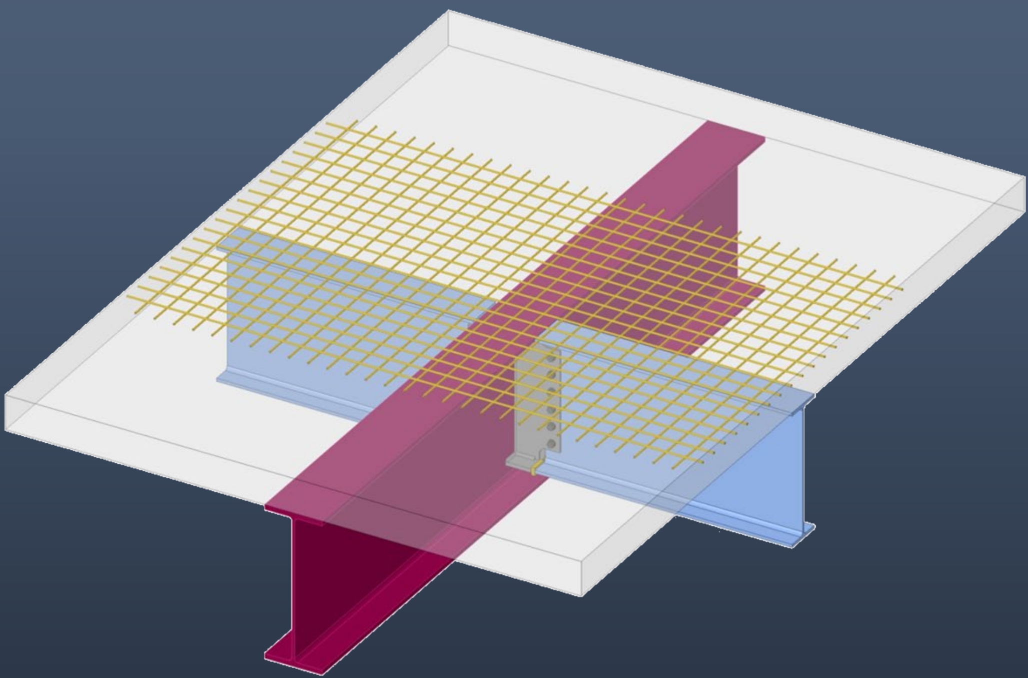
Design Guide for Semi-rigid Composite Joints and Beams

- Beam-to-Beam Composite Joints -



J Y Richard Liew & Yuichi Nishida

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DESIGN GUIDE FOR SEMI-RIGID COMPOSITE JOINTS AND BEAMS

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J Y Richard Liew &

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Forward

This publication is a follow-up with the previous work on design guide for buildable steel connections[1], which is meant for bolted and welded steel connection without considering the benefit of composite action between the steel connection and steel reinforcements in the concrete/composite slab.

For multi-storey composite buildings in which laterally stability resistance is provided by concrete core wall or steel bracing frame system, simple pin joints, such as fin plate bolted connections, are often used in beam-to-column and beam-to-beam joints. This is because these simplified joint details are relatively easier to install at site compared to moment joints and they are preferred in conditions where the structural frameworks are not subjected to significant horizontal loads. In modern commercial buildings, long span and open space floor beam layout plan are often preferred. Floor beams with semi-rigid end connections can achieve a more economical design without the need of complicated rigid joint detailing.

In EN 1993-1-8[2], joints may be classified as pin, semi-rigid or rigid in terms of their initial rotational stiffness and/or moment resistance depending on the analysis methods adopted in the design. In EN 1994-1-1[3], composite joints are defined as joints in which slab reinforcements are considered to calculate the rotational stiffness and moment resistance if the reinforcements are continuous over the joints. Therefore, in accordance with EN 1994-1-1, some of the simple pin joints defined in EN 1993-1-8 can be classified as semi-rigid joints if the reinforcing bars in the concrete slab are continuous over the joints and some degree of rotational restraint can be provided. Specifically, a mechanical model in which the tension force is transferred by reinforcing bars and compression force is transferred by bearing bolts can be assumed. However, there are often gaps between the bearing bolts and bolt holes that prevent the development of effective rotational restraint at the initial loading stage. Hence, this type of joints is designed as nominally pin joints in practice.

This book proposes a contact type of semi-rigid composite joints which can certainly develop higher rotational stiffness and moment resistance. In these joints, contact plates are inserted at the bottom flange of the steel beam. Although these contact plates are designed to transfer only the compression force, an effective measure is needed, either by bolts or welding, to ensure good contact.

This design guide is based on EN 1994-1-1 for the design of beam-to-beam composite joints with detailed methods developed for practical use. Design procedures for secondary composite beams with composite joints are also provided. This design guide will endow structural engineers with the confidence to use beam-to-beam composite joints in a safe and economic manner to design and construct composite structures.

[1](https://ssss.org.sg/~ssssorgs/images/stories/docs/Design_guide_for_buildable_steel_connections_Final_Version_20190327.pdf)

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Chapter 1 General

(1) Application

This design guide is applicable for the design of beam-to-beam composite joints and secondary composite beams with composite joints.

(2) Beam-to-beam composite joint with contact plates

This is a construction method for a composite floor system in which secondary composite beams are designed with semi-rigid joints by placing steel reinforcing bars in concrete slab continuous over the beam-to-beam joints and attaching contact plates at the bottom flange level of the secondary steel beams as shown in Figure 1.1.

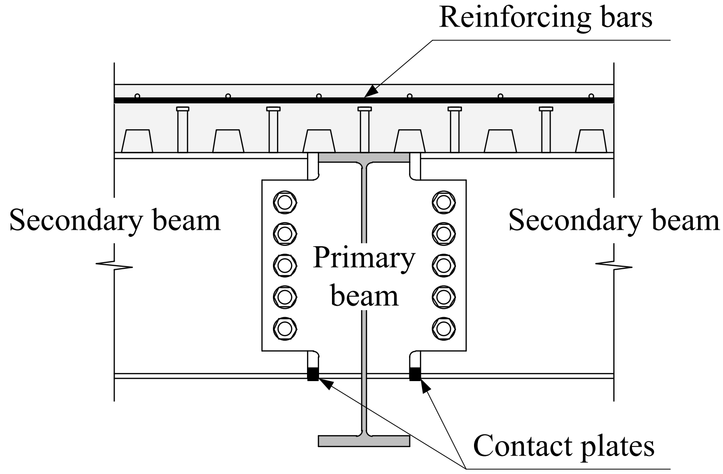


Figure 1.1: Beam-to-beam composite joint with contact plates

(3) Standard to be followed

This design guide is based on EN 1994-1-1 for the design of beam-to-beam 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.

(4) List of symbols

The following symbols are applied in this design guide.

A_a	is the cross-sectional area of secondary steel beam
A_b	is the tensile stress area of bolt
A_{bea}	is the bearing area between bottom flange of secondary steel beam and contact plate
A_{bf}	is the cross-sectional area of bottom flange of secondary steel beam
$A_{bwV,g}$	is the shear area of web of secondary steel beam for gross section
$A_{bwV,n}$	is the shear area of web of secondary steel beam for net section
A_c	is the area per unit length of concrete slab
$A_{c,c}$	is the area per unit length of concrete slab in compression

A_{cp}	is the cross-sectional area of contact plate
A_{cs}	is the cross-sectional area of composite slab within b_{effh} above profiled steel sheeting
a_{fp}	is the effective throat thickness of fillet weld of fin plate
$A_{fp,nt}$	is the net area of fin plate subjected to tension
$A_{fp,nV}$	is the net area of fin plate subjected to shear
$a_{fp,req}$	is the required minimum throat thickness of fillet weld of fin plate
$A_{fpV,n}$	is the shear area of fin plate for net section
A_{pse}	is the effective cross-sectional area of profiled steel sheeting per unit length
A_{sl}	is the cross-sectional area of longitudinal reinforcing bars within b_{effh}
$A_{sl,r}$	is the cross-sectional area of longitudinal reinforcing bars within $b_{eff,j}$ for a row r
$A_{sl,req}$	is the required minimum cross-sectional area of longitudinal reinforcing bars within b_{effh}
A_{st}	is the cross-sectional area of transverse reinforcing bars per unit length
$A_{st,req}$	is the required minimum cross-sectional area of transverse reinforcing bars per unit length
A_V	is the shear area of secondary steel beam
B_a	is the width of secondary steel beam
B_b	is the beam spacing
$b_{eff,b}$	is the effective width of secondary composite beam
b_{effh}	is the effective width of secondary composite beam on hogging moment region
$b_{eff,j}$	is the effective width of beam-to-beam composite joint
b_{effs}	is the effective width of secondary composite beam on sagging moment region
b_{eih}	is the value of effective width of secondary composite beam on each side of web of steel beam on hogging moment region
b_{eis}	is the value of effective width of secondary composite beam on each side of web of steel beam on sagging moment region
b_{ih}	is the distance from outstand headed stud to a point mid-way between adjacent webs of steel beams on hogging moment region
b_{is}	is the distance from outstand headed stud to a point mid-way between adjacent webs of steel beams on sagging moment region
b_{sl}	is the arrangement width of additional longitudinal reinforcing bars
b_{0h}	is the distance between centres of outstand headed studs on hogging moment region
$b_{0,max}$	is the maximum width for re-entrant of profiled steel sheeting
$b_{0,min}$	is the minimum width for re-entrant of profiled steel sheeting
b_{0s}	is the distance between centres of outstand headed studs on sagging moment region
C_1	is the correction factor for non-uniform bending moment
C_4	is the property of distribution of moment
D_a	is the depth of secondary steel beam

D_{cs}	is the overall depth of composite slab
D_{fp}	is the depth of fin plate
d_{hs}	is the diameter of shank of headed stud
D_{ps}	is the overall depth of profiled steel sheeting
d_0	is the hole diameter of bolt
e	is the *e* value
E_a	is the modulus of elasticity of secondary steel beam
$e_{b\text{-}bw,h}$	is the edge distance for web of secondary beam on horizontal line
$e_{b\text{-}bw,v}$	is the edge distance for web of secondary beam on vertical line
$e_{b\text{-}fp,h}$	is the edge distance for fin plate on horizontal line
$e_{b\text{-}fp,v}$	is the edge distance for fin plate on vertical line
E_{cm}	is the secant modulus of elasticity of normal weight concrete
E_{fp}	is the modulus of elasticity of fin plate
$(EI)_h$	is the hogging flexural rigidity of secondary composite beam
$(EI)_s$	is the sagging flexural rigidity of secondary composite beam
E_{lcm}	is the secant modulus of elasticity of lightweight concrete
E_s	is the modulus of elasticity of reinforcing bars
f_{au}	is the ultimate tensile resistance of secondary steel beam
f_{ay}	is the nominal value of yield strength of secondary steel beam
f_{ayd}	is the design yield strength of secondary steel beam
f_{bu}	is the ultimate tensile strength of bolt
$F_{bV,Rd}$	is the shear resistance of a single bolt
f_{by}	is the nominal value of yield strength of bolt
f_{cd}	is the design strength of normal weight concrete
f_{ck}	is the characteristic cylinder strength of normal weight concrete
f_{cpy}	is the nominal value of yield strength of contact plate
f_{cpyd}	is the design yield strength of contact plate
f_{ctm}	is the mean value of tensile strength of concrete
f_{fpu}	is the ultimate tensile strength of fin plate
f_{fpy}	is the nominal value of yield strength of fin plate
$F_{hbb,Rd}$	is the horizontal bearing resistance of a single bolt
f_{hsu}	is the ultimate strength of headed stud
f_{lck}	is the characteristic cylinder strength of lightweight concrete
f_{psd}	is the design yield strength of profiled steel sheeting
f_{psk}	is the characteristic yield strength of profiled steel sheeting
$f_{P+0.1V}$	is the natural frequency due to "dead loads, superimposed dead loads, and 10% of live loads"
f_{req}	is the required minimum natural frequency
f_{sd}	is the design yield strength of reinforcing bars
f_{sk}	is the characteristic yield strength of reinforcing bars
$F_{vbb,Rd}$	is the vertical bearing resistance of a single bolt
f_{wu}	is the ultimate tensile strength of web of secondary steel beam
f_{wy}	is the nominal value of yield strength of web of secondary steel beam
G_a	is the shear modulus of elasticity of secondary steel beam

g_a	is the mass per metre of secondary steel beam
$g_{k,1}$	is the dead load per unit area in construction stage
$g_{k,2}$	is the dead load per unit area in composite stage
$g_{k,3}$	is the superimposed dead load per unit area in composite stage
g_{ps}	is the mass per metre of profiled steel sheeting
h_{cs}	is the thickness of composite slab above profiled steel sheeting
$h_{eff,j}$	is the effective length of beam-to-beam composite joint
h_{hs}	is the overall height of headed stud
i_{ax}	is the polar radius of gyration of area of secondary steel beam
I_{ay}	is the second moment of area of secondary steel beam about major axis (y-y axis)
I_{az}	is the second moment of area of secondary steel beam about minor axis (z-z axis)
I_b	is the second moment of area of secondary composite beam
I_{bfz}	is the second moment of area of bottom flange of secondary steel beam about minor axis (z-z axis)
I_{cs2}	is the second moment of area of cracked composite slab in direction transverse to secondary steel beam
I_h	is the second moment of area of secondary composite beam on hogging moment region
i_{fpz}	is the radius of gyration of area of fin plate about minor axis (z-z axis)
$I_{T,a}$	is the torsion constant of secondary steel beam
$I_{w,a}$	is the warping constant of secondary steel beam
k_c	is the coefficient taking into account of stress distribution within section immediately prior to cracking; k_c factor
k_s	is the transverse (rotational) stiffness per unit length of secondary composite beam
k_{sc}	is the stiffness of one headed stud
K_{sc}	is the stiffness related to headed studs
k_{slip}	is the stiffness reduction factor due to deformation of headed studs
$k_{sl,r}$	is the stiffness coefficient of longitudinal reinforcing bars for a row r
$k_{sl,eq}$	is the equivalent stiffness coefficient of longitudinal reinforcing bars
k_{th}	is the reduction factor for shear resistance of a headed stud on hogging moment region
$k_{th,max}$	is the maximum reduction factor for shear resistance of a headed stud on hogging moment region
$k_{t,max}$	is the maximum reduction factor for shear resistance of a headed stud
k_{ts}	is the reduction factor for shear resistance of a headed stud on sagging moment region
$k_{ts,max}$	is the maximum reduction factor for shear resistance of a headed stud on sagging moment region
$k_{\tau,min}$	is the minimum shear buckling coefficient
k_1	is the flexural stiffness of cracked composite slab in direction transverse to secondary steel beam

$k_{1,hbb}$	is the k_1 factor for horizontal bolt bearing resistance
$k_{1,vbb}$	is the k_1 factor for vertical bolt bearing resistance
k_2	is the flexural stiffness of web of secondary steel beam
l	is the length of secondary composite beam in hogging moment region adjacent to joint
L_b	is the beam length; the beam span
$L_{b,A}$	is the beam length of composite beam (A)
$L_{b,B}$	is the beam length of composite beam (B)
$L_{b,l}$	is the beam length on left side
$L_{b,r}$	is the beam length on right side
l_b	is the basic anchorage length of longitudinal reinforcing bars
L_{cr}	is the length of secondary composite beam between points at which bottom flange is laterally restrained
$L_{cr,a}$	is the length of secondary steel beam between points at which top flange of steel beam is laterally restrained
L_e	is the distance between inflection points
L_{eh}	is the distance between inflection points on hogging moment region
L_{es}	is the distance between inflection points on sagging moment region
l_0	is the design lap length of longitudinal reinforcing bars
$l_{0,min}$	is the minimum lap length of longitudinal reinforcing bars
$M_{b,a,Rd}$	is the buckling moment resistance of laterally unrestrained secondary steel beam
$M_{b,fp,Rd}$	is the lateral torsional buckling moment resistance of fin plate
$M_{b,Rd}$	is the buckling moment resistance of laterally unrestrained secondary composite beam
M_{cr}	is the elastic critical moment for lateral-torsional buckling of secondary composite beam
$M_{cr,a}$	is the elastic critical moment for lateral-torsional buckling of secondary steel beam
M_{Edh}	is the design hogging moment
$M_{Edh,A}$	is the design hogging moment of composite beam (A)
$M_{Edh,B}$	is the design hogging moment of composite beam (B)
M_{Eds}	is the design sagging moment
$M_{Eds,A}$	is the design sagging moment of composite beam (A)
$M_{Eds,B}$	is the design sagging moment of composite beam (B)
$M_{el,fp,Rd}$	is the elastic moment resistance of fin plate
$M_{el,vbw,Rd}$	is the elastic moment resistance of web of secondary steel beam on vertical line of bolts
$M_{h,A}$	is the actual end moment of composite beam (A)
$M_{h,(wA,max)}$	is the end moment of composite beam (A) due to $w_{A,max}$; the released moment
$M_{h,(wB,min)}$	is the end moment of composite beam (B) due to $w_{B,min}$; the released moment
M_j	is the joint moment
$M_{j,Rd}$	is the yield moment resistance of beam-to-beam composite joint
$M_{pl,a,Rd}$	is the plastic moment resistance of secondary steel beam

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

$M_{plf,Rdh}$ is the plastic hogging moment resistance of secondary composite beam with full shear connection

