII.10 Design load with corresponding deflection

<u>*Check vibration*</u>

Required minimum natural frequency:

$$f_{req} = 4.0 \text{ [Hz]}$$

$$\therefore \; f_{P+0.1V} > f_{req} \quad \text{OK} \left(\frac{f_{req}}{f_{P+0.1V}} = 0.71 \right)$$

<u>*Control of crack width*</u>

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:

$$\sigma_{sl,lim} = 240 \text{ [N/mm}^2\text{] (for } w_k = 0.3 \text{ [mm] and 12 [mm]} < \phi^* \leq 16 \text{ [mm])}$$

< 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:

$$A_{cs} = b_{effh} h_{cs} = 1325.4 \text{ [cm}^2\text{]}$$

Required minimum cross-sectional area of longitudinal rebars within b_{effh}:

$$A_{sl,req} = \frac{0.72 k_c f_{ctm} A_{cs}}{\sigma_{sl,lim}} = 10.34 \text{ [cm}^2\text{]}$$

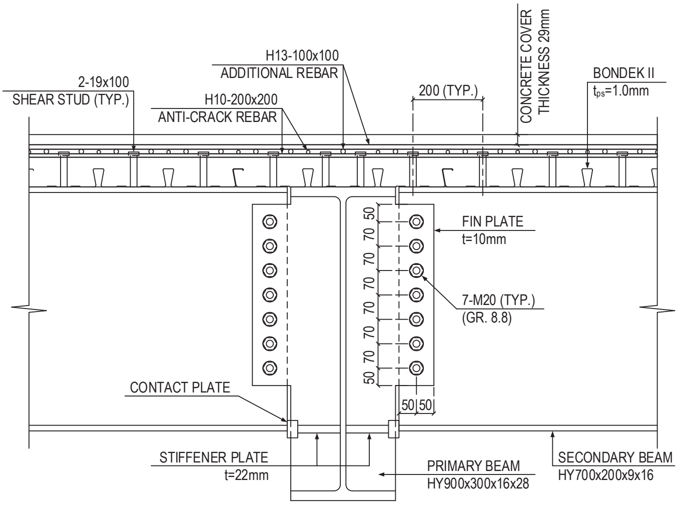
$$\therefore\ A_{sl} > A_{sl,req} \quad \text{OK} \left(\frac{A_{sl,req}}{A_{sl}} = 0.47\right)$$

Equivalent vertical distance between longitudinal rebars and neutral axis:

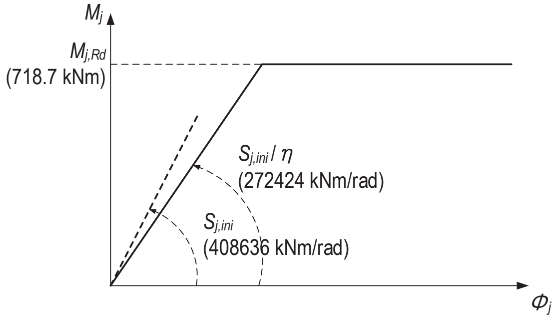
$$z_{sl,eq\text{-}na} = \frac{A_{sl} z_{tcs\text{-}sl,eq} + A_a\left(h_{cs} + D_{ps} + 0.5D_a\right)}{A_{sl} + A_a} - z_{tcs\text{-}sl,eq} = 391.5 \text{ [mm]}$$

Second moment of area in hogging moment region:

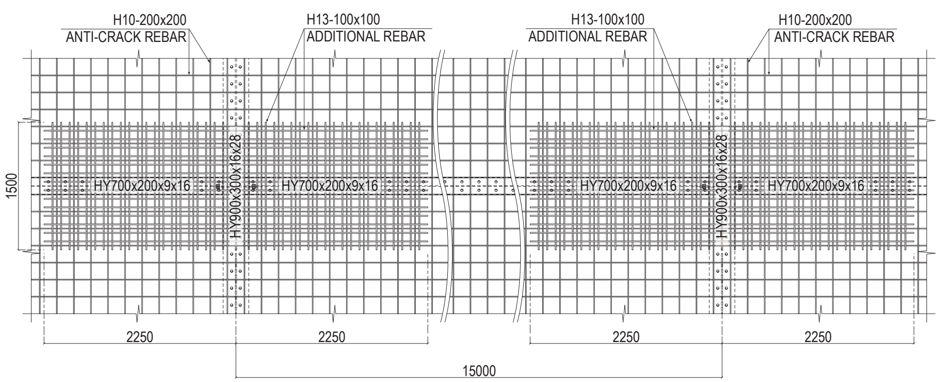
$$I_h = I_{ay} + \frac{A_a A_{sl}\left\{D_a + 2\left(D_{ps} + h_{cs} - z_{tcs\text{-}sl,eq}\right)\right\}^2}{4\left(A_a + A_{sl}\right)} = 139749 \text{ [cm}^4\text{]}$$



(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

Stress in longitudinal rebars caused by M_{Edh}:

$$\sigma_{sl,0} = \frac{M_{Edh}}{I_h} z_{sl,eq\text{-}na} = 112 \text{ [N/mm}^2\text{]}$$

Correction of stress in longitudinal rebars for tension stiffening:

$$\Delta\sigma_{sl} = \frac{0.4 f_{ctm}}{\frac{(A_a + A_{sl}) I_h}{A_a I_{ay}} \left(\frac{A_{sl}}{A_{cs}}\right)} = 38 \text{ [N/mm}^2\text{]}$$

Tensile stress in longitudinal rebars due to direct loading:

$$\sigma_{sl} = \sigma_{sl,0} + \Delta\sigma_{sl} = 151 \text{ [N/mm}^2\text{]}$$

$$\therefore \ \sigma_{sl} < \sigma_{sl,lim} \quad \text{OK} \left(\frac{\sigma_{sl}}{\sigma_{sl,lim}} = 0.63\right)$$

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

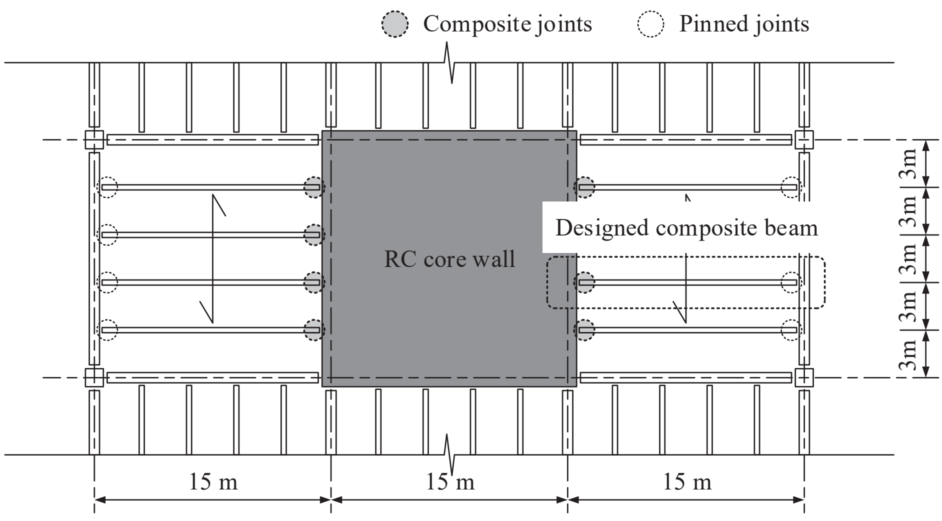


Figure AIV.1 Composite floor plan

Design Guide for Semi-rigid Composite Joints and Beams (2nd Edn.)
Authors: J Y Richard Liew, Yuichi Nishida & Masaki Arita
ISBN 978-981-18-7573-1 :: doi:10.3850/978-981-18-7573-1-AppIV

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**.