$M_{plf,Rds}$ is the plastic sagging moment resistance of secondary composite beam with full shear connection

$M_{plp,Rds}$ is the plastic sagging moment resistance of secondary composite beam with partial shear connection

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

$M_{pl,Rkh}$ is the characteristic value of plastic hogging moment resistance of secondary composite beam

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

$M_{y,v,Rdh}$ is the reduced moment resistance of secondary composite beam making allowance for presence of shear force

$M_{y,v,vbw,Rd}$ is the reduced moment resistance of web of secondary steel beam on vertical line of bolts making allowance for presence of shear force

M_0 is the mid-length moment of simply supported beam

N is the number of headed studs distributed over length l

$n_{b,h}$ is the number of bolts on horizontal line

$n_{b,v}$ is the number of bolts on vertical line

n_{hs} is the number of headed studs per sheeting rib

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

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

n_{hss} is the number of headed studs per sheeting rib on sagging moment region

N_{hss} is the number of headed studs arranged within half of L_{es}

N_L is the longitudinal force in composite slab

n_0 is the modular ratio for short-term loading

$p_{b,h}$ is the pitch of bolts on horizontal line

$p_{b,v}$ is the pitch of bolts on veritical line

p_{ps} is the pitch of ribs of profiled steel sheeting

P_{Rd} is the shear resistance of a headed stud

p_{sl} is the pitch of longitudinal reinforcing bars

$p_{sl,lim}$ is the limit of spacing of longitudinal reinforcing bars

p_{st} is the pitch of transverse reinforcing bars

$q_{k,1}$ is the construction load per unit area ~~in~~ construction stage

$q_{k,2}$ is imposed floor load per unit area in composite stage

r is the root radius of secondary steel beam

R_a is the tension (compression) resistance of secondary steel beam

$R_{eff,a}$ is the tension (compression) resistance of effective secondary steel beam

$R_{eff,v}$ is the tension (compression) resistance of effective clear web of secondary steel beam

R_{con} is the compression resistance of contact part

R_{cs} is the compression resistance of composite slab within b_{effs}

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

R_{qh} is the longitudinal shear force transfer within half of L_{eh}
R_{qs} is the longitudinal shear force transfer within half of L_{es}
$R_{st}+R_{pse}$ is the tension resistance of transverse reinforcement per unit length
R_{sl} is the tension resistance of longitudinal reinforcing bars within b_{effh}
$R_{sl,j}$ is the tension resistance of longitudinal reinforcing bars within $b_{eff,j}$
$R_{tr,\ req}$ is the required tension resistance of transverse reinforcement per unit length
R_v is the tension (compression) resistance of clear web of secondary steel beam
R_w is the tension (compression) resistance of overall web of secondary steel beam
s_{fp} is the length of fillet weld of fin plate
S_j is the rotational stiffness of beam-to-beam composite joint
$S_{j,A}$ is the rotational stiffness of beam-to-beam composite joint applied to composite beam (A)
$S_{j,B}$ is the rotational stiffness of beam-to-beam composite joint applied to composite beam (B)
$S_{j,ini}$ is the initial rotational stiffness of beam-to-beam composite joint
t_f is the flange thickness of secondary steel beam
t_{fp} is the thickness of fin plate
t_{ps} is the thickness of profiled steel sheeting
t_w is the web thickness of secondary steel beam
$V_{b,a,Rd}$ is the shear buckling resistance of secondary steel beam
$V_{bb,Rd}$ is the bolt bearing resistance
$V_{b,Rd}$ is the shear buckling resistance of secondary composite beam; the bolt shear resistance
$V_{bw,Rd,g}$ is the shear resistance of web of secondary steel beam for gross section
$V_{bw,Rd,n}$ is the shear resistance of web of secondary steel beam for net section
V_{Ed} is the design shear force
$V_{fp,Rd,b}$ is the fin plate block shear resistance
$V_{fp,Rd,g}$ is the fin plate shear resistance for gross section
$V_{fp,Rd,n}$ is the fin plate shear resistance for net section
$v_{L,Ed}$ is the design longitudinal shear stress in composite slab
$V_{pl,a,Rd}$ is the plastic shear resistance of secondary steel beam
$V_{pl,hbw,Rd}$ is the plastic shear resistance of web of secondary steel beam on top and bottom horizontal line of bolts
$V_{pl,Rd}$ is the plastic shear resistance of secondary composite beam
v_{Rd} is the crushing shear stress of concrete slab
$V_{vbw,Ed}$ is the design shear force of web of secondary steel beam on vertical line of bolts
w is the uniformly distributed load
$w_{A,max}$ is the maximum uniformly distributed load of composite beam (A)
$w_{A,min}$ is the minimum uniformly distributed load of composite beam (A)
$w_{B,max}$ is the maximum uniformly distributed load of composite beam (B)
$w_{B,min}$ is the minimum uniformly distributed load of composite beam (B)
$w_{C,max}$ is the maximum uniformly distributed load of composite beam (C)

$w_{C,min}$	is the minimum uniformly distributed load of composite beam (C)
$w_{com,max}$	is the maximum design distributed load in composite stage
$w_{com,min}$	is the minimum design distributed load in composite stage
$w_{com,P}$	is the design distributed load due to "superimposed dead loads" in composite stage
$w_{com,P+0.1V}$	is the design distributed load due to "dead loads, superimposed dead loads, and 10% of live loads"
$w_{com,V,max}$	is the maximum design distributed load due to "live loads" in composite stage
$w_{com,V,min}$	is the minimum design distributed load due to "live loads" in composite stage
$w_{con,max}$	is the maximum design distributed load in construction stage
$w_{con,P}$	is the design distributed load due to "dead loads" in construction stage
$w_{con,P+V}$	is the design distributed load due to "dead loads and live loads" in construction stage
$w_{con,V}$	is the design distributed load due to "live loads" in construction stage
$W_{eff,pl,a}$	is the effective plastic section modulus of secondary steel beam
w_k	is the design crack width
$W_{pl,a}$	is the plastic section modulus of secondary steel beam
x_δ	is the x-coordinate where deflection is maximized
x_0	is the x-coordinate at inflection point
$z_{ccs\text{-}ca}$	is the vertical distance between centre of composite slab and centre of secondary steel beam
$z_{sl,eq\text{-}ca}$	is the equivalent vertical distance between longitudinal reinforcing bars and centre of steel beam
$z_{sl,eq\text{-}cc}$	is the equivalent vertical distance between longitudinal reinforcing bars and centre of contact part
$z_{sl,eq\text{-}na}$	is the equivalent vertical distance between longitudinal reinforcing bars and neutral axis of secondary composite beam
$z_{sl,eq\text{-}tf}$	is the equivalent vertical distance between longitudinal reinforcing bars and top of flange of secondary steel beam
$z_{csl,r\text{-}cc}$	is the vertical distance between centre of longitudinal reinforcing bars and centre of contact part for a row r
$z_{csl\text{-}tf}$	is the vertical distance between centre of longitudinal reinforcing bars and top of flange of secondary steel beam
$z_{st,eq\text{-}ccs,c}$	is the equivalent vertical distance between transverse reinforcing bars and centre of concrete slab in compression
$z_{st,eq\text{-}na}$	is the equivalent vertical distance between transverse reinforcing bars and neutral axis of composite slab
$z_{cst\text{-}na}$	is the vertical distance between centre of transverse reinforcing bars and neutral axis of secondary composite beam
$z_{ctf\text{-}cbf}$	is the vertical distance between centres of top and bottom flanges of secondary steel beam
$z_{fs\text{-}b}$	is the distance between face of support and assumed line of shear transfer
$z_{na\text{-}ccs,c}$	is the vertical distance between neutral axis of composite slab and centre of concrete slab in compression

$z_{tcs\text{-}csl}$	is the covering depth of longitudinal reinforcing bars
$z_{tcs\text{-}cst}$	is the covering depth of transverse reinforcing bars
z_0	is the vertical distance between centre of un-cracked concrete flange and un-cracked composite section
α	is the portion of part of cross-section in compression; the α factor
α_{bV}	is the correction factor for bolt shear resistance
α_{hs}	is the correction factor of headed stud taking into account h_{hs}/d_{hs}
α_{LT}	is the imperfection factor corresponding to appropriate lateral-torsional buckling curve
α_{hbb}	is the correction factor for horizontal bolt bearing resistance
α_{vbb}	is the correction factor for vertical bolt bearing resistance
α_1	is the coefficient considering shape of bars
α_2	is the coefficient considering consrete 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
β	is the β factor
γ_a	is the partial factor of resistance of members and cross-sections of secondary steel beam
$\gamma_{a,2}$	is the partial factor of resistance of secondary steel beam in bearing
γ_b	is the partial factor of bolt
γ_c	is the partial factor of concrete
γ_{cp}	is the partial factor of resistance of members and cross-sections of contact plate
$\gamma_{cp,2}$	is the partial factor of resistance of contact plate in bearing
γ_{fp}	is the partial factor of resistance of members and cross-sections of fin plate
$\gamma_{fp,2}$	is the partial factor of resistance of fin plate in bearing
$\gamma_{G,sup}$	is the partial factor for permanent actions (unfavourable)
$\gamma_{G,inf}$	is the partial factor for permanent actions (favourable)
γ_{ps}	is the partial factor of profiled steel sheeting
γ_Q	is the partial factor for variable actions (unfavourable)
γ_{Qi}	is the partial factor for variable actions (favourable)
γ_s	is the partial factor of reinforcing bars
γ_V	is the partial factor of headed stud
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
δ_A	is the deflection of composite beam (A)
δ_B	is the deflection of composite beam (B)
δ_{max}	is the maximum deflection of secondary composite beam
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”

$\delta_{P+0.1V}$	is the deflection due to "dead loads, superimposed dead loads, and 10% of live loads"
δ_{tP}	is the deflection due to "dead loads and superimposed dead loads"
δ_{tP+V}	is the deflection due to "dead loads, superimposed dead loads, and live loads"
δ_V	is the deflection due to "live loads"
$\delta_{V,lim}$	is the limit of deflection due to "live loads"
η	is the stiffness modification coefficient
η_h	is the degree of shear connection on hogging moment region
$\eta_{h,req}$	is the required minimum degree of shear connection on hogging moment region
η_s	is the degree of shear connection on sagging moment region
$\eta_{s,req}$	is the required minimum degree of shear connection on sagging moment region
θ	is the angle between diagonal strut and axis of secondary beam
θ_{min}	is the minimum angle to minimize cross-sectional area of transverse reinforcing bars
θ_{pin}	is the rotation of pin joint
λ_{LT}	is the non-dimensional slenderness for lateral-torsional buckling of secondary composite beam
$\lambda_{LT,a}$	is the non-dimensional slenderness for lateral-torsional buckling of secondary steel beam
$\lambda_{LT,fp}$	is the non-dimensional slenderness for lateral torsional buckling of fin plate
λ_w	is the modified slenderness of web of secondary steel beam
μ_A	is the distribution factor for composite beam (A)
ν	is the parameter related to deformation of headed studs
ξ	is the parameter related to deformation of headed studs
ρ_c	is the dry density of normal weight concrete concrete
ρ_{lc}	is the dry density of lightweight concrete
$\rho_{sl,req}$	is the required minimum reinforcement ratio
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
$\sigma_{sl,0}$	is the stress in longitudinal reinforcing bars caused by M_{Edh}
Φ_{LT}	is the value to determine reduction factor for lateral-torsional buckling of secondary composite beam
$\Phi_{LT,a}$	is the value to determine reduction factor for lateral-torsional buckling of secondary steel beam
$\Phi_{LT,fp}$	is the value to determine reduction factor for lateral-torsional buckling of fin plate
ϕ_j	is the joint rotation
ϕ_{sl}	is the diameter of longitudinal reinforcing bars
$\phi_{sl,r}$	is the diameter of longitudinal reinforcing bars for a row r

ϕ_{sl}^*	is the maximum diameter of longitudinal reinforcing bars
$\phi_{st,r}$	is the diameter of transverse reinforcing bars for a row r
χ_{LT}	is the reduction factor for lateral-torsional buckling of secondary composite beam
$\chi_{LT,a}$	is the reduction factor for lateral-torsional buckling of secondary steel beam
$\chi_{LT,fp}$	is the reduction factor for lateral-torsional buckling of fin plate
χ_w	is the factor for contribution of web of secondary steel beam to shear buckling resistance
ψ	is the ratio of the design hogging moment to M_0

Chapter 2 Materials

2.1 Structural Steel

(1) Yield strength

The nominal value of yield strength of secondary 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 primary steel beams, fin plates, stiffeners, and contact plates should be at least a matching grade as the secondary beam but cannot be higher than 460 [N/mm^2].

(2) Steel grade

The typical steel grades of structural steel given in Table 2.1 can be used for the design of beam-to-beam composite joints and secondary composite beams with composite joints.

Table 2.1: Steel grades of structural steel

Steel grade	Nominal values of yield strength / Ultimate tensile strength [N/mm^2] with thickness [mm] less than or equal to					
	16	40	63	80	100	150
S235	235 / 360	225 / 360	215 / 360	215 / 360	215 / 360	195 / 350
S275	275 / 410	265 / 410	255 / 410	245 / 410	235 / 410	225 / 400
S355	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) can be also used only if they are certified in accordance 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. On the other hand, the partial factor of the resistance of plates in bearing should be taken as 1.25.

2.2 Concrete

(1) Strength classes

The concrete strength classes given in Table 2.2 and Table 2.3 should be conformed to for the design of beam-to-beam composite joints and secondary composite beams with composite joints. The concrete strength classes lower than C20/25 (LC20/22) and higher than C60/75 (LC60/66) are not covered in this design guide.

(2) Other properties

Unless otherwise given by this design guide, other concrete properties can be referred to EN 1992-1-1[5] for both normal weight concrete and lightweight concrete.

Table 2.2: Strength classes of normal weight 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: Strength classes of lightweight concrete

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

where

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

(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 beam-to-beam composite joints and secondary composite beams with composite joints.

Table 2.4: Strength classes of reinforcing steel

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

(3) Modulus of elasticity

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

(4) Partial factor

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

2.4 Shear Studs

(1) Mechanical characteristics and nominal dimensions

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

(2) Weldability and welding examination

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

(3) Shear resistance

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

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

α_{hs} is given by:

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

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

where

f_{hsu}	is the ultimate strength of headed stud
d_{hs}	is the diameter of the shank of headed stud
γ_V	is the partial factor of headed stud
f_{ck}	is the characteristic cylinder strength of normal weight concrete
E_{cm}	is the secant modulus of elasticity of normal weight concrete
α_{hs}	is the correction factor of headed stud taking into account h_{hs}/d_{hs}
h_{hs}	is the overall height of headed stud

(4) Alternative shear studs

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

(5) Partial factor

The partial factor of headed stud, γ_V, should be taken as 1.25.

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 the provisions in BC1:2012.

(3) Partial factor

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

2.6 Bolts

(1) Tensile strength

The nominal value of tensile strength of bolts f_{ub} shall be in the range of 300 [N/mm^2] to 1200 [N/mm^2].

(2) Strength classes

The strength classes of bolts given in Table 2.5 can be used for the design of beam-to-beam composite joints and secondary composite beams with composite joints.

Table 2.5: Strength classes of bolts

Strength class	4.6	4.8	5.6	5.8	6.8	8.8	10.9
Nominal value of yield strength f_{by} [N/mm^2]	240	320	300	400	480	640	900
Ultimate tensile strength f_{bu} [N/mm^2]	400	400	500	500	600	800	1000

(3) Alternative strength classes

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

(4) Partial factor

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

Chapter 3 Scope of Application

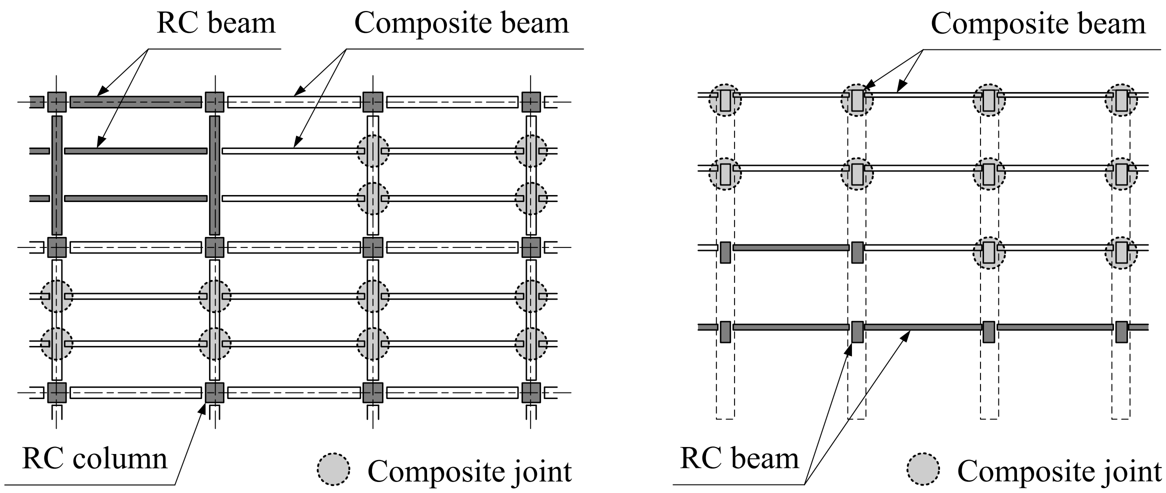
3.1 General

(1) Structural type

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

(2) Beam members

Beam members should be the composite beams where steel beams and floor slab are connected through shear studs. Different types of beam members not based on this design guide, such as reinforced concrete beams and steel encased reinforced concrete beams, can be combined in a same structure. However, this design guide is only applicable to the composite steel-concrete beams and composite joints as shown in Figure 3.1.