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: $\gamma_a = 1.00$

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]

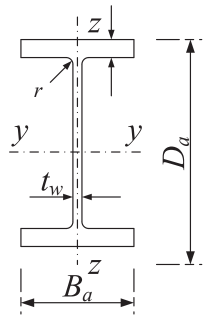


Figure AIV.2 Cross-section of steel beam

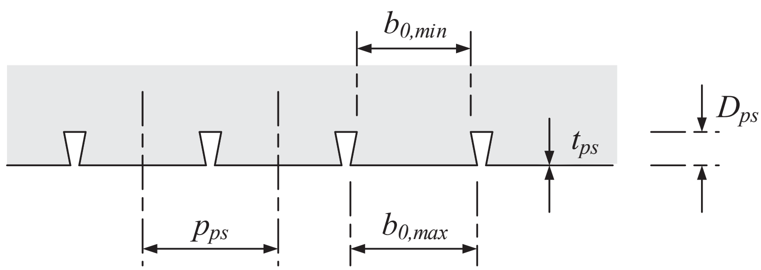


Figure AIV.3 Cross-section of profiled steel sheeting

[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: $\gamma_{ps} = 1.00$
Design yield strength: $f_{psd} = f_{psk}/\gamma_{ps} = 550$ [N/mm^2]

[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: $\rho_c = 2400$ [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: $h_{cw-fhs} = 50$ [mm]

Distance between centre of joint and first headed stud at semi-rigid end: $h_{cj-fhs} = 250$ [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: $\gamma_{fp} = 1.00$
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: $\gamma_{cp} = 1.00$
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: $\gamma_b = 1.25$
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:

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

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

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:

$$g_{k,3} = 2.00 \text{ [kN/m}^2\text{]}$$

[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:

$$q_{k,2} = 6.00 \text{ [kN/m}^2\text{]}$$

[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): $\gamma_Q = 1.50$
Partial factor for variable actions (favourable): $\gamma_{Qi} = 0.00$

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 partfor 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 partfor 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:

$$N = \left\lfloor\frac{\left(l - h_{cj\text{-}fhs}\right)}{p_{ps}}\right\rfloor n_{hsh} = 20$$

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:

$$z_{sl,eq\text{-}cc} = D_a + D_{cs} - z_{tcs\text{-}sl,eq} - \frac{t_f}{2} = 790 \text{ [mm]}$$

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

$$z_{sl,eq\text{-}ca} = \frac{D_a}{2} + D_{cs} - z_{tcs\text{-}sl,eq} = 448 \text{ [mm]}$$

Parameter related to deformation of headed studs:

$$\xi = \frac{E_a I_{ay}}{z_{sl,eq\text{-}ca}^2 E_s \left(A_{sl,1} + A_{sl,2}\right)} = 2.42$$

$$\nu = \sqrt{\frac{(1+\xi)\, N k_{sc} l z_{sl,eq\text{-}ca}^2}{E_a I_{ay}}} = 3.83$$

Stiffness related to headed studs:

$$K_{sc} = \frac{N k_{sc}}{\nu - \left(\frac{\nu - 1}{1+\xi}\right)\left(\frac{z_{sl,eq\text{-}cc}}{z_{sl,eq\text{-}ca}}\right)} = 843798 \text{ [N/mm]}$$

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:

$$\alpha = 2.52 \text{ (determined by convergence calculation)}$$

Design bearing strength of concrete for RC core wall:

$$f_{jd,cw} = \beta_j \alpha f_{cd,cw} = 56.0 \text{ [N/mm}^2\text{]}$$

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:

$$d_{eff} = t_f + c + \min\left(D_{ep} - D_a; c\right) = 79.0 \text{ [mm]}$$

Effective width of equivalent T-stub flange in compression:

$$b_{eff} = \min\left(B_a + 2c; B_{ep}\right) = 263.0 \text{ [mm]}$$

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:

$$t_{cw} \geq \max\left\{(\alpha - 1)\, d_{eff}; (\alpha - 1)\, b_{eff}\right\} \rightarrow \text{ can be calculated}$$

$$\therefore\ S_{j,ini} > \frac{0.5E_a I_b}{L_b} \text{ OK } \left(\frac{\left(\frac{0.5E_a I_b}{L_b}\right)}{S_{j,ini}} = 0.06\right)$$

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

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

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

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

Cross-sectional area of bottom flange of steel beam:

$$A_{bf} = B_a t_f = 32.0\ [\text{cm}^2]$$

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

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

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

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

Compression resistance of contact part:

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

Moment resistance:

$$M_{j,Rd} = z_{sl,eq\text{-}cc} \min\left(R_{sl,j}; R_{con}\right) = 709.3\ [\text{kNm}]$$

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

$$M_{pl,Rd} = 1263.7\ [\text{kNm}]$$

Upper boundary of moment resistance for nominally pinned joint:

$$0.25M_{pl,Rd} = 315.9 \text{ [kNm]}$$

$$\therefore\ M_{j,Rd} > 0.25M_{pl,Rd} \quad \text{OK} \left(\frac{0.25M_{pl,Rd}}{M_{j,Rd}} = 0.45\right)$$

Check compression resistance

Compression resistance of equivalent T-stub flange:

$$F_{C,Rd} = f_{jd,cw} d_{eff} b_{eff} = 1164.7 \text{ [kNm]}$$

$$\therefore\ F_{C,Rd} > R_{con} \quad \text{OK} \left(\frac{R_{con}}{F_{C,Rd}} = 0.91\right)$$

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

$$\therefore\ \text{Joint details} \rightarrow \text{Pre-qualified specification} \quad \text{OK}$$

[Verifications of structural resistance in composite stage]

Check bolt group resistance

Correction factor for bolt shear resistance:

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

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:

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

α factor:

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

β factor:

$$\beta = \frac{6z_{fs\text{-}b}}{n_{b,v}\left(n_{b,v}+1\right)p_{b,v}} = 0.12 \text{ (for } n_{b,h} = 1)$$

Bolt shear resistance:

$$V_{b,Rd} = \frac{n_{b,v} n_{b,h} F_{bV,Rd}}{\sqrt{(1 + \alpha n_{b,v} n_{b,h})^2 + (\beta n_{b,v} n_{b,h})^2}} = 454.9 \text{ [kN]}$$

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

$$V_{Ed} = 427.3 \text{ [kN]}$$

$$\therefore V_{b,Rd} > V_{Ed} \quad \text{OK} \left(\frac{V_{Ed}}{V_{b,Rd}} = 0.95 \right)$$

< **In fin plate** >

k_1 factor for vertical bolt bearing resistance:

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

Correction factor for vertical bolt bearing resistance:

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

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:

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

Correction factor for horizontal bolt bearing resistance:

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

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:

$$V_{bb,Rd} = \frac{n_{b,v} n_{b,h}}{\sqrt{\left(\frac{1+\alpha n_{b,v} n_{b,h}}{F_{vbb,Rd}}\right)^2 + \left(\frac{\beta n_{b,v} n_{b,h}}{F_{hbb,Rd}}\right)^2}} = 688.7\ [\mathrm{kN}]$$

< **In web of steel beam** >

Nominal value of yield strength of web of steel beam:

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

Ultimate tensile strength of web of steel beam:

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

k_1 factor for vertical bolt bearing resistance:

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

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:

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

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:

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

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:

$$A_{fpV,n} = t_{fp}\left(D_{fp} - n_{b,v} d_0\right) = 31.8\ [\text{cm}^2]$$

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:

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

Net area subjected to shear:

$$A_{fp,nV} = t_{fp}\left\{D_{fp} - e_{b\text{-}fp,v} - \left(n_{b,v} - 0.5\right) d_0\right\} = 27.9\ [\text{cm}^2]$$

Block shear resistance:

$$V_{fp,Rd,b} = \frac{0.5 f_{fpu} A_{fp,nt}}{\gamma_{fp,2}} + \frac{f_{fpy} A_{fp,nV}}{\sqrt{3}\gamma_{fp}} = 645.2 \text{ [kN]}$$

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

$$\left(\frac{V_{Ed}}{\min\left(V_{fp,Rd,g}; V_{fp,Rd,n}; V_{fp,Rd,b}\right)} = 0.67\right)$$

Elastic moment resistance:

$$M_{el,fp,Rd} = \frac{t_{fp} D_{fp}^2}{6} \frac{f_{fpy}}{\gamma_{fp}} = 119.8 \text{ [kNm]}$$

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

Fin plate type:

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

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:

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

Reduction factor for lateral-torsional buckling:

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

Lateral torsional buckling moment resistance:

$$M_{LT,fp,Rd} = M_{el,fp,Rd} = 119.8 \text{ [kNm] (for short fin plate)}$$

$$\therefore M_{LT,fp,Rd} > V_{Ed} z_{fs\text{-}b} \quad \text{OK} \quad \left(\frac{V_{Ed} z_{fs\text{-}b}}{M_{LT,fp,Rd}} = 0.22\right)$$

Check resistance of web of steel beam

Shear area for gross section:

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

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:

$$A_{bwV,n} = A_{bwV,g} - n_{b,v} d_0 t_w = 58.2 \text{ [cm}^2\text{]}$$

Shear resistance for net section:

$$V_{bw,Rd,n} = A_{bwV,n} \frac{f_{wu}}{\sqrt{3}\gamma_{a,2}} = 1263.9 \text{ [kN]}$$

$$\therefore \min\left(V_{bw,Rd,g}; V_{bw,Rd,n}\right) > V_{Ed} \quad \text{OK} \quad \left(\frac{V_{Ed}}{\min\left(V_{bw,Rd,g}; V_{bw,Rd,n}\right)} = 0.34\right)$$

Elastic moment resistance on vertical line of bolts:

$$M_{el,vbw,Rd} = \frac{t_w \left\{(n_{b,v} - 1)\, p_{b,v}\right\}^2}{6} \frac{f_{wy}}{\gamma_a} = 65.2 \text{ [kNm]}$$

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

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

Design shear force on vertical line of bolts:

$$V_{vbw,Ed} = V_{Ed} \frac{(n_{b,v} - 1)\, p_{b,v}}{D_a} = 217.0 \text{ [kN]}$$

Plastic shear resistance on vertical line of bolts:

$$V_{pl,vbw,Rd} = t_w \left(n_{b,v} - 1\right) p_{b,v} \frac{f_{wy}}{\sqrt{3}\gamma_a} = 645.6 \text{ [kN]}$$

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

$$M_{y,v,vbw,Rd} = M_{el,vbw,Rd} = 65.2 \text{ [kNm]} \left(\text{for } V_{vbw,Ed} \leq \frac{V_{pl,vbw,Rd}}{2} \right)$$

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

$$\left(\frac{V_{Ed} z_{fs\text{-}b}}{\min\left(M_{el,vbw,Rd}; M_{y,v,vbw,Rd}\right) + V_{pl,hbw,Rd}\left(n_{b,v} - 1\right) p_{b,v}} = 0.27 \right)$$

Check fillet weld of fin plate

Effective throat thickness of fillet weld:

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

Required minimum throat thickness of fillet weld:

$$a_{fp,req} = 0.6 t_{fp} = 6.0 \text{ [mm] (for S355 steel grade)}$$

$$\therefore a_{fp} > a_{fp,req} \quad \text{OK} \quad \left(\frac{a_{fp,req}}{a_{fp}} = 0.86 \right)$$

[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:

$$w_{com,max} = B_b\left(g_{k,3} + q_{k,2}\right) = 24.0 \text{ [kN/m]}$$

Design hogging moment:

$$M_{Edh} = 412.0 \text{ [kNm]}$$

Control of crack width

Maximum diameter of longitudinal rebars:

$$\phi_{sl}^{*} = \phi_{sl,max}\left(\frac{2.9}{f_{ctm}}\right) = 13.0 \text{ [mm]}$$

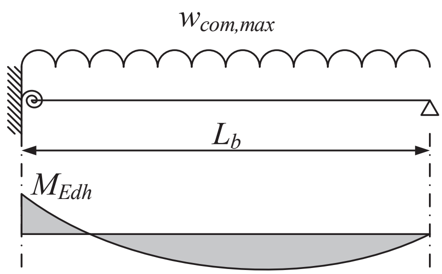


Figure AIV.4 Design load with corresponding moment

Limit of stress permitted in longitudinal rebars immediately after cracking:

$$\sigma_{sl,lim} = 320\ [\mathrm{N/mm^2}]\ (\text{ for } w_k = 0.3\ [\mathrm{mm}] \text{ and } p_{sl} = 100\ [\mathrm{mm}])$$

$$\therefore z_{sl,eq\text{-}cc} \sum A_{sl,r}\sigma_{sl,lim} > M_{Edh} \quad \text{OK}$$

$$\left(\frac{M_{Edh}}{z_{sl,eq\text{-}cc} \sum A_{sl,r}\sigma_{sl,lim}} = 0.79 \right)$$

Design of Composite Beam

[Verifications of structural resistance in construction stage]

Analysis of design moment and shear force

Maximum design distributed load:

$$w_{con,max} = B_b \left(g_{k,1}\gamma_{G,sup} + q_{k,1}\gamma_Q \right) = 19.3\ [\mathrm{kN/m}]$$

Design sagging moment:

$$M_{Eds} = \frac{w_{con,max} L_b^2}{8} = 543.0\ [\mathrm{kNm}]$$

Design shear force:

$$V_{Ed} = \frac{w_{con,max} L_b}{2} = 144.8\ [\mathrm{kN}]$$

Check section classification

Classification of steel flange:

$$\frac{B_a - t_w - 2r}{2t_f} = 4.84 < 9\sqrt{\frac{235}{f_{ay}}} = 7.32 \rightarrow \text{ Class 1}$$

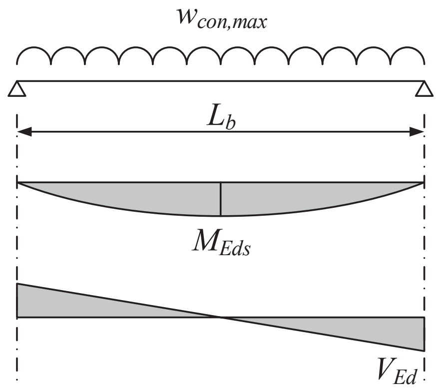


Figure AIV.5 Design load with corresponding moment and shear force

Classification of steel web:

$$\frac{D_a - 2t_f - 2r}{t_w} = 70.2 < 124\sqrt{\frac{235}{f_{ay}}} = 100.9 \rightarrow \text{ Class 3}$$

$\therefore$ Class 1 steel flange & Class 3 steel web $\rightarrow$ Effective Class 2 OK

Check shear resistance and moment resistance

Shear area:

$$A_V = \max\left\{A_a - 2B_a t_f + (t_w + 2r)\,t_f; 1.2D_a t_w\right\} = 72.1\ [\text{cm}^2]$$

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:

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

Minimum shear buckling coefficient:

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

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:

$$\chi_w = \frac{0.83}{\lambda_w} = 0.79 \left(\text{for } \frac{0.83}{1.2} \leq \lambda_w \right)$$

Shear buckling resistance:

$$V_{b,a,Rd} = \min \left\{ \frac{\chi_w f_{wy} \left(D_a - 2t_f - 2r \right) t_w}{\sqrt{3}\gamma_a} ; \frac{1.2 f_{wy} \left(D_a - 2t_f - 2r \right) t_w}{\sqrt{3}\gamma_a} \right\}$$

$$= 969.3 \text{ [kN]} \qquad \left(\text{for } \frac{D_a - 2t_f}{t_w} > \frac{72}{1.2} \sqrt{\frac{235}{f_{ay}}} \right)$$

$$\therefore \min \left(V_{pl,a,Rd} ; V_{b,a,Rd} \right) > V_{Ed} \quad \text{OK} \quad \left(\frac{V_{Ed}}{\min \left(V_{pl,a,Rd} ; V_{b,a,Rd} \right)} = 0.15 \right)$$

Effective plastic section modulus:

$$W_{eff,pl,a} = \left\{ \begin{array}{l} B_a t_f \left(D_a - t_f \right) + 0.4292 r^2 \left(D_a - 2t_f - 0.4467 r \right) + t_w r \left(D_a - 2t_f - r \right) \\ \qquad + \frac{t_w}{2} \left(40 t_w \sqrt{\frac{235}{f_{ay}}} \right)^2 + \frac{t_w}{2} \left(20 t_w \sqrt{\frac{235}{f_{ay}}} \right)^2 \\ + t_w \left(20 t_w \sqrt{\frac{235}{f_{ay}}} \right) \left(D_a - t_f - r - 40 t_w \sqrt{\frac{235}{f_{ay}}} - t_f - r - 10 t_w \sqrt{\frac{235}{f_{ay}}} \right) \end{array} \right\}$$

$$= 3219 \text{ [cm}^3\text{]}$$

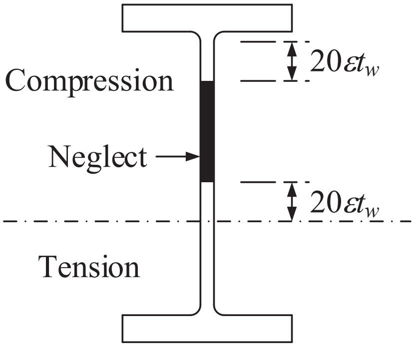


Figure AIV.6 Effective cross-section for effective Class 2

Plastic moment resistance:

$$M_{pl,a,Rd} = W_{eff,pl,a} f_{ayd} = 1142.7 \text{ [kNm] (for effective Class 2 cross-section)}$$

$$\therefore\ M_{pl,a,Rd} > M_{Eds} \quad \text{OK} \left(\frac{M_{Eds}}{M_{pl,a,Rd}} = 0.48 \right)$$

[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:

$$M_{Eds} = 1117.5 \text{ [kNm]}$$

Design hogging moment:

$$M_{Edh} = 709.3 \text{ [kNm]}$$

Design shear force:

$$V_{Ed} = 434.1 \text{ [kN]}$$

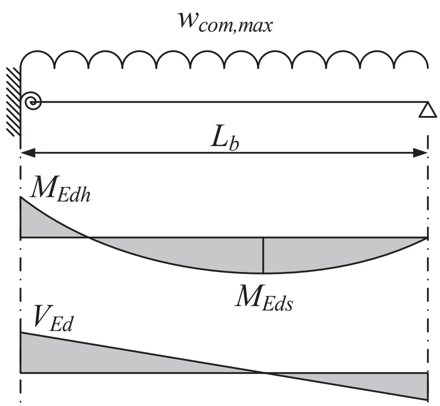


Figure AIV.7 Design load with corresponding moment and shear force

Check section classification

Classification of steel flange in sagging moment region:

$$\text{Bottom flange is in tension} \rightarrow \text{Class 1}$$

Effective width in sagging moment region:

$$b_{effs} = b_{0s} + \min\left(\frac{x_0}{4}; B_b - b_{0s}\right) = 3000.0 \text{ [mm]}$$

Compression resistance of composite slab within b_{effs}:

$$R_{cs} = b_{effs} h_{cs} \left(0.85 f_{cd,cs}\right) = 5049.0 \text{ [kN]}$$

Portion of part of cross-section in compression:

$$\alpha = \frac{\left(\frac{D_a - 2t_f - 2r}{2}\right) - \left(\frac{R_{cs}}{2t_w f_{ayd}}\right)}{D_a - 2t_f - 2r} = -0.75$$

Classification of steel web in sagging moment region:

$$\text{Full web is in tension} \rightarrow \text{Class 1}$$

$\therefore$ Class 1 steel flange & Class 1 steel web $\rightarrow$ Class 1 OK

Classification of steel flange in hogging moment region:

$$\frac{(B_a - t_w - 2r)}{2t_f} = 4.84 < 9\sqrt{\frac{235}{f_{ay}}} = 7.32 \rightarrow \text{ Class 1}$$

Effective width in hogging moment region:

$$b_{effh} = b_{0h} + \min\left\{\frac{2\left(L_b - x_0\right)}{4}; B_b - b_{0h}\right\} = 1016.9 \text{ [mm]}$$

Cross-sectional area of longitudinal rebars within b_{effh}:

$$A_{sl} = \pi\left(\frac{\phi_{sl,1}}{2}\right)^2 \left\lfloor\frac{b_{effh}}{p_{sl,1}}\right\rfloor + \pi\left(\frac{\phi_{sl,2}}{2}\right)^2 \min\left(\left\lfloor\frac{b_{effh}}{p_{sl,2}}\right\rfloor; \left\lfloor\frac{b_{sl,2}}{p_{sl,2}}\right\rfloor\right)$$

$$= 17.20 \text{ [cm}^2\text{]}$$

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:

$$\alpha = \frac{\left(\frac{D_a - 2t_f - 2r}{2}\right) + \left(\frac{R_{sl}}{2t_w f_{ayd}}\right)}{D_a - 2t_f - 2r} = 0.69$$

Equivalent vertical distance between longitudinal rebars and bottom of flange of steel beam:

$$z_{sl,eq\text{-}bf} = D_a + D_{cs} - z_{tcs\text{-}sl,eq} = 798.4 \text{ [mm]}$$