(a) Combination on a same floor plan (b) Combination on a same elevation

Figure 3.1: Floor plan and elevation views showing a combination of different types of beam members

(3) Column members

Column members are not limited as long as the above applicable conditions are satisfied. Beam-to-column joints are not covered in this design guide and will be considered in future revision.

Commentary:

(2) Beam members

The beam members should not be subject to excessive axial force that could affect the moment and rotational behaviour of the connections.

(3) Column members

Steel columns, reinforced concrete columns, steel encased reinforced concrete columns, concrete filled steel tubular columns, and other types of column members can be used.

3.2 Steel Beams

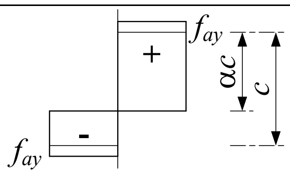
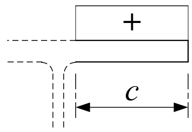
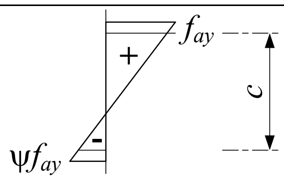
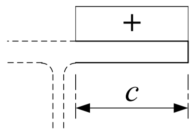
(1) Steel section

The steel section of secondary beams 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 3.2 should be used for secondary beams. Effective Class 2 cross-sections where the web is Class 3 and the compression flange is Class 1 or 2 can be also used.

Table 3.1: Range of cross-sectional dimensions of I-section

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

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

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

Commentary:

(2) Classification of cross-section

As described in Chapter 4, the plastic moment resistances of secondary beams are checked at ultimate limit state. Therefore, local buckling of web and flange should be prevented so that secondary beams can develop their plastic moment resistance.

3.3 Floor Slab

(1) Slab arrangement

Floor slab should be arranged on the 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 slabs with profiled steel sheeting.

(3) Reinforcing bars

The longitudinal reinforcing bars in concrete slab should be continuous over the beam-to-beam joints.

(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 design limit states.

Commentary:

(2) Slab type

In the case of using other types of floor slab in which rotational restraint may not be exhibited at the beam-to-beam joints, the joints should be designed as nominally pinned joints.

(3) Reinforcing bars

In circumstances that the reinforcing bars in the slab cannot be continuous over the primary beam and the beam-to-beam joints cannot develop sufficient rotational restraint, the joints should be designed as nominally pinned joints.

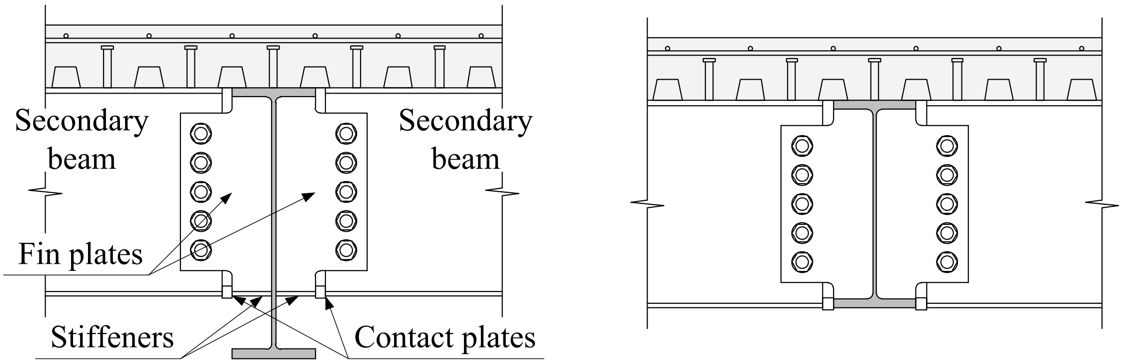
3.4 Beam-to-Beam Joints

(1) Joint type

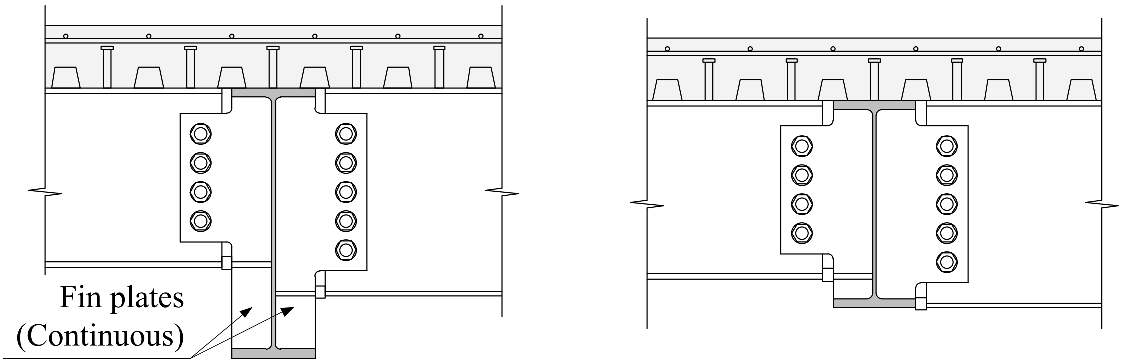
The joint type should be the double-sided type consists of a primary beam bolt-connected to double-sided secondary beams with extended fin plates as shown in Figure 3.2. In the case for the secondary beams of different depths shown in Figure 3.2(b), fin plates should be continuous between the top and bottom flanges to prevent out-plane deformation of the primary beam. In addition, the shear resistance of the panel zone formed by the fin plates, stiffeners and bottom flange should be checked.

(2) Contact plates and stiffeners

As shown in Figure 3.2, contact plates and stiffeners should be attached at the bottom flange level of secondary steel beams to ensure the initial rotational stiffness of beam-to-beam joints.



(a) Case for secondary beams of same depth



(b) Case for secondary beams of different depth

Figure 3.2: Double-sided with extended fin plate bolted type

Commentary:

(1) Joint type

The one-sided joint type where a secondary beam is attached from one side of a primary beam may be required in the outer periphery and around the voids as shown in Figure 3.3.

This type of joints should be designed as nominally pinned joints because the longitudinal reinforcing bars in concrete slab are not continuous over the beam-to-beam joints and they cannot transfer the tension force unless special measures are taken to ensure proper anchorage of the reinforcing bars.

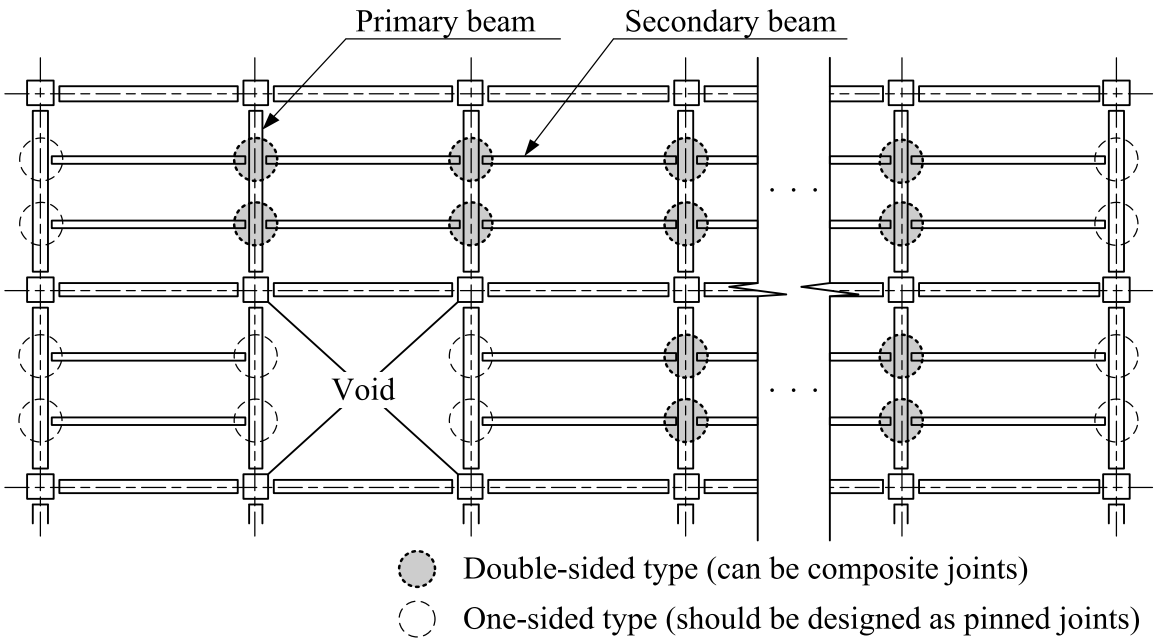


Figure 3.3: Floor beam layout

Chapter 4 Design of Beam-to-Beam Composite Joint and Beam

4.1 General

(1) Basis of design

In the design of beam-to-beam composite joints and secondary composite beams with composite joints in accordance with the provisions of this chapter, the design moment and deflection of the secondary composite beams are analysed considering the structural properties of the beam-to-beam composite joints classified as semi-rigid joints, and then all design criteria are checked for satisfaction.

(2) Design flow

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

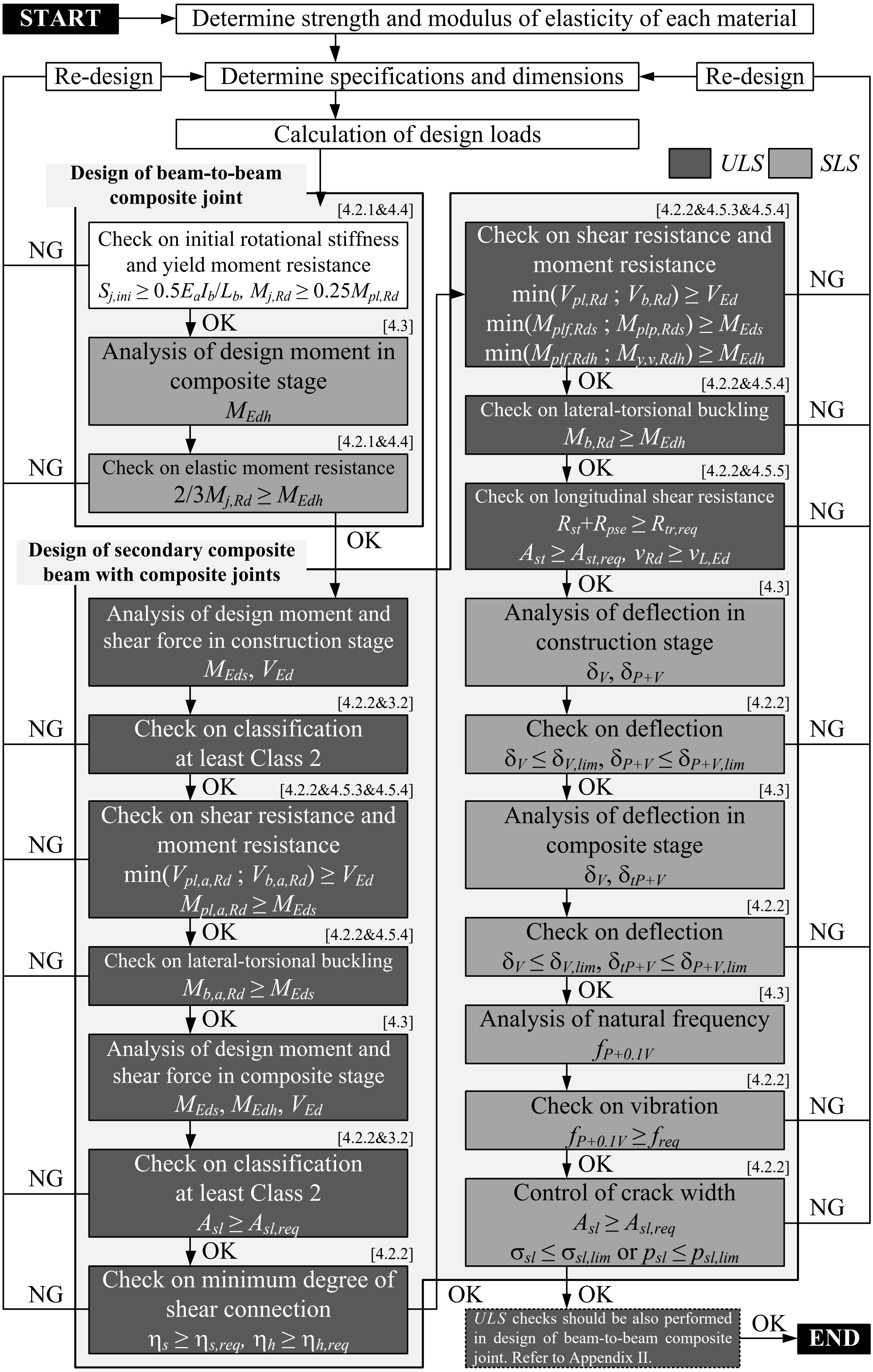


Figure 4.1: Design flow for beam-to-beam composite joint and secondary composite beam

Commentary:

(2) Design flow

The following design should be performed in accordance with this design flow.

a) Design of beam-to-beam composite joint

Based on the design criteria in subsection 4.2.1, joint classification should be conducted to check if the beam-to-beam composite joints are classified as semi-rigid. The initial rotational stiffness and the yield moment resistance can be determined according to subsection 4.4.2 and 4.4.3, respectively. In addition, serviceability check should also be carried out. The design hogging moments at the beam ends can be obtained by carrying out analysis of composite beam with end restraints in accordance with Section 4.3.

b) Design of secondary composite beam with composite joints

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 4.5.2, 4.5.3, 4.5.4, and 4.5.5, 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 4.3.

4.2 Design Criteria

4.2.1 Beam-to-Beam Composite Joint

(1) Joint classification

The following criteria of initial rotational stiffness and yield moment resistance should be satisfied so that beam-to-beam composite joints can be classified as semi-rigid.

a) Initial rotational stiffness

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

I_b is given by:

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

where

$S_{j,ini}$	is the initial rotational stiffness of beam-to-beam composite joint, see 4.4.2
E_a	is the modulus of elasticity of secondary steel beam
L_b	is the beam length
I_b	is the second moment of area of secondary composite beam
A_a	is the cross-sectional area of secondary 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 secondary steel beam
E_{cm}	is the secant modulus of elasticity of normal weight concrete
$b_{eff,b}$	is the effective width of secondary composite beam
I_{ay}	is the second moment of area of secondary steel beam about major axis (y-y axis)

b) Yield moment resistance

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

where

$M_{j,Rd}$	is the yield moment resistance of beam-to-beam composite joint, see 4.4.3
$M_{pl,Rd}$	is the plastic moment resistance of secondary composite beam

(2) Serviceability check in composite stage

The following criteria of elastic moment resistance at serviceability limit state should be satisfied to control the crack width of floor slab.

$$\frac{2}{3} M_{j,Rd} \geq M_{Edh} \tag{4.4}$$

where

$M_{j,Rd}$	is the yield moment resistance of beam-to-beam composite joint
M_{Edh}	is the design hogging moment

Commentary:

(1) Joint classification

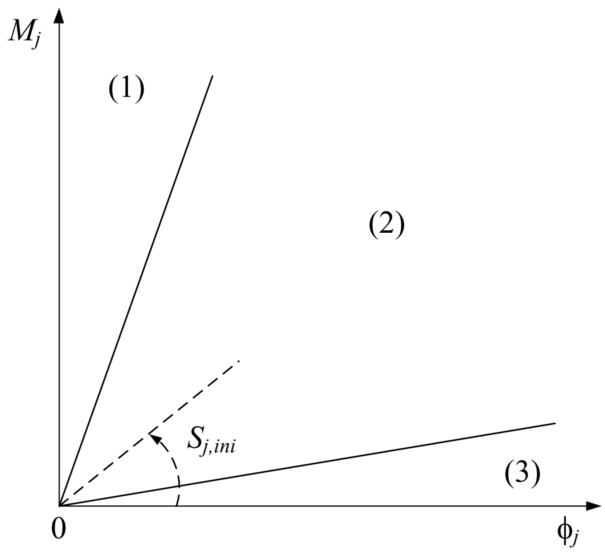
There are various definitions of semi-rigid joints for the structural analysis, and one of the representative joint classification schemes is described in EN 1993-1-8. According to EN 1993-1-8, joints are classified as rigid joints, nominally pinned joints, or semi-rigid joints by comparing the initial rotational stiffness of the joints $S_{j,ini}$ with the boundaries shown in Figure 4.2. 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.

In addition, joints are also classified as full-strength joints, nominally pinned joints, or partial-strength joints by comparing the yield 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 secondary composite beams are designed with semi-rigid joints. Therefore, beam-to-beam composite joints should satisfy the following design criteria on the initial rotational stiffness and the yield moment resistance based on the above joint classification scheme.