Stress or strain ratio:

$$\psi = 1 - \frac{D_a - 2t_f - 2r}{\frac{A_a D_a + 2A_{sl} z_{sl,eq\text{-}bf}}{2(A_a + A_{sl})} - t_f - r} = -0.71$$

Classification of steel web in hogging moment region:

$$\frac{D_a - 2t_f - 2r}{t_w} = 70.2 < \frac{42\sqrt{\frac{235}{f_{ay}}}}{0.67 + 0.33\psi} = 78.4 \rightarrow \text{ Class 3}$$

$\therefore$ Class 1 steel flange & Class 3 steel web $\rightarrow$ Effective Class 2 OK

Second moment of area in hogging moment region:

$$I_h = I_{ay} + \frac{A_a A_{sl} \left\{D_a + 2\left(D_{ps} + h_{cs} - z_{tcs\text{-}sl,eq}\right)\right\}^2}{4\left(A_a + A_{sl}\right)} = 130712 \text{ [cm}^4\text{]}$$

Elastic hogging moment resistance:

$$M_{el,Rdh} = \min\left\{\frac{I_h f_{sd}}{z_{sl,eq\text{-}bf} - \frac{A_a D_a + 2A_{sl} z_{sl,eq\text{-}bf}}{2(A_a + A_{sl})}}; \frac{I_h f_{ayd}}{\frac{A_a D_a + 2A_{sl} z_{sl,eq\text{-}bf}}{2(A_a + A_{sl})}}\right\}$$

$$= 1149.9 \text{ [kNm]}$$

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:

$$z_0 = \frac{(A_a + A_{sl})\left(0.5D_a + D_{ps} + 0.5h_{cs}\right)}{A_a + A_{sl} + \left(\frac{h_{cs}b_{effh}}{n_0}\right)} = 214.7 \text{ [mm]}$$

Coefficient taking into account of stress distribution within section immediately prior to cracking:

$$k_c = \min\left\{\frac{1}{1 + \left(\frac{h_{cs}}{2z_0}\right)} + 0.3; 1.0\right\} = 1.00$$

Required minimum reinforcement ratio:

$$\rho_{sl,req} = \frac{f_{ay}}{235}\frac{f_{ctm}}{f_{sk}}\sqrt{k_c} = 0.88\% \text{ (for Class 2 cross-section)}$$

Cross-sectional area of composite slab within b_{effh} above profiled steel sheeting:

$$A_{cs} = b_{effh}h_{cs} = 1006.8 \text{ [cm}^2\text{]}$$

Required minimum cross-sectional area of longitudinal rebars within b_{effh}:

$$A_{sl,req} = \rho_{sl,req}A_{cs} = 8.82 \text{ [cm}^2\text{]}$$

$M_{el,Rdh} > M_{j,Rd} \rightarrow$ 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:

$$P_{Rd} = \min\left(\frac{0.8f_{hsu}\pi d_{hs}^2}{4\gamma_V}; \frac{0.29\alpha d_{hs}^2\sqrt{f_{ck,cs}E_{cm,cs}}}{\gamma_V}\right) = 81.7 \text{ [kN]}$$

Number of headed studs arranged within half of L_{es}:

$$N_{hss} = \frac{\left(\frac{L_{es}}{2}\right) - h_{cj\text{-}fhs}}{p_{ps}} n_{hss} = 64$$

Maximum reduction factor for shear resistance of a headed stud in sagging moment region:

$$k_{ts,max} = 0.60 \; (\text{for } n_{hss} = 2, t_{ps} \leq 1, d_{hs} \leq 20, \text{ and sheeting with holes})$$

Reduction factor for shear resistance of a headed stud in sagging moment region:

$$k_{ts} = \min \left\{ \frac{0.7}{\sqrt{\min(n_{hss}; 2)}} \frac{b_{0,min}}{D_{ps}} \left(\frac{h_{hs}}{D_{ps}} - 1 \right); k_{ts,max} \right\} = 0.60$$

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:

$$\eta_{s,req} = \max \left\{ 1 - \left(\frac{355}{f_{ayd}} \right) (0.75 - 0.03 L_{es}); 0.4 \right\} = 0.64 \; (\text{for } L_{es} \leq 25 \, [m])$$

$$\therefore \; \eta_s > \eta_{s,req} \quad \text{OK} \left(\frac{\eta_{s,req}}{\eta_s} = 0.93 \right)$$

Half of distance between inflection points in hogging moment region:

$$\frac{L_{eh}}{2} = L_b - x_0 = 1834 \text{ [mm]}$$

Number of headed studs arranged within half of L_{eh}:

$$N_{hsh} = \left\lfloor \frac{\left(\frac{L_{eh}}{2}\right) - h_{cj\text{-}fhs}}{p_{ps}} \right\rfloor n_{hsh} = 16$$

Maximum reduction factor for shear resistance of a headed stud in hogging moment region:

$$k_{th,max} = 0.60 \left(\text{for } n_{hsh} = 2, t_{ps} \leq 1, d_{hs} \leq 20, \text{ and sheeting with holes}\right)$$

Reduction factor for shear resistance of a headed stud in hogging moment region:

$$k_{th} = \min\left\{\frac{0.7}{\sqrt{\min\left(n_{hsh}; 2\right)}} \frac{b_{0,min}}{D_{ps}} \left(\frac{h_{hs}}{D_{ps}} - 1\right); k_{th,max}\right\} = 0.60$$

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:

$$\eta_h = \frac{R_{qh}}{\min\left(R_a; R_{sl}\right)} = 1.05$$

Required minimum degree of shear connection in hogging moment region:

$$\eta_{h,req} = 1.00 \text{ (full shear connection)}$$

$$\therefore\ \eta_h > \eta_{h,req} \quad \text{OK} \left(\frac{\eta_{h,req}}{\eta_h} = 0.95\right)$$

<u>*Check shear resistance and moment resistance*</u>

Plastic shear resistance:

$$V_{pl,Rd} = 1478.7 \text{ [kN]}$$

Shear buckling resistance:

$$V_{b,Rd} = 969.3 \text{ [kN]}$$

$$\therefore\ \min\left(V_{pl,Rd}; V_{b,Rd}\right) > V_{Ed} \quad \text{OK} \left(\frac{V_{Ed}}{\min\left(V_{pl,Rd}; V_{b,Rd}\right)} = 0.45\right)$$

Tension (Compression) resistance of overall web of steel beam:

$$R_w = R_a - 2B_a t_f f_{ayd} = 2233.0 \text{ [kN]}$$

Tension (Compression) resistance of clear web of steel beam:

$$R_v = \left(D_a - 2t_f - 2r\right) t_w f_{ayd} = 2019.2 \text{ [kN]}$$

Tension (Compression) resistance of effective clear web of steel beam:

$$R_{eff,v} = 40t_w^2 f_{ayd} \sqrt{\frac{235}{f_{ay}}} = 935.8 \text{ [kN]}$$

Location of plastic neutral axis for full shear connection:

$$Ra \leq Rcs \rightarrow PNA \text{ in concrete flange}$$

Plastic sagging moment resistance with full shear connection:

$$M_{plf,Rds} = R_a \left\{ \frac{D_a}{2} + D_{cs} - \frac{R_a}{R_{cs}} \frac{\left(D_{cs} - D_{ps}\right)}{2} \right\} = 2053.5 \text{ [kNm]}$$

Location of plastic neutral axis for partial shear connection:

$$R_w \leq R_{qs} \rightarrow PNA \text{ in steel flange}$$

Plastic sagging moment resistance with partial shear connection:

$$M_{plp,Rds} = R_a \frac{D_a}{2} + R_{qs} \left(D_{cs} - \frac{R_{qs}}{R_{cs}} \frac{D_{cs} - D_{ps}}{2} \right) - \frac{\left(R_a - R_{qs}\right)^2}{4B_a f_{ayd}} = 1944.1 \text{ [kNm]}$$

$$\therefore \min\left(M_{plf,Rds}; M_{plp,Rds}\right) > M_{Eds} \quad \text{OK}$$

$$\left(\frac{M_{Eds}}{\min\left(M_{plf,Rds}; M_{plp,Rds}\right)} = 0.57 \right)$$

Equivalent vertical distance between longitudinal rebars and top of flange of steel beam:

$$z_{sl,eq\text{-}tf} = D_{cs} - z_{tcs\text{-}sl,eq} = 98.4 \text{ [mm]}$$

Tension (Compression) resistance of effective steel beam:

$$R_{eff,a} = R_a - R_v + R_{eff,v} = 3421.6 \text{ [kN]}$$

Location of plastic neutral axis for with full shear connection:

$$R_{sl} < R_{eff,v} \rightarrow PNA \text{ in steel web}$$

Plastic hogging moment resistance with full shear connection:

$$M_{plf,Rdh} = \left\{ \begin{array}{c} W_{pl,a}f_{ayd} + R_{sl}\left(\dfrac{D_a}{2} + z_{sl,eq\text{-}tf}\right) \\ -\dfrac{R_{sl}^2 + (R_v + R_{sl})\left(R_v + R_{sl} - 2R_{eff,v}\right)}{4t_w f_{ayd}} \end{array} \right\} = 1263.7 \text{ [kNm]}$$

Tension (Compression) resistance of flange of steel beam:

$$R_f = B_a t_f f_{ayd} = 1136.0 \text{ [kN]}$$

Vertical distance between centres of top and bottom flange of steel beam:

$$z_{ctf\text{-}cbf} = D_a - t_f = 684.0 \text{ [mm]}$$

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:

$$M_{y,v,Rdh} = M_{plf,Rdh} = 1263.7 \text{ [kNm]} \left(\text{for } V_{Ed} \le \frac{V_{pl,Rd}}{2}\right)$$

$$\therefore \ \min\left(M_{plf,Rdh}; M_{y,v,Rdh}\right) > M_{Edh} \quad \text{OK}$$

$$\left(\frac{M_{Edh}}{\min\left(M_{plf,Rdh}; M_{y,v,Rdh}\right)} = 0.56\right)$$

Check lateral-torsional buckling

Polar radius of gyration of area of steel beam:

$$i_{ax} = \sqrt{\frac{(I_{ay} + I_{az})}{A_a}} = 284.1\ [\text{mm}]$$

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:

$$e = \frac{(A_a + A_{sl})\, I_{ay}}{A_a z_{ccs\text{-}ca} A_{sl}} = 1557.3$$

k_c factor:

$$k_c = \frac{\frac{z_{ctf\text{-}cbf} I_h}{I_{ay}}}{\left(\frac{z_{ctf\text{-}cbf}^2}{4} + i_{ax}^2\right) / e + z_{ctf\text{-}cbf}} = 1.10$$

Property of distribution of moment (conservative value):

$$C_4 = 33.9 \left(\text{for } \psi = \frac{M_{Edh}}{\left(\frac{w_{com,max} L_b^2}{8}\right)} < 0.50 \right)$$

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:

$$A_{c,c} = \frac{A_c - 1000 h_{cs}}{\left(\frac{E_a}{E_{cm,cs}}\right)} = 71.1\ [\text{cm}^2/\text{m}]$$

Equivalent vertical distance between transverse rebars and concrete slab in compression:

$$z_{st,eq\text{-}ccs,c} = D_{cs} - z_{tcs\text{-}st,eq} - \frac{D_{ps}}{2} = 79.5 \text{ [mm]}$$

Equivalent Vertical distance between transverse rebars and neutral axis of composite slab:

$$z_{st,eq\text{-}na} = \frac{z_{st,eq\text{-}ccs,c}}{\left(1 + \frac{A_{st}}{A_{c,c}}\right)} = 75.3 \text{ [mm]}$$

Vertical distance between neutral axis of composite slab and centre of concrete slab in compression:

$$z_{na\text{-}ccs,c} = z_{st,eq\text{-}ccs,c} - z_{st,eq\text{-}na} = 4.2 \text{ [mm]}$$

Second moment of area of cracked composite slab in direction transverse to steel beam:

$$I_{cs2} = A_{st} z^2_{st,eq\text{-}na} + A_{c,c}\left(z^2_{na\text{-}ccs,c} + \frac{D^2_{ps}}{12}\right) = 389.4 \text{ [cm}^4\text{]}$$

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:

$$k_2 = \frac{E_a t^3_w}{4\left(1 - 0.3^2\right) z_{ctf\text{-}cbf}} = 61.5 \text{ [kN/rad]}$$

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):

$$I_{bfz} = \frac{t_f B^3_a}{12} = 1067 \text{ [cm}^4\text{]}$$

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 $M_{pl,Rkh}$:

$$A_{sl} f_{sk} < R_{eff,v} \rightarrow PNA \text{ in steel web}$$

Characteristic value of plastic hogging moment resistance:

$$M_{pl,Rkh} = \left\{ \begin{array}{c} W_{pl,a} f_{ayd} + A_{sl} f_{sk} \left(\dfrac{D_a}{2} + z_{sl,eq\text{-}tf} \right) \\ - \dfrac{(A_{sl} f_{sk})^2 + (R_v + A_{sl} f_{sk}) \left(R_v + A_{sl} f_{sk} - 2R_{eff,v} \right)}{4 t_w f_{ayd}} \end{array} \right\}$$

$$= 1266.8 \text{ [kNm]}$$

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 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:

$$\Phi_{LT} = 0.5 \left\{ 1 + \alpha_{LT} \left(\lambda_{LT} - 0.4 \right) + 0.75 \lambda_{LT}^2 \right\} = 0.66$$

Reduction factor for lateral-torsional buckling:

$$\chi_{LT} = \min \left(\frac{1}{\Phi_{LT} + \sqrt{\Phi_{LT}^2 - 0.75 \lambda_{LT}^2}}; 1.0; \frac{1}{\lambda_{LT}^2} \right) = 0.89$$

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:

$$E_{cm,cs} I_{cs2} \geq 0.35 E_a t_w^2 \frac{B_b}{D_a} \text{ and } \frac{p_{ps}}{B_a} \leq 0.4 f_{hsu} d_{hs}^2 \frac{1 - \chi_{LT} \lambda_{LT}^2}{k_s \chi_{LT} \lambda_{LT}^2} \rightarrow \text{ can be used}$$

$$\therefore\ M_{LT,Rd} > M_{Edh} \quad \text{OK} \left(\frac{M_{Edh}}{M_{LT,Rd}} = 0.63 \right)$$