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

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



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

(2) Serviceability check in composite stage

When beam-to-beam composite joints are classified as semi-rigid joints, it is concerned that cracking of floor slab is occurred 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 in EN 1992-1-1, the limiting value of the crack width, 0.3 [mm], is recommended for reinforced members in terms of the functioning and durability. The crack width is related to the tensile stress in reinforcing bars, so that the design criteria on the longitudinal reinforcing bars to control the crack width of floor slab are imposed on the design of secondary composite beam with beam-to-beam composite joints as described in subsection 4.2.2. However, according to the full-scale joint component tests[11], it is desirable to keep the beam-to-beam composite joints elastic at serviceability limit state in order to limit the crack width below the value recommended in EN 1992-1-1. Therefore, beam-to-beam composite joints should satisfy the following design criteria on the elastic moment resistance.

$$\frac{2}{3} M_{j,Rd} \geq M_{Edh} \tag{4.4}$$

4.2.2 Secondary Composite Beam with Composite Joints

(1) Structural resistance check in construction stage

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

a) Classification of cross-section

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

b) Shear resistance

$$\min\left(V_{pl,a,Rd}\,;\,V_{b,a,Rd}\right) \geq V_{Ed} \tag{4.5}$$

where

$V_{pl,a,Rd}$	is the plastic shear resistance of secondary steel beam, see 4.5.3
$V_{b,a,Rd}$	is the shear buckling resistance of secondary steel beam, see 4.5.3
V_{Ed}	is the design shear force

c) Moment resistance

$$M_{pl,a,Rd} \geq M_{Eds} \tag{4.6}$$

where

$M_{pl,a,Rd}$	is the plastic moment resistance of secondary steel beam, see 4.5.4
M_{Eds}	is the design sagging moment

d) Lateral-torsional buckling moment resistance

$$M_{b,a,Rd} \geq M_{Eds} \tag{4.7}$$

where

$M_{b,a,Rd}$	is the buckling moment resistance of laterally unrestrained secondary steel beam, see 4.5.4

(2) Structural resistance check in composite stage

Regardless of propped or un-propped constructions, the following criteria of 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 joints because composite action can be considered after the concrete of floor slab is hardened.

a) Classification of cross-section

The classification of cross-section of secondary composite beam should be at least Class 2. Additionally, if the longitudinal reinforcing bars in concrete slab are in tension, the following condition should be satisfied.

$$A_{sl} \geq A_{sl,req} \tag{4.8}$$

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

$$A_{sl,req} = \rho_{sl,req} A_{cs} \tag{4.9}$$

$$\rho_{sl,req} = \delta \frac{f_{ay}}{235} \frac{f_{ctm}}{f_{sk}} \sqrt{k_c} \tag{4.10}$$

$$k_c = \min \left\{ \frac{1}{1+\left(\frac{h_{cs}}{2z_0}\right)} +0.3 \; ; \; 1.0 \right\} \tag{4.11}$$

$$z_0 = \frac{(A_a+A_{sl})\left(0.5D_a+D_{ps}+0.5h_{cs}\right)}{A_a+A_{sl}+\left(\frac{h_{cs}b_{effh}}{n_0}\right)} \tag{4.12}$$

$$n_0 = \frac{E_a}{E_{cm}} \tag{4.13}$$

where

A_{sl}	is the cross-sectional area of longitudinal reinforcing bars within b_{effh}
b_{effh}	is the effective width of secondary composite beam on hogging moment region, see 4.5.1
$A_{sl,req}$	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 secondary 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 secondary steel beam
D_a	is the depth of secondary steel beam
D_{ps}	is the overall depth of profiled steel sheeting
n_0	is the modular ratio for short-term loading
E_a	is the modulus of elasticity of secondary steel beam
E_{cm}	is the secant modulus of elasticity of normal weight concrete

b) Degree of shear connection

$$\eta_s \geq \eta_{s,req} \tag{4.14}$$

$$\eta_h \geq \eta_{h,req} \tag{4.15}$$

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

$$\eta_{s,req} = \max\left\{1\text{-}\left(\frac{355}{f_{ayd}}\right)(0.75\text{-}0.03L_{es});\ 0.4\right\} \qquad \text{for } L_{es} \leq 25\text{m} \qquad (4.16)$$

$$\eta_{s,req} = 1 \qquad \text{for } L_{es} > 25\text{m} \qquad (4.17)$$

$$\eta_{h,req} = 1 \qquad (4.18)$$

$$f_{ayd} = \frac{f_{ay}}{\gamma_a} \qquad (4.19)$$

where

η_s	is the degree of shear connection on sagging moment region, see 4.5.2
η_h	is the degree of shear connection on hogging moment region, see 4.5.2
$\eta_{s,req}$	is the required minimum degree of shear connection on sagging moment region
$\eta_{h,req}$	is the required minimum degree of shear connection on hogging moment region
L_{es}	is the distance between inflection points on sagging moment region
f_{ayd}	is the design yield strength of secondary steel beam
γ_a	is the partial factor of resistance of members and cross-sections of secondary steel beam

c) Shear resistance

$$\min\left(V_{pl,Rd}\ ;\ V_{b,Rd}\right) \geq V_{Ed} \qquad (4.20)$$

where

$V_{pl,Rd}$	is the plastic shear resistance of secondary composite beam, see 4.5.3
$V_{b,Rd}$	is the shear buckling resistance of secondary composite beam, see 4.5.3
V_{Ed}	is the design shear force

d) Moment resistance

$$\min\left(M_{plf,Rds}\ ;\ M_{plp,Rds}\right) \geq M_{Eds} \qquad (4.21)$$

$$\min\left(M_{plf,Rdh}\ ;\ M_{y,v,Rdh}\right) \geq M_{Edh} \qquad (4.22)$$

where

$M_{plf,Rds}$	is the plastic sagging moment resistance of secondary composite beam with full shear connection, see 4.5.4
$M_{plp,Rds}$	is the plastic sagging moment resistance of secondary composite beam with partial shear connection, see 4.5.4
M_{Eds}	is the design sagging moment
$M_{plf,Rdh}$	is the plastic hogging moment resistance of secondary composite beam with full shear connection, see 4.5.4
$M_{y,v,Rdh}$	is the reduced moment resistance of secondary composite beam making allowance for presence of shear force, see 4.5.4
M_{Edh}	is the design hogging moment

e) Lateral-torsional buckling moment resistance

$$M_{b,Rd} \geq M_{Edh} \tag{4.23}$$

where

$M_{b,Rd}$	is the buckling moment resistance of laterally unrestrained secondary composite beam, see 4.5.4

f) Longitudinal shear resistance

$$R_{st}+R_{pse} \geq R_{tr,req} \tag{4.24}$$

$$A_{st} \geq A_{st,req} \tag{4.25}$$

$$v_{Rd} \geq v_{L,Ed} \tag{4.26}$$

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

$$R_{tr,req} = h_{cs}\frac{v_{L,Ed}}{\cot\theta} \tag{4.27}$$

$$A_{st,req} = h_{cs}\frac{0.08\sqrt{f_{ck}}}{f_{sk}} \tag{4.28}$$

where

$R_{st}+R_{pse}$	is the tension resistance of transverse reinforcement per unit length, see 4.5.5
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 4.5.5
$v_{L,Ed}$	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 secondary beam, $26.5° \leq \theta \leq 45°$ for concrete flange in compression and $38.6° \leq \theta \leq 45°$ for concrete flange in tension
f_{ck}	is the characteristic cylinder strength of normal weight concrete

(3) Serviceability check in construction stage

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

$$\delta_V \leq \delta_{V,lim} \tag{4.29}$$

$$\delta_{P+V} \leq \delta_{P+V,lim} \tag{4.30}$$

where

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

(4) Serviceability check in construction stage

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

a) Deflection

$$\delta_V \leq \delta_{V,lim} \tag{4.31}$$

$$\delta_{tP+V} \leq \delta_{P+V,lim} \tag{4.32}$$

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

$$f_{P+0.1V} \geq f_{req} \tag{4.33}$$

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

$$A_{sl} \geq A_{sl,req} \tag{4.34}$$

$$\sigma_{sl} \leq \sigma_{sl,lim} \quad \text{or} \quad p_{sl} \leq p_{sl,lim} \tag{4.35}$$

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

$$A_{sl,req} = \frac{0.72 k_c f_{ctm} A_{cs}}{\sigma_{s,lim}} \tag{4.36}$$

$$k_c = \min\left\{ \frac{1}{1+\left(\frac{h_{cs}}{2z_0}\right)} + 0.3 \ ; \ 1.0 \right\} \tag{4.37}$$

$$z_0 = \frac{(A_a + A_{sl})\left(0.5D_a + D_{ps} + 0.5h_{cs}\right)}{A_a + A_{sl} + \left(\frac{h_{cs} b_{effh}}{n_0}\right)} \tag{4.38}$$

$$n_0 = \frac{E_a}{E_{cm}} \tag{4.39}$$

where

A_{sl}	is the cross-sectional area of longitudinal reinforcing bars within b_{effh}
b_{effh}	is the effective width of secondary composite beam on hogging moment region, see 4.5.1
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 4.1

p_{sl} is the pitch of longitudinal reinforcing bars

$p_{sl,lim}$ is the limit of spacing of longitudinal reinforcing bars, given in Table 4.2

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

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

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

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

Table 4.1: 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 4.2: 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	-

where

w_k is the design crack width

Commentary:

(1) Structural resistance check in construction stage

Unless propped 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 is 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 secondary steel beams to prevent the local buckling of web and flange as mentioned in Section 3.2.

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

$$\min\left(V_{pl,a,Rd}\,;\,V_{b,a,Rd}\right) \geq V_{Ed} \tag{4.5}$$

With respect to the moment resistance, the following criteria should be satisfied to make the plastic moment resistance of secondary steel beam larger than the design sagging moment at ultimate limit state. According to EN 1993-1-1[12], 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, secondary 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 needs not to be considered.

$$M_{pl,a,Rd} \geq M_{Eds} \tag{4.6}$$

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

$$M_{b,a,Rd} \geq M_{Eds} \tag{4.7}$$

(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 is hardened. Therefore, structural resistance as the composite beams with semi-rigid joints 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 secondary 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, 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.

$$A_{sl} \geq A_{sl,req} \tag{4.8}$$

With respect to the degree of shear connection, the following criteria should be satisfied to make the degree of shear connection on 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 a large number of 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 on sagging moment region. Therefore, the use of the composite beams with partial shear connection on sagging moment region is permitted in EN 1994-1-1. On the other hand, composite beams should be designed with full shear connection on hogging moment region because the structural behaviour of the composite beams with partial shear connection has not been clearly elucidated.

$$\eta_s \geq \eta_{s,req} \tag{4.14}$$

$$\eta_h \geq \eta_{h,req} \tag{4.15}$$

With respect to the shear resistance, the following criteria should be satisfied to make the plastic shear resistance and shear buckling resistance of secondary 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 secondary composite beams can be considered to be equal to those of secondary steel beams.

$$\min\left(V_{pl,Rd}\ ;\ V_{b,Rd}\right) \geq V_{Ed} \tag{4.20}$$

With respect to the moment resistance, the following criteria should be satisfied to make the plastic moment resistance of secondary composite beam larger than the design moment at ultimate limit state. When secondary composite beams are designed with partial shear connection on 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 on hogging moment region, thus the reduction of plastic moment resistance should be considered also on hogging moment region.

$$\min\left(M_{plf,Rds}\ ;\ M_{plp,Rds}\right) \geq M_{Eds} \tag{4.21}$$

$$\min\left(M_{plf,Rdh}\ ;\ M_{y,v,Rdh}\right) \geq M_{Edh} \tag{4.22}$$

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

$$M_{b,Rd} \geq M_{Edh} \tag{4.23}$$

With respect to the longitudinal shear resistance, the following criteria 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.

$$R_{st}+R_{pse} \geq R_{tr,req} \tag{4.24}$$

$$A_{st} \geq A_{st,req} \tag{4.25}$$

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

$$v_{Rd} \geq v_{L,Ed} \tag{4.26}$$

(3) Serviceability check in construction stage

As with (1), unless propped 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 is hardened. Therefore, serviceability deflection of the steel beam with simply supported ends should be checked.

With respect to the deflection, the following criteria should be satisfied to make the deflection due to "live loads only" and "dead loads and live loads" smaller than the respective limits. Based on EN 1990[13], 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.

$$\delta_V \leq \delta_{V,lim} \tag{4.29}$$

$$\delta_{P+V} \leq \delta_{P+V,lim} \tag{4.30}$$

(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 joints should be checked.

With respect to the deflection, the following criteria 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. However, unless propped constructions, dead loads in construction stage are supported by steel beams, so that the deflection due to "dead loads, superimposed dead loads, and live loads" 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 "dead loads, superimposed dead loads, and live loads", respectively.

$$\delta_V \leq \delta_{V,lim} \tag{4.31}$$

$$\delta_{tP+V} \leq \delta_{P+V,lim} \tag{4.32}$$

With respect to the vibration, the following criteria should be satisfied to make the natural frequency of secondary 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.

$$f_{P+0.1V} \geq f_{req} \tag{4.33}$$

With respect to the control of crack width, the following criteria 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.

$$A_{sl} \geq A_{sl,req} \tag{4.34}$$

Beside, the following criteria should be also 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.

$$\sigma_{sl} \leq \sigma_{sl,lim} \quad \text{or} \quad p_{sl} \leq p_{sl,lim} \tag{4.35}$$

4.3 Structural Analysis

4.3.1 Structural Modelling of Beam-to-Beam Composite Joint

(1) Concept

In structural analysis, beam-to-beam composite joints should be modelled as rotational springs, and then the design moment and deflection of secondary composite beams with composite joints should be analysed considering the moment-rotation characteristics of the rotational springs.

(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 without significant loss of accuracy.

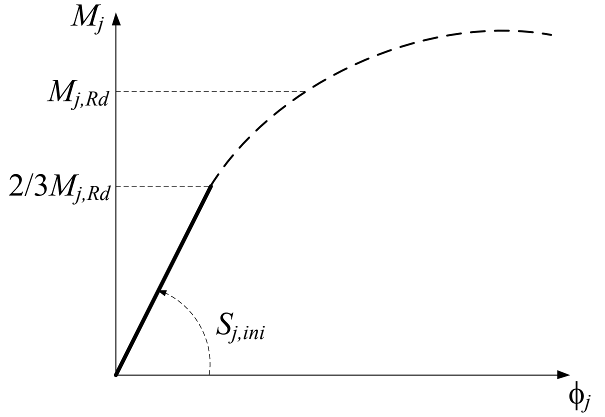
(3) Simplified moment-rotation characteristics

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

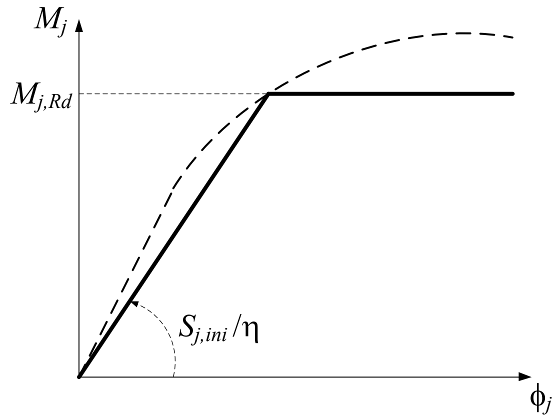
a) When the joint moment M_j is less than or equal to the elastic moment resistance $2/3M_{j,Rd}$, the linear M_j-ϕ_j curve with the rotational stiffness S_j taken as the initial rotational stiffness $S_{j,ini}$ can be applied as shown in Figure 4.3 (a). Here, $M_{j,Rd}$ is the yield moment resistance.

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 4.3 (b). The stiffness modification coefficient η for beam-to-beam composite joints with contact plates should be taken as 1.5.

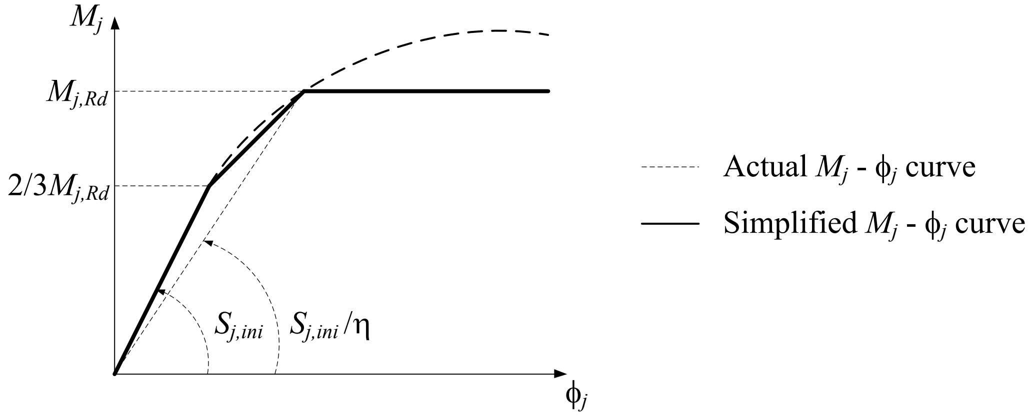
c) Besides the aboves, the tri-linear M_j-ϕ_j curve combining a) and b) can be also applied as shown in Figure 4.3 (c).