Check longitudinal shear resistance

<**Beam-to-wall composite joint side**>

Change of longitudinal force in composite slab:

$$\Lambda N_L = \left\{ \begin{array}{c} \min \left(R_a ; R_{cs} ; N_{hss} P_{Rd} \right) \\ + \min \left(\left\lfloor \dfrac{L_b - x_0 - h_{cj\text{-}fhs}}{p_{ps}} \right\rfloor n_{hsh} P_{Rd} ; R_{sl} \right) \end{array} \right\} = 5253 \text{ [kN]}$$

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:

$$v_{L,Ed} = \frac{\Delta N_L}{2 h_{cs} \left(L_b - x_s \right)} = 3.15 \text{ [N/mm}^2\text{]}$$

Minimum angle to minimize cross-sectional area of transverse rebars:

$$\theta_{min} = 38.6^\circ$$

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:

$$\Delta N_L = \min \left(R_a ; R_{cs} ; N_{hss} P_{Rd} \right) = 4505 \text{ [kN]}$$

Design longitudinal shear stress in composite slab:

$$v_{L,Ed} = \frac{\Delta N_L}{2 h_{cs} x_s} = 3.46 \text{ [N/mm}^2\text{]}$$

Minimum angle to minimize cross-sectional area of transverse rebars:

$$\theta_{min} = 38.6^\circ$$

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:

$$R_{st} + R_{pse} = A_{st,1} f_{sd} + A_{pse} f_{psd} = 668.5 \text{ [kN/m]}$$

$$\therefore\ R_{st} + R_{pse} > \max\left(R_{tr,req}\right) \quad \text{OK} \left(\frac{\max\left(R_{tr,req}\right)}{R_{st} + R_{pse}} = 0.41\right)$$

Required minimum cross-sectional area of transverse rebars per unit length:

$$A_{st,req} = 1000 h_{cs} \frac{0.08\sqrt{f_{ck,cs}}}{f_{sk}} = 0.87 \text{ [cm}^2\text{/m]}$$

$$\therefore\ A_{st,1} > A_{st,req} \quad \text{OK} \left(\frac{A_{st,req}}{A_{st,1}} = 0.22\right)$$

Crushing shear stress of concrete slab:

$$v_{Rd} = 0.6\left(1 - \frac{f_{ck,cs}}{250}\right) f_{cd,cs} \sin\theta_{min} \cos\theta_{min} = 5.15 \text{ [N/mm}^2\text{]}$$

$$\therefore\ v_{Rd} > \max\left(v_{L,Ed}\right) \quad \text{OK} \left(\frac{max\left(v_{L,Ed}\right)}{v_{Rd}} = 0.67\right)$$

[Verifications of serviceability in construction stage]

Analysis of deflection

Design distributed load due to "dead loads":

$$w_{con,P} = B_b g_{k,1} = 12.6 \text{ [kN/m]}$$

Design distributed load due to "live loads":

$$w_{con,V} = B_b q_{k,1} = 1.5 \text{ [kN/m]}$$

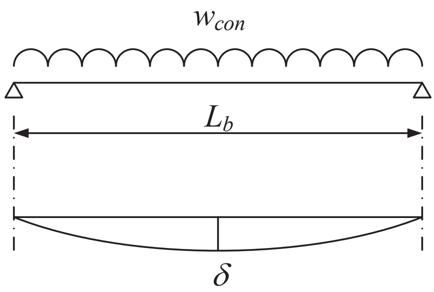


Figure AIV.8 Design load with corresponding deflection

Design distributed load due to "dead loads andlive loads":

$$w_{con,P+V} = B_b \left(g_{k,1} + q_{k,1}\right) = 14.1\ [\text{kN/m}]$$

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 andlive loads":

$$\delta_{P+V} = \frac{w_{con,P+V} L_b^4}{384 E_a I_{ay}} = 44.3\ [\text{mm}]$$

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":

$$w_{com,P} = B_b g_{k,3} = 6.0 \text{ [kN/m]}$$

Maximum design distributed load due to "live loads":

$$w_{com,V,max} = B_b q_{k,2} = 18.0 \text{ [kN/m]}$$

Deflection due to "superimposed dead loads":

$$\delta_P = 4.4 \text{ [mm]}$$

Deflection due to "live loads" (Load-case 1):

$$\delta_V = 13.2 \text{ [mm]}$$

Deflection due to "dead loads and superimposed dead loads":

$$\delta_{tP} = 42.6 \text{ [mm]}$$

Deflection due to "dead loads, superimposed dead loads, and live loads":

$$\delta_{tP+V} = \delta_{tP} + \delta_V = 55.8 \text{ [mm]}$$

Check deflection

Limit of deflection due to "live loads":

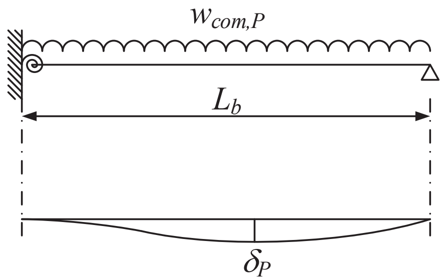
$$\delta_{V,lim} = \frac{L_b}{360} = 41.7 \text{ [mm]}$$

Limit of deflection due to "dead loads, superimposed dead loads, and live loads":

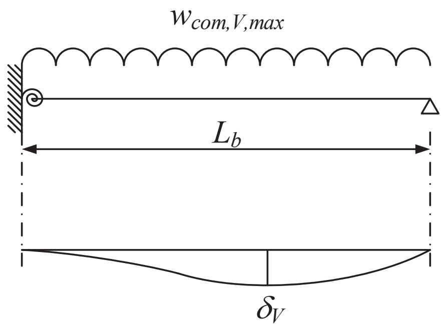
$$\delta_{P+V,lim} = \frac{L_b}{250} = 60.0 \text{ [mm]}$$

$$\therefore\ \delta_V < \delta_{V,lim} \quad \text{OK} \left(\frac{\delta_V}{\delta_{V,lim}} = 0.32 \right)$$

$$\therefore\ \delta_{tP+V} < \delta_{P+V,lim} \quad \text{OK} \left(\frac{\delta_{tP+V}}{\delta_{P+V,lim}} = 0.93 \right)$$



(a) *Superimposed dead loads*



(b) *Live loads*

Figure AIV.9 Design load with corresponding deflection

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":

$$w_{com,P+0.1V} = B_b\left(g_{k,2} + g_{k,3} + 0.1q_{k,2}\right) = 20.0\ [\text{kN/m}]$$

Deflection due to "dead loads, superimposed dead loads, and 10% of live loads":

$$\delta_{P+0.1V} = 14.7\ [\text{mm}]$$

Natural frequency due to "dead loads, superimposed dead loads, and 10% of live loads":

$$f_{P+0.1V} = \frac{18}{\sqrt{\delta_{P+0.1V}}} = 4.7\ [\text{Hz}]$$

<u>*Check vibration*</u>

Required minimum natural frequency:

$$f_{req} = 4.0 \text{ [Hz]}$$

$$\therefore\ f_{P+0.1V} > f_{req} \quad \text{OK} \left(\frac{f_{req}}{f_{P+0.1V}} = 0.85 \right)$$

<u>*Control of crack width*</u>

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:

$$\sigma_{sl,lim} = 240 \text{ [N/mm}^2\text{]} \left(\text{for } w_k = 0.3 \text{ [mm]} \quad \text{and} \quad \phi^{*} = 13 \text{ [mm]}\right)$$