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



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



(c) Tri-linear M_j-ϕ_j curve

Figure 4.3: Simplified moment-rotation characteristics (M_j-ϕ_j curves)

Commentary:

(1) Concept

When the structural behaviour of beam members is affected by the joint structural properties, structural analysis is generally carried out modelling the joints as rotational springs. In this design guide, secondary composite beams are designed with semi-rigid joints, 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, beam-to-beam composite joints should be modelled as rotational springs and then the design moment and deflection of secondary composite beams with composite joints should be analysed in structural analysis.

(2) Analysis method

In this design guide, elastic-plastic analysis is recommended as the analysis method because structural resistance of secondary composite beams at ultimate limit state can be checked utilizing the rotational capacity of the beam-to-beam 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 without significant loss of accuracy.

(3) Simplified moment-rotation characteristics

In EN 1993-1-8, the simplified M_j-ϕ_j curves shown in Figure 4.3 (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 4.3 (a), where $M_{j,Rd}$ is the yield 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 4.3 (b). The latter S_j is the intermediate value between $S_{j,ini}$ and the secant stiffness of the yield 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 beam-to-beam composite joints with contact plates is proposed as 1.5 in EN 1994-1-1. Besides the aboves, the simplified M_j-ϕ_j curve shown in Figure 4.3 (c) combining a) and b) can be also applied to get closer to the actual M_j-ϕ_j curve.

4.3.2 Design Moment and Deflection of Secondary Composite Beam

(1) Concept

The effects of cracking of reinforced concrete slab should be taken into account in the structural analysis of secondary composite beams with composite joints.

(2) Calculation of design moment and deflection

Because the flexural rigidity of composite beams on the hogging moment region may be smaller than that on the sagging moment region due to the effects of cracking of reinforced concrete slab, these beams should be designed as non-uniform sections with the different flexural rigidities within the same beam span. When the uniformly distributed load is considered, the design moment and deflection of secondary composite beams with composite joints can be calculated by the following Eqs.. In this method, however, if the joint moment M_j is more than the yield moment resistance of the beam-to-beam composite joints $M_{j,Rd}$, M_j should be reduced to $M_{j,Rd}$ and the corresponding redistributed moment should be taken as the design moment.

a) Secondary composite beam with composite joints at both ends

$$M(x) = -\frac{w}{2}x^2+\frac{wL_b}{2}x-M_j \tag{4.40}$$

$$\delta_{max} = \frac{1}{(EI)_h}\left(A(x_0)\frac{L_b}{2}-C(x_0)\right)+\frac{1}{(EI)_s}\left(B\left(\frac{L_b}{2}\right)-A(x_0)\frac{L_b}{2}+C(x_0)\right)+\frac{M_j}{S_j}\cdot\frac{L_b}{2} \tag{4.41}$$

M_j is given by the following Eq. using convergence calculation.

$$\frac{1}{(EI)_h}A(x_0)+\frac{1}{(EI)_s}\left(A\left(\frac{L_b}{2}\right)-A(x_0)\right)+\frac{M_j}{S_j} = 0 \tag{4.42}$$

$$(EI)_h = E_a\left[I_{ay}+\frac{A_aA_{sl}\{D_a+2(D_{ps}+h_{cs}-z_{tcs\text{-}csl})\}^2}{4(A_a+A_{sl})}\right] \tag{4.43}$$

$$(EI)_s = E_a\left\{\frac{A_a(D_{cs}+D_{ps}+D_a)^2}{4\left(1+\frac{n_0A_a}{b_{effs}h_{cs}}\right)}+\frac{b_{effs}h_{cs}^{\ 3}}{12\left(\frac{2E_a}{E_{cm}}\right)}+I_{ay}\right\} \tag{4.44}$$

$$A(x) = \frac{w}{6}x^3-\frac{wL_b}{4}x^2+M_jx \tag{4.45}$$

$$B(x) = \frac{w}{24}x^4-\frac{wL_b}{12}x^3+\frac{M_j}{2}x^2 \tag{4.46}$$

$$C(x) = \frac{w}{8}x^4-\frac{wL_b}{6}x^3+\frac{M_j}{2}x^2 \tag{4.47}$$

$$x_0 = \frac{L_b}{2}\left(1-\sqrt{1-\frac{8M_j}{wL_b^{\ 2}}}\right) \tag{4.48}$$

where

$M(x)$	is the moment of secondary composite beam along x-axis
w	is the uniformly distributed load

L_b	is the beam length
δ_{max}	is the maximum deflection of secondary composite beam
S_j	is the rotational stiffness of beam-to-beam composite joint
M_j	is the joint moment
$(EI)_h$	is the hogging flexural rigidity of secondary composite beam
$(EI)_s$	is the sagging flexural rigidity of secondary composite beam
E_a	is the modulus of elasticity of secondary steel beam
I_{ay}	is the second moment of area of secondary steel beam about major axis (y-y axis)
A_a	is the cross-sectional area of secondary steel beam
A_{sl}	is the cross-sectional area of longitudinal reinforcing bars within b_{effh}
b_{effh}	is the effective width of secondary composite beam on hogging moment region, see 4.5.1
D_a	is the depth of secondary 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 secondary composite beam on sagging moment region, see 4.5.1
E_{cm}	is the secant modulus of elasticity of normal weight concrete
x_0	is the *x*-coordinate at inflection point

b) Secondary composite beam with composite joints at one end

$$M(x) = -\frac{w}{2}x^2+\left(\frac{wL_b}{2}-\frac{M_j}{L_b}\right)x \tag{4.49}$$

$$\delta_{max} = \frac{1}{(EI)_s}E(x_\delta)+\theta_{pin}x_\delta \tag{4.50}$$

M_j and x_δ are given by the following Eqs. using convergence calculation.

$$\frac{1}{(EI)_h}\left(D(L_b)-D(x_0)\right)+\frac{1}{(EI)_s}D(x_0)+\theta_{pin}+\frac{M_j}{S_j} = 0 \tag{4.51}$$

$$\frac{1}{(EI)_s}D(x_\delta)+\theta_{pin} = 0 \tag{4.52}$$

$$D(x) = \frac{w}{6}x^3-\frac{1}{2}\left(\frac{wL_b}{2}-\frac{M_j}{L_b}\right)x^2 \tag{4.53}$$

$$E(x) = \frac{w}{24}x^4-\frac{1}{6}\left(\frac{wL_b}{2}-\frac{M_j}{L_b}\right)x^3 \tag{4.54}$$

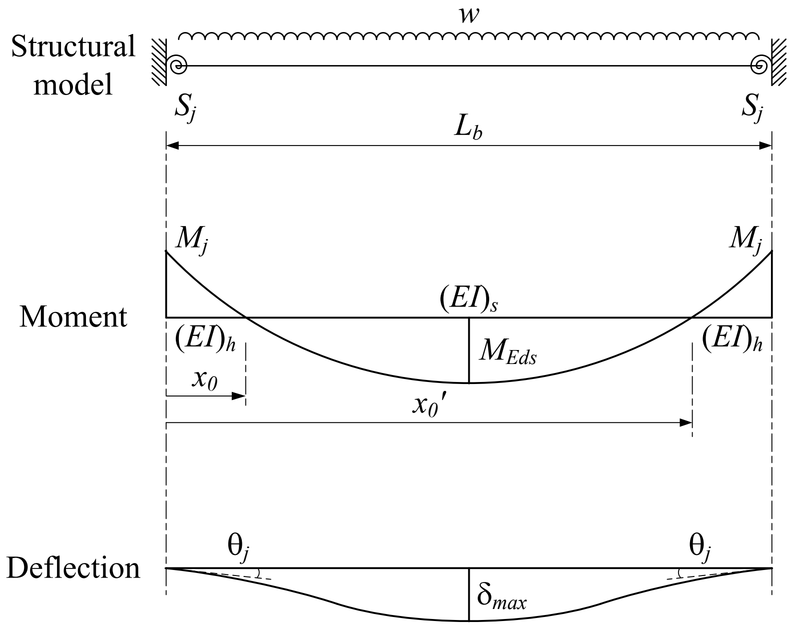
$$F(x) = \frac{w}{8}x^4-\frac{1}{3}\left(\frac{wL_b}{2}-\frac{M_j}{L_b}\right)x^3 \tag{4.55}$$

$$x_0 = L_b-\frac{2M_j}{wL_b} \tag{4.56}$$

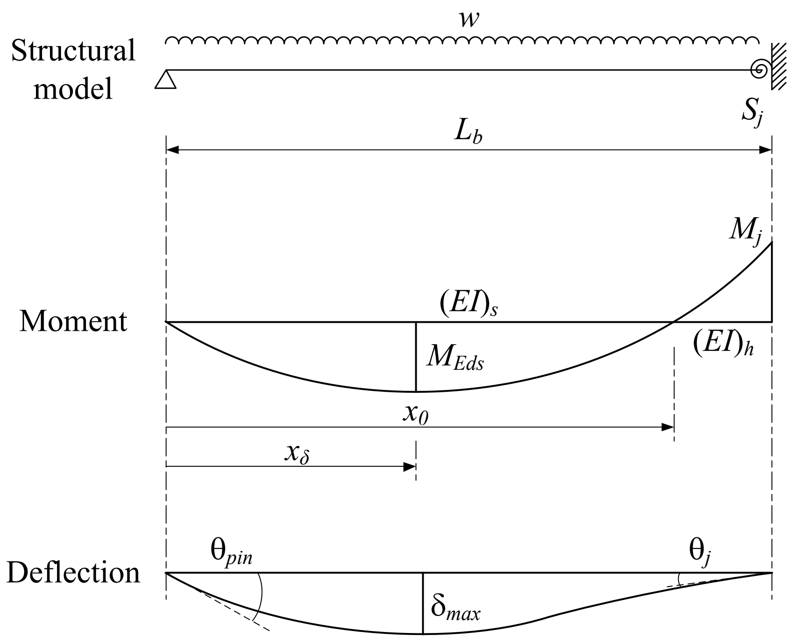
$$\theta_{pin} = -\frac{1}{(EI)_hL_b}\left(E(L_b)-D(x_0)L_b+F(x_0)\right)-\frac{1}{(EI)_sL_b}\left(D(x_0)L_b-F(x_0)\right) \tag{4.57}$$

where

x_δ	is the x-coordinate where deflection is maximized
θ_{pin}	is the rotation of pin joint



(a) Secondary composite beam with composite joints at both ends



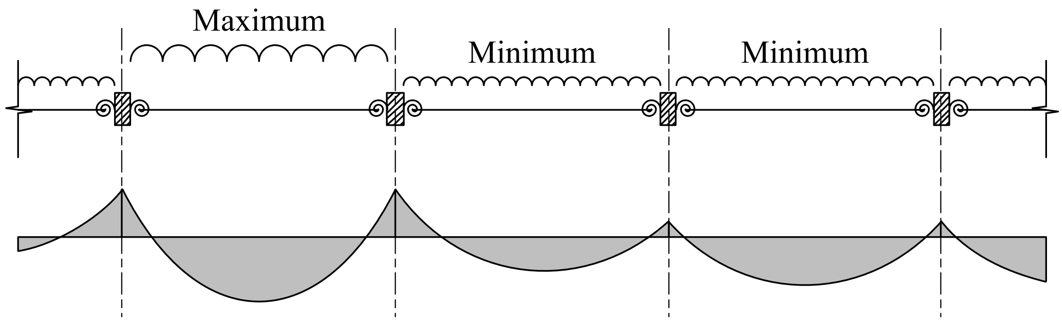
(b) Secondary composite beam with composite joint at one end

Figure 4.4: Design moment and deflection

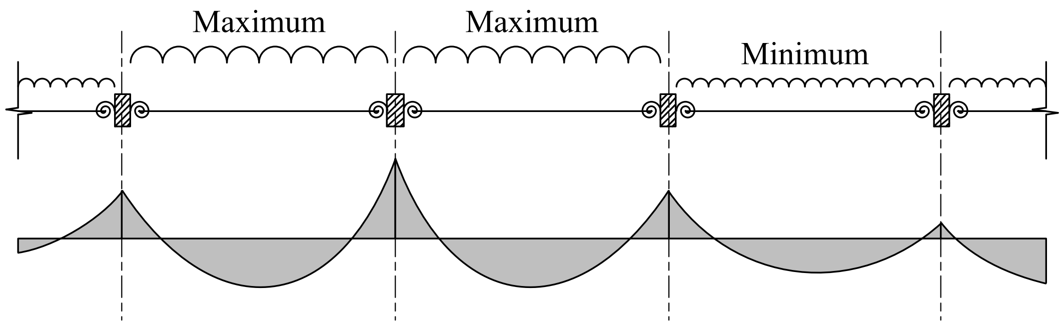
(3) Effect of loading patterns

As shown in Figure 4.5, the design moment and deflection of the secondary composite beams with composite joints may be also affected by loading patterns because the moment between the adjacent secondary beams may be continuous through the primary beam.

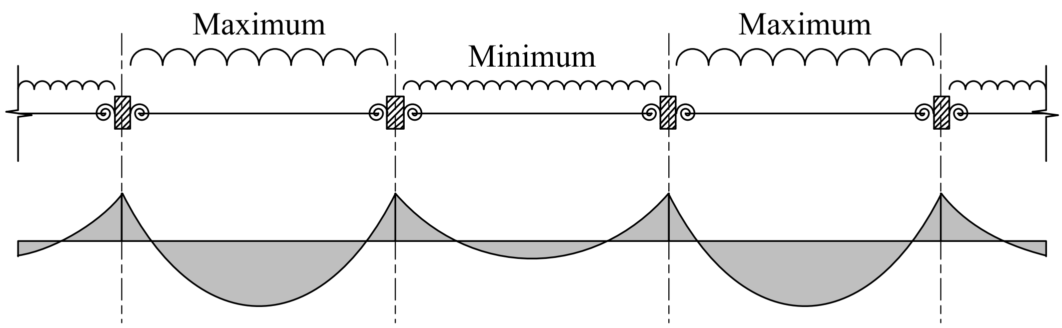
Therefore, in a precise manner, all the secondary composite beams should be individually designed based on their own critical loading pattern selected from all the possible loading patterns. Unless more accurate methods are provided, the simplified analysis method, in which all the secondary composite beams are classified into external or internal beams and the virtual floor plans including the adjacent beam-to-beam composite beams are extracted as shown in Figure 4.6, 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 I.



Loading pattern (a)

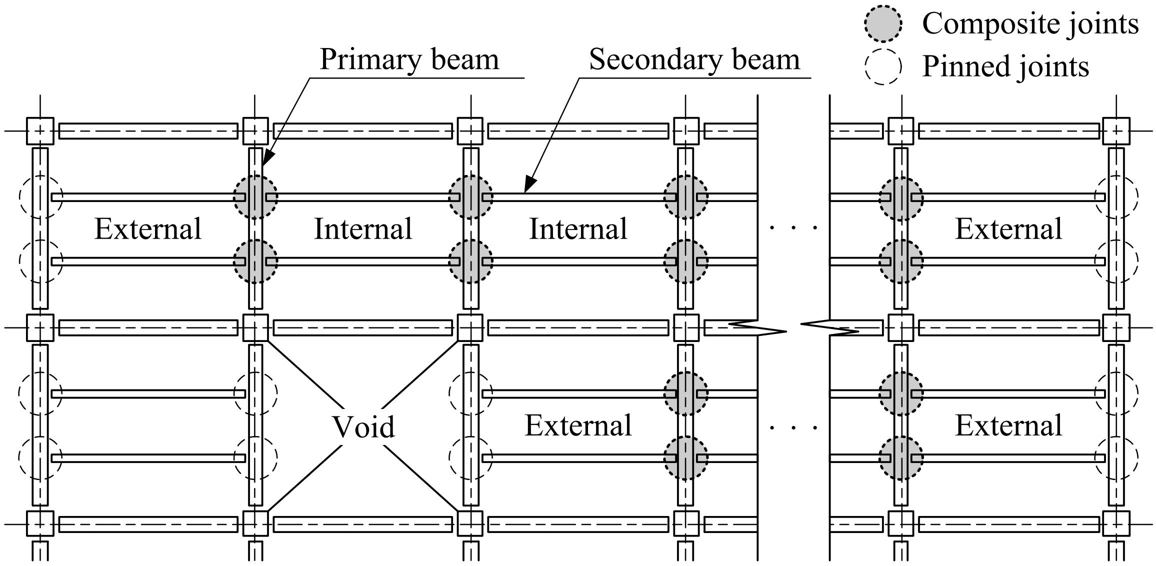


Loading pattern (b)

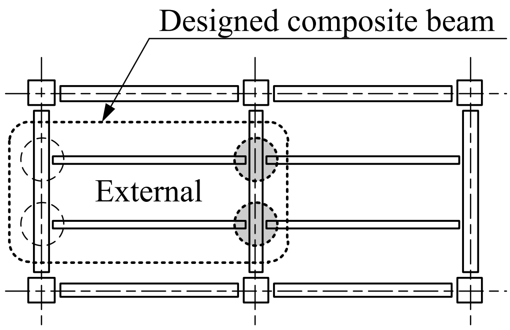


Loading pattern (c)

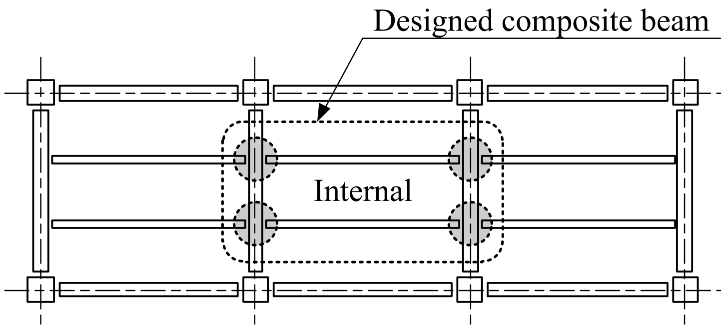
Figure 4.5: Moment distribution in response to loading patterns



(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 4.6: Virtual floor plans in simplified analysis method

4.4 Structural Properties of Beam-to-Beam Composite Joint

4.4.1 Effective Width and Effective Length

(1) Effective width of beam-to-beam composite joint

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

(2) Effective length of beam-to-beam composite joint

The effective length of beam-to-beam composite joints $h_{eff,j}$ which is an essential design parameter to evaluate the rotational stiffness of the joints can be the distance between the first headed studs on the opposite secondary composite beams shown in Figure 4.7.

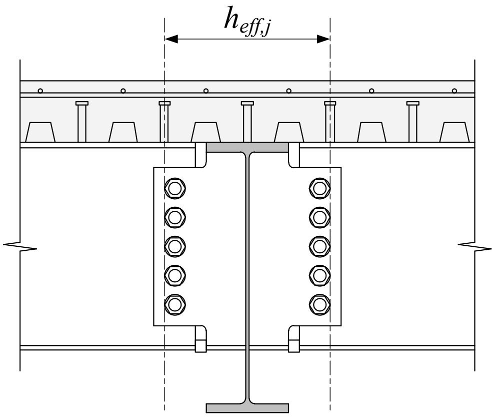


Figure 4.7: Effective length of composite joint

Commentary:

(1) Effective width of beam-to-beam composite joint

The effective width of beam-to-beam composite joints $b_{eff,j}$ is an essential design parameter to evaluate the rotational stiffness and the moment resistance of the joints. EN 1994-1-1 specifies that it should be equivalent to that of the adjacent beam. However, $b_{eff,j}$ actually depends on the joint detail and the arrangement of the various joint components, and it can be assumed that the structural properties may not be accurately predicted based on the current definition. Inaccurate predictions may lead to unsafe design in which the design moment and deflection of the adjacent secondary composite beams are underestimated. Therefore, in this design guide, $b_{eff,j}$ can be modified based on the experimental evidence or advanced calculation method supported by structural testing.

(2) Effective length of beam-to-beam composite joint

In addition to $b_{eff,j}$, also the effective length of beam-to-beam composite joints $h_{eff,j}$ is an essential design parameter to evaluate the rotational stiffness of the joints. In EN 1994-1-1, the effective length of beam-to-column composite joints is specified, but that of beam-to-beam composite joints is not defined. In this design guide it can be the distance between the first headed studs on the opposite secondary composite beams shown in Figure 4.7.

4.4.2 Initial Rotational Stiffness

(1) Concept

The initial rotational stiffness of beam-to-beam 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

The initial rotational stiffness of beam-to-beam composite joints $S_{j,ini}$ can be determined by the following Eqs. assuming the assembly of the two elastic springs for each component, see Figure 4.8. Here, the stiffness of the elastic spring for the contact parts 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.

$$S_{j,ini} = E_s k_{slip} \sum k_{sl,r} z_{csl,r\text{-}cc}{}^2 \tag{4.58}$$

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

$$k_{slip} = \frac{1}{1+\left(\frac{E_s k_{sl,eq}}{K_{sc}}\right)} \tag{4.59}$$

$$k_{sl,r} = \frac{A_{sl,r}}{\left(\frac{h_{eff,j}}{2}\right)} \tag{4.60}$$

$$k_{sl,eq} = \frac{\sum A_{sl,r}}{\left(\frac{h_{eff,j}}{2}\right)} \tag{4.61}$$

$$K_{sc} = \frac{N k_{sc}}{\nu\text{-}\left(\frac{\nu\text{-}1}{1+\xi}\right)\left(\frac{z_{sl,eq\text{-}cc}}{z_{sl,eq\text{-}ca}}\right)} \tag{4.62}$$

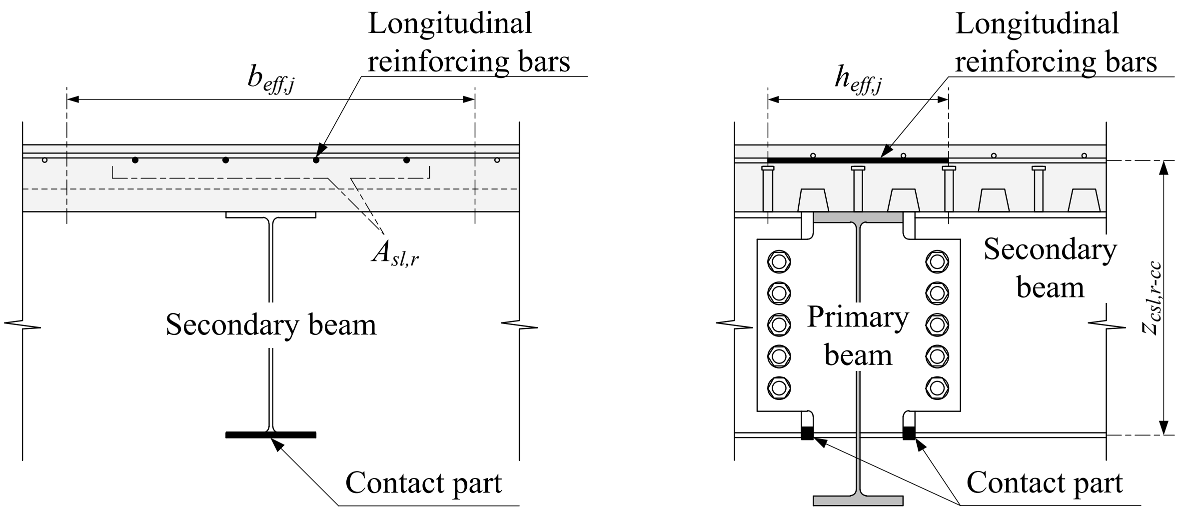
$$\nu = \sqrt{\frac{(1+\xi) N k_{sc} l z_{sl,eq\text{-}ca}{}^2}{E_a I_{ay}}} \tag{4.63}$$

$$\xi = \frac{E_a I_{ay}}{z_{sl,eq\text{-}ca} E_s A_{sl,r}} \tag{4.64}$$

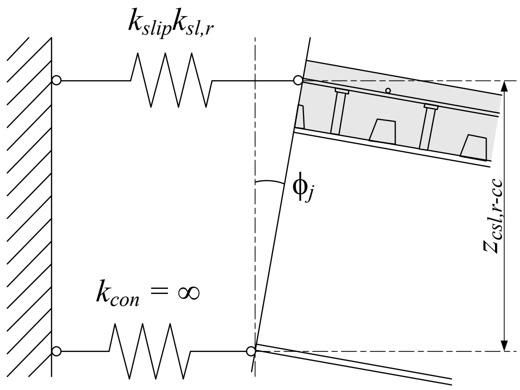
where

E_s	is the modulus of elasticity of reinforcing bars
$z_{csl,r\text{-}cc}$	is the vertical distance between centre of longitudinal reinforcing bars and centre of contact part for a row r
k_{slip}	is the stiffness reduction factor due to deformation of headed studs
$k_{sl,r}$	is the stiffness coefficient of longitudinal reinforcing bars for a row r
$A_{sl,r}$	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 beam-to-beam composite joint, see 4.4.1
$h_{eff,j}$	is the effective length of beam-to-beam composite joint

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



(a) Beam-to-beam composite joint with contact plates



(b) Assembly of two elastic springs for each component

Figure 4.8: Modelling of beam-to-beam composite joint to evaluate initial rotational stiffness

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 beam-to-beam composite joint with contact plates can be modelled as an assembly of two components represented by the elastic springs as shown in Figure 4.8, 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.

4.4.3 Yield Moment Resistance

(1) Concept

The yield moment resistance of beam-to-beam 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 yield moment resistance

As shown in Figure 4.9, the yield moment resistance of beam-to-beam composite joints $M_{j,Rd}$ can be determined by the following Eqs. using the tension resistance of the longitudinal reinforcing bars $R_{sl,j}$ or the compression resistance of the contact parts R_{con}, whichever is smaller. However, it is limited to the condition that the stiffeners 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 secondary steel beams; otherwise the stiffeners should be considered in calculating R_{con}.

$$M_{j,Rd} = z_{sl,eq\text{-}cc}\min\left(R_{sl,j}\ ;\ R_{con}\right) \tag{4.65}$$

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

$$R_{sl,j} = \sum A_{sl,r} f_{sd} \tag{4.66}$$

$$R_{con} = \min\left\{A_{bf} f_{ayd}\ ;\ A_{cp} f_{cpyd}\ ;\ 1.5 A_{bea} \min\left(\frac{f_{ay}}{\gamma_{a,2}}\ ;\ \frac{f_{cpy}}{\gamma_{cp,2}}\right)\right\} \tag{4.67}$$

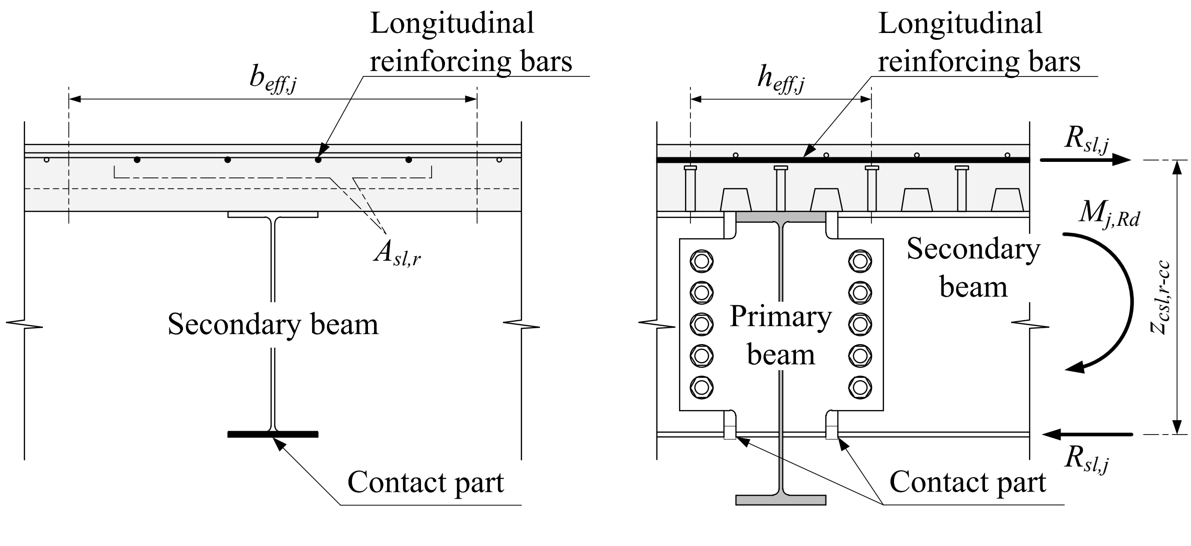
$$f_{sd} = \frac{f_{sk}}{\gamma_s} \tag{4.68}$$

$$f_{cpyd} = \frac{f_{cpy}}{\gamma_{cp}} \tag{4.69}$$

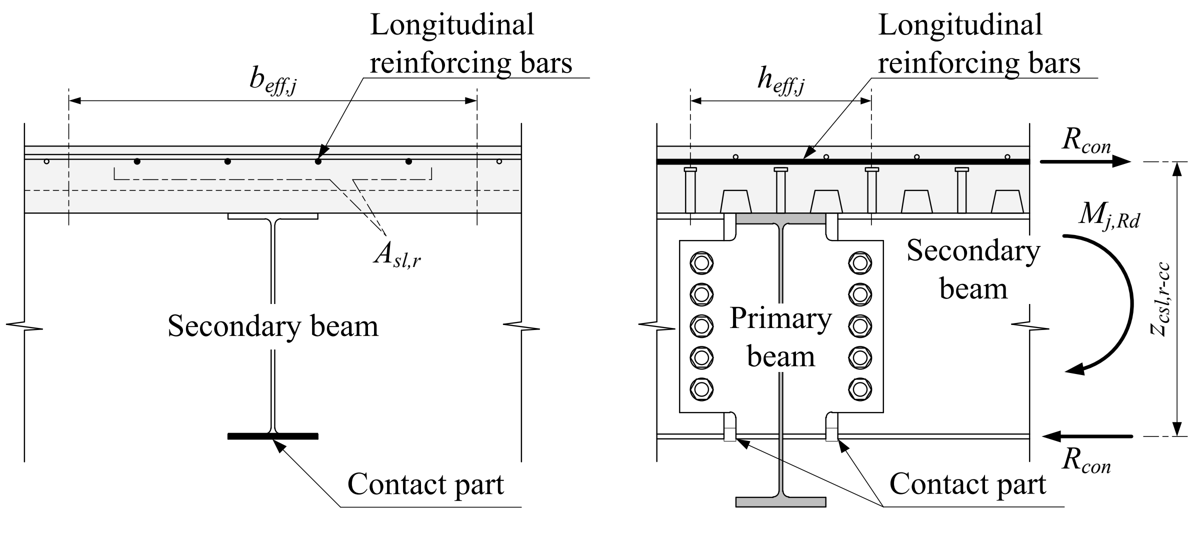
where

$z_{sl,eq\text{-}cc}$	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 beam-to-beam composite joint, see 4.4.1
$A_{sl,r}$	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 secondary steel beam
f_{ayd}	is the design yield strength of secondary 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 secondary steel beam and contact plate
f_{ay}	is the nominal value of yield strength of secondary steel beam
$\gamma_{a,2}$	is the partial factor of resistance of secondary steel beam in bearing
f_{cpy}	is the nominal value of yield strength of contact plate
$\gamma_{cp,2}$	is the partial factor of resistance of contact plate in bearing

f_{sd}	is the design yield strength of reinforcing bars
f_{sk}	is the characteristic yield strength of reinforcing bars
γ_s	is the partial factor of reinforcing bars
f_{cpyd}	is the design yield strength of contact plate
γ_{cp}	is the partial factor of resistance of members and cross-sections of contact plate



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



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

Figure 4.9: Modelling of beam-to-beam composite joint to evaluate yield moment resistance

4.5 Structural Properties of Secondary Composite Beam

4.5.1 Effective Width

(1) Effective width of secondary composite beams on sagging moment region

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

$$b_{effs} = b_{0s} + \sum b_{eis} \tag{4.70}$$

b_{eis} is given by:

$$b_{eis} = \min\left(\frac{L_{es}}{8}\ ;\ b_{is}\right) \tag{4.71}$$

where

b_{0s}	is the distance between centres of outstand headed studs on sagging moment region
b_{eis}	is the value of effective width of secondary composite beam on each side of web of steel beam on sagging moment region
L_{es}	is the distance between inflection points on sagging moment region, see Figure 4.10
b_{is}	is the distance from outstand headed stud to a point mid-way between adjacent webs of steel beams on sagging moment region

(2) Effective width of secondary composite beams on hogging moment region

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

$$b_{effh} = b_{0h} + \sum b_{eih} \tag{4.72}$$

b_{eih} is given by:

$$b_{eih} = \min\left(\frac{L_{eh}}{8}\ ;\ b_{ih}\right) \tag{4.73}$$

where

b_{0h}	is the distance between centres of outstand headed studs on hogging moment region
b_{eih}	is the value of effective width of secondary composite beam on each side of web of steel beam on hogging moment region
L_{eh}	is the distance between inflection points on hogging moment region, see Figure 4.10
b_{ih}	is the distance from outstand headed stud to a point mid-way between adjacent webs of steel beams on 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 secondary composite beams should be evaluated for both the limit states individually.

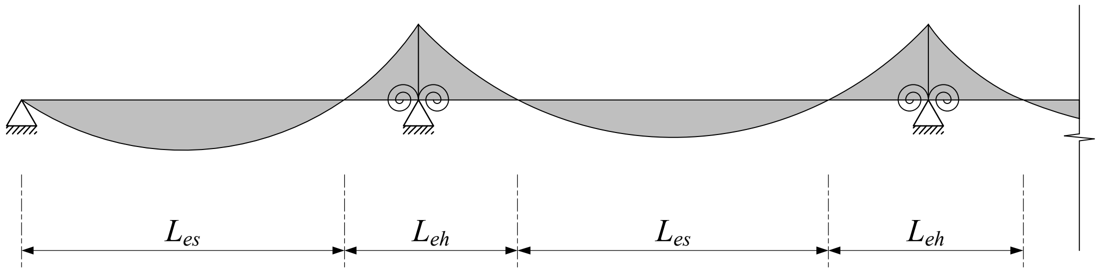


Figure 4.10: Distance between inflection points

Commentary:

(3) Distance between inflection points

According to the design criteria in subsection 4.2.1 (2), beam-to-beam 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 in subsection 4.3.1 (3). 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 secondary composite beams should be evaluated for both the limit states individually.