Effective width in hogging moment region:

$$b_{effh} = b_{0h} + \min \left\{ \frac{2\left(L_b - x_0\right)}{4}; B_b - b_{0h} \right\} = 1244.4 \text{ [mm]}$$

Cross-sectional area of longitudinal rebars within b_{effh}:

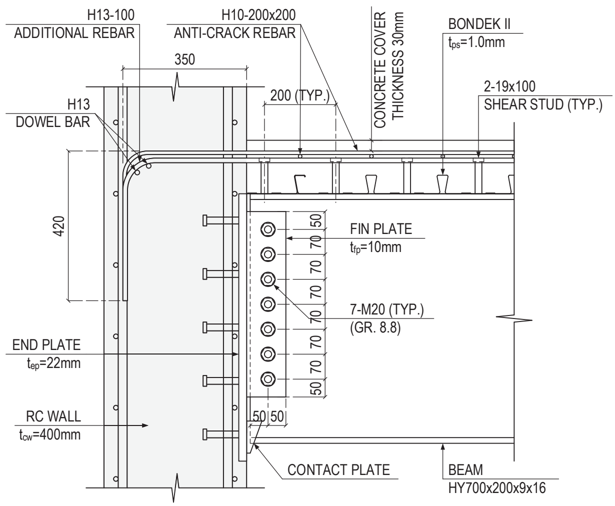
$$\begin{aligned} A_{sl} = {} & \pi \left(\frac{\phi_{sl,1}}{2} \right)^2 \left\lfloor \frac{b_{effh}}{p_{sl,1}} \right\rfloor \\ & + \pi \left(\frac{\phi_{sl,2}}{2} \right)^2 \min \left(\left\lfloor \frac{b_{effh}}{p_{sl,2}} \right\rfloor ; \left\lfloor \frac{b_{sl,2}}{p_{sl,2}} \right\rfloor \right) = 20.64 \text{ [cm}^2\text{]} \end{aligned}$$

Vertical distance between centre of un-cracked concrete flange and un-cracked composite section:

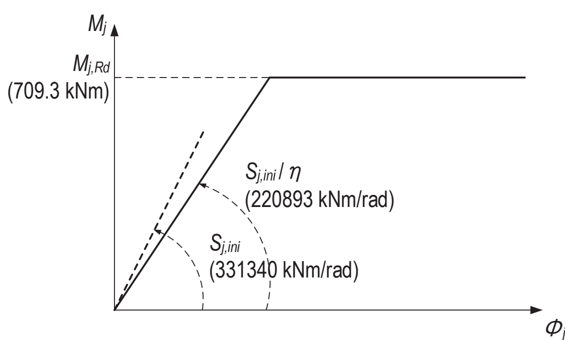
$$z_0 = \frac{\left(A_a + A_{sl}\right)\left(0.5D_a + D_{ps} + 0.5h_{cs}\right)}{A_a + A_{sl} + \left(\frac{h_{cs} b_{effh}}{n_0}\right)} = 194.8 \text{ [mm]}$$

Coefficient taking into account of stress distribution within section immediately prior to cracking:

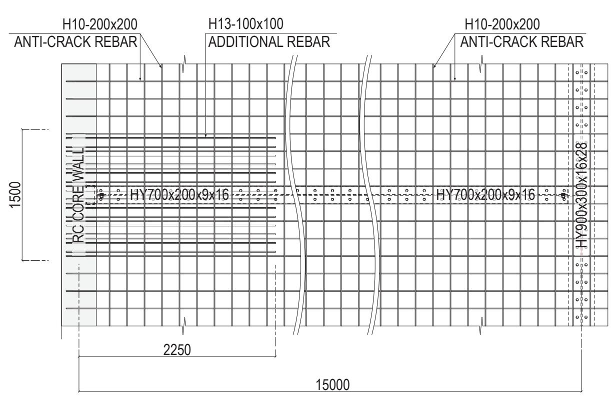
$$k_c = \min \left\{ \frac{1}{1 + \left(\frac{h_{cs}}{2z_0}\right)} + 0.3; 1.0 \right\} = 1.00$$



(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

Cross-sectional area of composite slab within b_{effh} above profiled steel sheeting:

$$A_{cs} = b_{effh} h_{cs} = 1231.9 \text{ [cm}^2\text{]}$$

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:

$$I_h = I_{ay} + \frac{A_a A_{sl} \left\{ D_a + 2 \left(D_{ps} + h_{cs} - z_{tcs\text{-}sl,eq} \right) \right\}^2}{4 \left(A_a + A_{sl} \right)} = 135951 \text{ [cm}^4\text{]}$$

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:

$$\sigma_{sl} = \sigma_{sl,0} + \Delta\sigma_{sl} = 161 \text{ [N/mm}^2\text{]}$$

$$\therefore \; \sigma_{sl} < \sigma_{sl,lim} \quad \text{OK} \left(\frac{\sigma_{sl}}{\sigma_{sl,lim}} = 0.67 \right)$$

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 Case1 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.

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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]	76.0 [kg/m]
			(−18.6%)	(−24.8%)

Table AV.2 Comparison of pinned joints and semi-rigid joints on weight of secondary steel beam under 15 [m] beam span

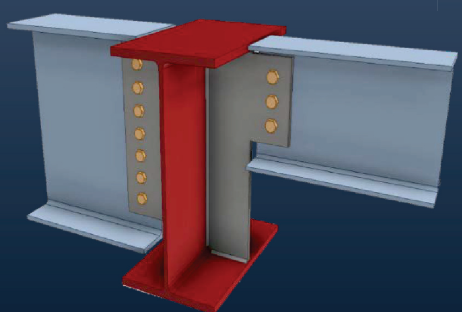
Case		Case 1	Case 2	Case 3
Beam span L_b			15.0 [m]	
Beam spacing B_b			3.0 [m]	
Design load	SDL $g_{k,3}$		3.0 [kN/m^2]	
	IL $q_{k,2}$		5.0 [kN/m^2]	
Joint classification		Pinned	Semi-rigid	Semi-rigid
Steel beam	Cross-section	UB762x267x147	UB610x229x125	JIS700x200x9x16
	Mass per metre	146.9 [kg/m]	125.1 [kg/m]	99.6 [kg/m]
			(−14.8%)	(−32.2%)

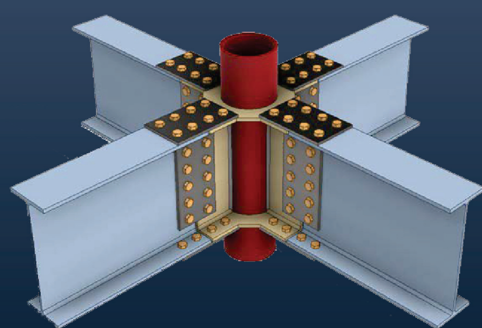
A complimentary book "Liew J Y R (2019), Design Guide for Buildable Steel Connections, Published by Singapoer Structural Steel Society, 671pp" can be downloaded from https://ssss.org.sg/~ssssorgs/images/stories/docs/Design_guide_for_buildable_steel_connections_Final_Version_20190327.pdf

Design Guide for Buildable Steel Connections

-Bolted and Welded Connection to SS EN1993-1-8

J Y Richard Liew





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Design of Steel-Concrete Composite Structures Using High-Strength Materials

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