4.5.2 Degree of Shear Connection

(1) Degree of shear connection on sagging moment region

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

$$\eta_s = \frac{R_{qs}}{\min(R_a\ ;\ R_{cs})} \tag{4.74}$$

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

$$R_{qs} = N_{hss} k_{ts} P_{Rd} \tag{4.75}$$

$$R_a = A_a f_{ayd} \tag{4.76}$$

$$R_{cs} = b_{effs} h_{cs} \left(0.85 f_{cd}\right) \tag{4.77}$$

$$k_{ts} = \min\left\{\frac{0.7}{\sqrt{\min(n_{hss}\ ;\ 2)}} \frac{b_{0,min}}{D_{ps}}\left(\frac{h_{hs}}{D_{ps}} - 1\right)\ ;\ k_{ts,max}\right\} \tag{4.78}$$

$$f_{cd} = \frac{f_{ck}}{\gamma_c} \tag{4.79}$$

where

R_{qs}	is the longitudinal shear force transfer within half of L_{es}
R_a	is the tension (compression) resistance of secondary steel beam
R_{cs}	is the compression resistance of composite slab within b_{effs}
L_{es}	is the distance between inflection points on sagging moment region
b_{effs}	is the effective width of secondary composite beams on sagging moment region, see 4.5.1
N_{hss}	is the number of headed studs arranged within half of L_{es}
P_{Rd}	is the shear resistance of a headed stud, see 2.4
A_a	is the cross-sectional area of secondary steel beam
f_{ayd}	is the design yield strength of secondary 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 on sagging moment region
n_{hss}	is the number of headed studs per sheeting rib on 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 on sagging moment region, given in Table 4.3
f_{cd}	is the design strength of normal weight concrete
f_{ck}	is the characteristic cylinder strength of normal weight concrete
γ_c	is the partial factor of concrete

Table 4.3: Maximum reduction factor for shear resistance of a headed stud $k_{t,max}$

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

(2) Degree of shear connection on hogging moment region

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

$$\eta_h = \frac{R_{qh}}{\min(R_a\ ;\ R_{sl})} \tag{4.80}$$

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

$$R_{qh} = N_{hsh} k_{th} P_{Rd} \tag{4.81}$$

$$R_{sl} = A_{sl} f_{sd} \tag{4.82}$$

$$k_{th} = \min\left\{\frac{0.7}{\sqrt{\min(n_{hsh}\ ;\ 2)}}\frac{b_{0,min}}{D_{ps}}\left(\frac{h_{hs}}{D_{ps}}-1\right)\ ;\ k_{th,max}\right\} \tag{4.83}$$

where

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 on hogging moment region
b_{effh}	is the effective width of secondary composite beams on hogging moment region, see 4.5.1
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 4.4.3
k_{th}	is the reduction factor for shear resistance of a headed stud on hogging moment region
n_{hsh}	is the number of headed studs per sheeting rib on hogging moment region
$k_{th,max}$	is the maximum reduction factor for shear resistance of a headed stud on hogging moment region, given in Table 4.3

4.5.3 Shear Resistance

(1) Shear resistance in construction stage

a) Plastic shear resistance of secondary steel beam

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

$$V_{pl,a,Rd} = A_V\left(\frac{f_{ayd}}{\sqrt{3}}\right) \tag{4.84}$$

A_V is given by:

$$A_V = \max\left\{A_a\text{-}2B_a t_f\text{+}(t_w\text{+}2r)t_f\,;\;1.2\left(D_a\text{-}2t_f\right)t_w\right\} \tag{4.85}$$

where

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

b) Shear buckling resistance of secondary steel beam

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

$$V_{b,a,Rd} = V_{pl,a,Rd} \qquad \text{for } \frac{\left(D_a\text{-}2t_f\right)}{t_w} \le \frac{72}{1.2}\sqrt{\frac{235}{f_{ay}}} \tag{4.86}$$

$$V_{b,a,Rd} = \min\left\{\frac{\chi_w f_{wy}\left(D_a\text{-}2t_f\right)t_w}{\sqrt{3}\gamma_a}\;;\;\frac{1.2 f_{wy}\left(D_a\text{-}2t_f\right)t_w}{\sqrt{3}\gamma_a}\right\} \qquad \text{for } \frac{\left(D_a\text{-}2t_f\right)}{t_w} > \frac{72}{1.2}\sqrt{\frac{235}{f_{ay}}} \tag{4.87}$$

χ_w is given by:

$$\chi_w = 1.2 \qquad \text{for } \lambda_w < \frac{0.83}{1.2} \tag{4.88}$$

$$\chi_w = \frac{0.83}{\lambda_w} \qquad \text{for } \frac{0.83}{1.2} \le \lambda_w \tag{4.89}$$

$$\lambda_w = 0.76\sqrt{\frac{f_{wy}}{k_{\tau,min}\left[190000\left\{\frac{t_w}{\left(D_a\text{-}2t_f\right)}\right\}^2\right]}} \tag{4.90}$$

$$k_{\tau,min} = 5.34 \text{ (without rigid transverse and longitudinal stiffeners)} \tag{4.91}$$

where

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

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

λ_w is the modified slenderness of web of secondary 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 secondary composite beam, $V_{pl,Rd}$ and $V_{b,Rd}$, can be considered to be equal to those of secondary steel beam.

Commentary:

(2) Shear resistance in composite stage

The contribution of floor slab cannot be considered for the shear resistance of composite beams. In other words, it should be assumed that the web of the steel beams is subjected to total shear force.

4.5.4 Moment Resistance

(1) Moment resistance in construction stage

a) Plastic moment resistance of secondary steel beam

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

$$M_{pl,a,Rd} = W_{pl,a} f_{ayd} \quad \text{for Class 1 or Class 2 cross-sections} \tag{4.92}$$

$$M_{pl,a,Rd} = W_{eff,pl,a} f_{ayd} \quad \text{for effective Class 2 cross-sections} \tag{4.93}$$

where

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

b) Buckling moment resistance of secondary steel beam

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

$$M_{b,a,Rd} = \chi_{LT,a} M_{pl,a,Rd} \tag{4.94}$$

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

$$\chi_{LT,a} = \min\left(\frac{1}{\Phi_{LT,a} + \sqrt{\Phi_{LT,a}^{\ 2} - 0.75\lambda_{LT,a}^{\ 2}}} \ ; 1.0 \ ; \frac{1}{\lambda_{LT,a}^{\ 2}} \right) \tag{4.95}$$

$$\Phi_{LT,a} = 0.5\left\{1 + \alpha_{LT}\left(\lambda_{LT,a} - 0.4\right) + 0.75\lambda_{LT,a}^{\ 2}\right\} \tag{4.96}$$

$$\lambda_{LT,a} = \sqrt{\frac{M_{pl,a,Rd}}{M_{cr,a}}} \tag{4.97}$$

$$M_{cr,a} = C_1 \frac{\pi^2 E_a I_{az}}{L_{cr,a}^{\ 2}} \sqrt{\frac{I_{w,a}}{I_{az}} + \frac{L_{cr,a}^{\ 2} G_a I_{T,a}}{\pi^2 E_a I_{az}}} \tag{4.98}$$

where

$\chi_{LT,a}$	is the reduction factor for lateral-torsional buckling of secondary steel beam
$\Phi_{LT,a}$	is the value to determine reduction factor for lateral-torsional buckling of secondary steel beam
α_{LT}	is the imperfection factor corresponding to appropriate lateral-torsional buckling curve, recommended in Table 4.4
$\lambda_{LT,a}$	is the non-dimensional slenderness for lateral-torsional buckling of secondary steel beam
$M_{cr,a}$	is the elastic critical moment for lateral-torsional buckling of secondary 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 secondary steel beam

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

Table 4.4: 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

where
D_a is the depth of secondary steel beam
B_a is the width of secondary steel beam

(2) Moment resistance in composite stage

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

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

< Class 1 or Class 2 cross-sections >

$$M_{plf,Rds} = R_a \left\{ \frac{D_a}{2} + D_{cs} - \frac{R_a}{R_{cs}} \frac{\left(D_{cs} - D_{ps}\right)}{2} \right\} \tag{4.99}$$

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

$$M_{plf,Rds} = R_a \frac{D_a}{2} + R_{cs} \left(\frac{D_{cs} + D_{ps}}{2} \right) - \frac{(R_a - R_{cs})^2}{4 B_a f_{ayd}} \tag{4.100}$$

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

$$M_{plf,Rds} = W_{pl,a} f_{ayd} + R_{cs} \left(\frac{D_a + D_{cs} + D_{ps}}{2} \right) - \frac{{R_{cs}}^2}{4 t_w f_{ayd}} \tag{4.101}$$

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

< Effective Class 2 cross-sections >

$$M_{plf,Rds} = R_a \left\{ \frac{D_a}{2} + D_{cs} - \frac{R_a}{R_{cs}} \frac{\left(D_{cs} - D_{ps}\right)}{2} \right\} \tag{4.102}$$

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

$$M_{plf,Rds} = R_a \frac{D_a}{2} + R_{cs} \left(\frac{D_{cs} + D_{ps}}{2} \right) - \frac{(R_a - R_{cs})^2}{4 B_a f_{ayd}} \tag{4.103}$$

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

$$M_{plf,Rds} = W_{pl,a} f_{ayd} + R_{cs} \left(\frac{D_a + D_{cs} + D_{ps}}{2} \right) - \frac{R_{cs}^{\ 2} + (R_v - R_{cs})\left(R_v - R_{cs} - 2R_{eff,v}\right)}{4 t_w f_{ayd}} \tag{4.104}$$

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 Eqs..

$$R_a = A_a f_{ayd} \tag{4.105}$$

$$R_{cs} = b_{effs} h_{cs} \left(0.85 f_{cd}\right) \tag{4.106}$$

$$R_w = R_a - 2 B_a t_f f_{ayd} \tag{4.107}$$

$$R_{eff,v} = 40 t_w^{\ 2} f_{ayd} \sqrt{\frac{235}{f_{ayd}}} \tag{4.108}$$

$$R_v = \left\{ D_a - 2\left(t_f + r\right) \right\} t_w f_{ayd} \tag{4.109}$$

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 secondary steel beam
R_a	is the tension (compression) resistance of secondary 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 secondary steel beam
$R_{eff,v}$	is the tension (compression) resistance of effective clear web of secondary steel beam
R_v	is the tension (compression) resistance of clear web of secondary steel beam
b_{effs}	is the effective width of secondary composite beams on sagging moment region, see 4.5.1
A_a	is the cross-sectional area of secondary steel beam
h_{cs}	is the thickness of composite slab above profiled steel sheeting
f_{cd}	is the design strength of normal weight concrete, see 4.5.2
t_f	is the flange thickness of secondary steel beam
r	is the root radius of secondary steel beam

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

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

< Class 1 or Class 2 cross-sections >

$$M_{plp,Rds} = M_{plf,Rds} \qquad \text{for } \eta_s \geq 1 \tag{4.110}$$

$$M_{plp,Rds} = R_a \frac{D_a}{2} + R_{qs}\left(D_{cs} - \frac{R_{qs}}{R_{cs}}\frac{D_{cs}-D_{ps}}{2}\right) - \frac{\left(R_a - R_{qs}\right)^2}{4B_a f_{ayd}} \tag{4.111}$$

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

$$M_{plp,Rds} = W_{pl,a} f_{ayd} + R_{qs}\left(\frac{D_a}{2} + D_{cs} - \frac{R_{qs}}{R_{cs}}\frac{D_{cs}-D_{ps}}{2}\right) - \frac{{R_{qs}}^2}{4t_w f_{ayd}} \tag{4.112}$$

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

< Effective Class 2 cross-sections >

$$M_{plp,Rds} = M_{plf,Rds} \qquad \text{for } \eta_s \geq 1 \tag{4.113}$$

$$M_{plp,Rds} = R_a \frac{D_a}{2} + R_{qs}\left(D_{cs} - \frac{R_{qs}}{R_{cs}}\frac{D_{cs}-D_{ps}}{2}\right) - \frac{\left(R_a - R_{qs}\right)^2}{4B_a f_{ayd}} \tag{4.114}$$

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

$$M_{plp,Rds} = W_{pl,a} f_{ayd} + R_{qs}\left(\frac{D_a}{2} + D_{cs} - \frac{R_{qs}}{R_{cs}}\frac{D_{cs}-D_{ps}}{2}\right) - \frac{{R_{qs}}^2 + \left(R_v - R_{qs}\right)\left(R_v - R_{qs} - 2R_{eff,v}\right)}{4t_w f_{ayd}} \tag{4.115}$$

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

where

η_s is the degree of shear connection on sagging moment region, see 4.5.2

R_{qs} is the longitudinal shear force transfer within the half of L_{es}, see 4.5.2

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

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

< Class 1 or Class 2 cross-sections >

$$M_{plf,Rdh} = R_a\left(\frac{D_a}{2} + z_{csl\text{-}tf}\right) \tag{4.116}$$

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

$$M_{plf,Rdh} = R_a \frac{D_a}{2} + R_{sl} z_{csl\text{-}tf} - \frac{(R_a - R_{sl})^2}{4B_a f_{ayd}} \tag{4.117}$$

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

$$M_{plf,Rdh} = W_{pl,a} f_{ayd} + R_{sl}\left(\frac{D_a}{2} + z_{csl\text{-}tf}\right) - \frac{{R_{sl}}^2}{4t_w f_{ayd}} \tag{4.118}$$

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

< Effective Class 2 cross-sections >

$$M_{plf,Rdh} = R_{eff,a}\left(\frac{D_a}{2}+z_{csl\text{-}tf}\right) \tag{4.119}$$

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

$$M_{plf,Rdh} = R_{eff,a}\frac{D_a}{2}+R_{sl}z_{csl\text{-}tf}-\frac{\left(R_{eff,a}\text{-}R_{sl}\right)^2}{4B_af_{ayd}} \tag{4.120}$$

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

$$M_{plf,Rdh} = W_{pl,a}f_{ayd}+R_{sl}\left(\frac{D_a}{2}+z_{csl\text{-}tf}\right)-\frac{R_{sl}{}^2+(R_v+R_{sl})\left(R_v+R_{sl}\text{-}2R_{eff,v}\right)}{4t_wf_{ayd}} \tag{4.121}$$

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

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

$$R_{eff,a} = R_a\text{-}R_v+R_{eff,v} \tag{4.122}$$

where

$z_{csl\text{-}tf}$	is the vertical distance between centre of longitudinal reinforcing bars and top of flange of secondary steel beam
R_{sl}	is the tension resistance of longitudinal reinforcing bars within b_{effh}, see 4.5.2
$R_{eff,a}$	is the tension (compression) resistance of effective secondary steel beam

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

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

$$M_{y,v,Rdh} = M_{plf,Rdh} \qquad \text{for } V_{Ed} \le \frac{V_{pl,Rd}}{2} \tag{4.123}$$

$$M_{y,v,Rdh} = M_{plf,Rdh}\text{-}\left(M_{plf,Rdh}\text{-}M_{pl,f,Rd}\right)\left(\frac{2V_{Ed}}{V_{pl,Rd}}\text{-}1\right)^2 \tag{4.124}$$

for $V_{Ed} > \dfrac{V_{pl,Rd}}{2}$

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

$$M_{pl,f,Rd} = 2R_f\left(\frac{D_a}{2}+z_{csl\text{-}tf}\right) \qquad \text{for } PNA \text{ outside steel beam} \tag{4.125}$$

$$M_{pl,f,Rd} = R_fD_a+R_{sl}z_{csl\text{-}tf}-\frac{\left(2R_f\text{-}R_{sl}\right)^2}{4B_af_{ayd}} \qquad \text{for } PNA \text{ in steel flange} \tag{4.126}$$

$$M_{pl,f,Rd} = R_fz_{ctf\text{-}cbf}+R_{sl}\left(\frac{D_a}{2}+z_{csl\text{-}tf}\right) \qquad \text{for } PNA \text{ in steel web} \tag{4.127}$$

$$R_f = B_at_ff_{ayd} \tag{4.128}$$

where

V_{Ed}	is the design shear force

$V_{pl,Rd}$	is the plastic shear resistance of secondary composite beam, see 4.5.3
$V_{b,Rd}$	is the shear buckling resistance of secondary composite beam, see 4.5.3
$M_{pl,f,Rd}$	is the plastic moment resistance of secondary composite beam after deducting shear area
R_f	is the tension (compression) resistance of flange of secondary steel beam
$z_{ctf\text{-}cbf}$	is the vertical distance between centres of top and bottom flanges of secondary steel beam

e) Buckling moment resistance of laterally unrestrained composite beam

As far as Eq.(4.129) and (4.130) are satisfied, the buckling moment resistance of laterally unrestrained secondary composite beam $M_{b,Rd}$ can be determined by Eq.(4.131).

$$E_a I_{cs2} \geq 0.35 E_a t_w^{\ 2} B_b / D_a \tag{4.129}$$

$$p_{ps}/B_a \leq 0.4 f_{hsu} d_{hs}^{\ 2} \frac{1-\chi_{LT}\lambda_{LT}^{\ 2}}{k_s \chi_{LT}\lambda_{LT}^{\ 2}} \tag{4.130}$$

$$M_{b,Rd} = \chi_{LT} M_{plf,Rdh} \tag{4.131}$$

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

$$I_{cs2} = A_{st} z_{cst\text{-}na}^{\ 2} + A_{c,c}\left(z_{na\text{-}ccs,c}^{\ 2} + \frac{D_{ps}^{\ 2}}{12}\right) \tag{4.132}$$

$$\chi_{LT} = \min\left(\frac{1}{\Phi_{LT}+\sqrt{\Phi_{LT}^{\ 2}-0.75\lambda_{LT}^{\ 2}}}\ ;\ 1.0\ ;\ \frac{1}{\lambda_{LT}^{\ 2}}\right) \tag{4.133}$$

$$\lambda_{LT} = \sqrt{\frac{M_{pl,Rkh}}{M_{cr}}} \tag{4.134}$$

$$k_s = \frac{k_1 k_2}{k_1 + k_2} \tag{4.135}$$

$$\Phi_{LT} = 0.5\left\{1+\alpha_{LT}(\lambda_{LT}-0.4)+0.75\lambda_{LT}^{\ 2}\right\} \tag{4.136}$$

$$M_{cr} = \frac{k_c C_4}{L_{cr}}\sqrt{\left[\left\{\frac{E_a}{2(1+0.3)}\right\} I_{T,a} + \frac{k_s L_{cr}^{\ 2}}{\pi^2}\right] E_a I_{bfz}} \tag{4.137}$$

$$k_c = \frac{\left(\dfrac{z_{ctf\text{-}cbf} I_h}{I_{ay}}\right)}{\left(\dfrac{z_{ctf\text{-}cbf}^{\ 2}}{4} + i_{ax}^{\ 2}\right)/e + z_{ctf\text{-}cbf}} \tag{4.138}$$

$$I_h = I_{ay} + \frac{A_a A_{sl}\left\{D_a + 2\left(D_{ps} + h_{cs} - z_{tcs\text{-}csl}\right)\right\}^2}{4(A_a + A_{sl})} \tag{4.139}$$

$$i_{ax} = \sqrt{\frac{I_{ay}+I_{az}}{A_a}} \tag{4.140}$$

$$e = \frac{(A_a + A_{sl}) I_{ay}}{A_a z_{ccs\text{-}ca} A_{sl}} \tag{4.141}$$

$$I_{bfz} = \frac{t_f B_a^{\ 3}}{12} \tag{4.142}$$

$$k_1 = \frac{4E_a I_{cs2}}{B_b} \tag{4.143}$$

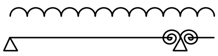
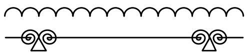
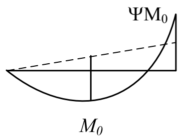
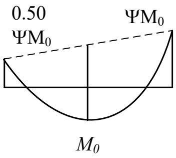
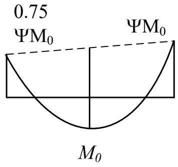
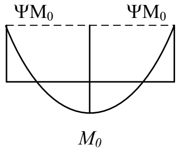
$$k_2 = \frac{E_a t_w^{\ 3}}{4\left(1\text{-}0.3^2\right) z_{ctf\text{-}cbf}} \tag{4.144}$$

where

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 secondary composite beam
I_{cs2}	is the second moment of area of cracked composite slab in direction transverse to secondary steel beam
λ_{LT}	is the non-dimensional slenderness for lateral-torsional buckling of secondary composite beam
k_s	is the transverse (rotational) stiffness per unit length of secondary composite beam
Φ_{LT}	is the value to determine reduction factor for lateral-torsional buckling of secondary composite beam
A_{st}	is the cross-sectional area of transverse reinforcing bars per unit length
$z_{cst\text{-}na}$	is the vertical distance between centre of transverse reinforcing bars and neutral axis of composite slab
$A_{c,c}$	is the area per unit length of concrete slab in compression
$z_{na\text{-}ccs,c}$	is the vertical distance between neutral axis of composite slab and centre of concrete slab in compression
$M_{pl,Rkh}$	is the characteristic value of plastic hogging moment resistance of secondary composite beam calculated by Eqs.(113)-(118) 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 secondary composite beam
C_4	is the property of distribution of moment, given in Table 4.5
L_{cr}	is the length of secondary 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 secondary steel beam about y-y axis
I_h	is the second moment of area of secondary composite beam on hogging moment region
A_{sl}	is the cross-sectional area of longitudinal reinforcing bars within b_{effh}
$z_{tcs\text{-}csl}$	is the covering depth of longitudinal reinforcing bars
i_{ax}	is the polar radius of gyration of area of secondary steel beam
e	is the e value

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

Table 4.5: 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

where

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

Commentary:

(1) Moment resistance in construction stage

Generally, Class 3 cross-sections would assume an elastic distribution of stresses, and the moment resistance can be calculated using their elastic section modulus. However, EN 1993-1-1 makes special allowances for the cross-sections with Class 3 web and Class 1 or 2 flanges by permitting the cross-sections to be classified as effective Class 2 cross-sections. Accordingly, part of the web in compression is neglected, and the plastic section properties for the remainder of the cross-section can be determined. In EN 1993-1-1, the proportion of the web in compression should be replaced by a part of $20\varepsilon t_w$ adjacent to the compression flange measured from the base of the root radius, with another part of $20\varepsilon t_w$ adjacent to the plastic neutral axis of the effective cross-section in accordance with Figure 4.12. A similar distribution can be applied to welded sections with the part of $20\varepsilon t_w$ adjacent to the compression flange measured from the base of the weld.

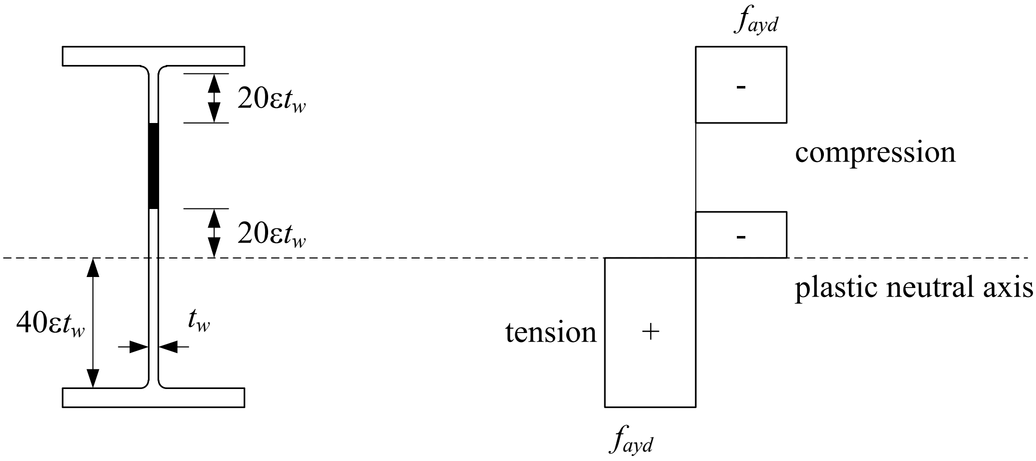


Figure 4.11: Effective Class 2 web

4.5.5 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 Eqs..

$$R_{st}+R_{pse} = A_{st}f_{sd}+A_{pse}f_{psd} \tag{4.145}$$

f_{psd} is given by:

$$f_{psd} = \frac{f_{psk}}{\gamma_{ps}} \tag{4.146}$$

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 4.4.3
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 the following Eq..

$$v_{Rd} = 0.6\left(1\text{-}\frac{f_{ck}}{250}\right)f_{cd}\sin\theta\cos\theta \tag{4.147}$$

where

f_{ck}	is the characteristic cylinder strength of normal weight concrete
f_{cd}	is the design strength of normal weight concrete, see 4.5.2
θ	is the angle between diagonal strut and axis of secondary beam, $26.5° \le \theta \le 45°$ for concrete flange in compression and $38.6° \le \theta \le 45°$ for concrete flange in tension

Chapter 5 Application to Construction

5.1 General

(1) Basis of construction

The performance of beam-to-beam composite joints with contact plates may be affected by the construction sequence and installation of the contact plates. Therefore, on-site construction can be implemented in accordance with the construction processes and the constructional requirements in this chapter in order to ensure the beam-to-beam joint can achieve the expected rotational stiffness and moment capacity.

(2) Construction process

On-site construction including the installation of contact plates can follow the procedures shown in Figure 5.1.

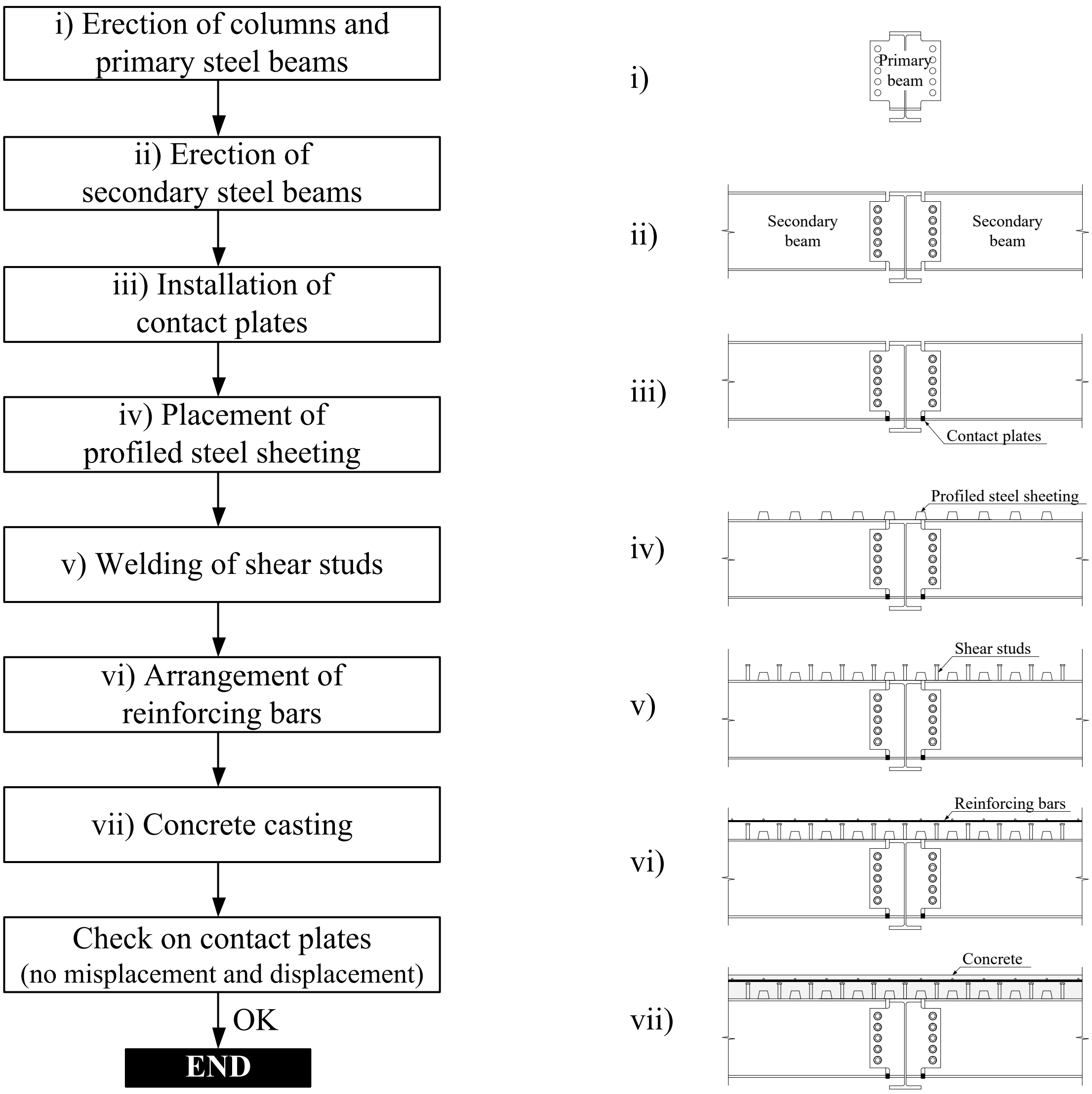


Figure 5.1: Construction process

5.2 Constructional Requirements

5.2.1 Contact Plates

(1) Detailing and installation method

The detailing and installation method of contact plates shall be specified for each project. A reliable contact between “bottom flange of secondary steel beam and contact plate” and between “stiffener (or bottom flange of primary steel beam) and contact plate” should be achieved to develop the required rotational stiffness of beam-to-beam joints.

(2) Quality assurance

Some kinds of preventable measures should be in placed to ensure that contact plates will not be easily displaced and fallen off during the construction stage and throughout the service life of the structure.

(3) Proper contact

One way to achieve proper contact is by direct welding the bottom flange of the secondary steel beam to the stiffener (or the bottom flange of the primary steel beam) through the gap as shown in Figure 5.2. Another way to achieve contact is to insert shimming plates or contact plates with some form of adjustment to ensure the gap is closed. Examples of contact plate are shown in Figure 5.3.

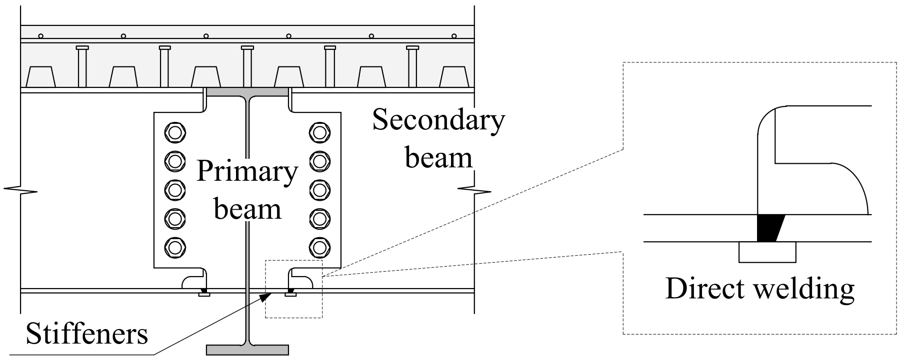
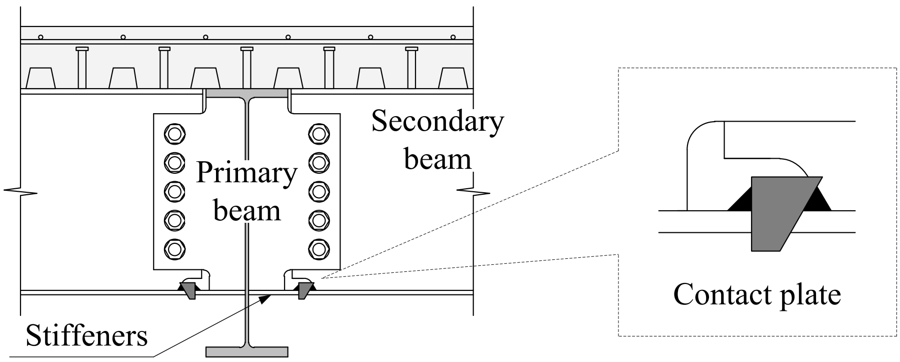
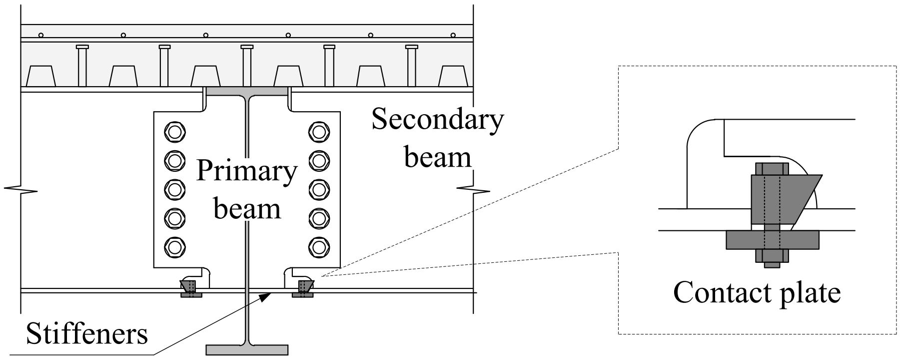


Figure 5.2: Direct welding through gap



(a) Welded contact plate



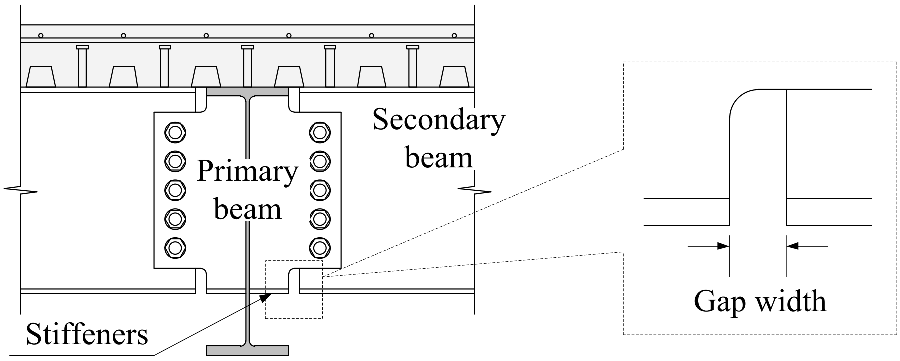
(b) Bolted contact plate

Figure 5.3: Examples of contact plate

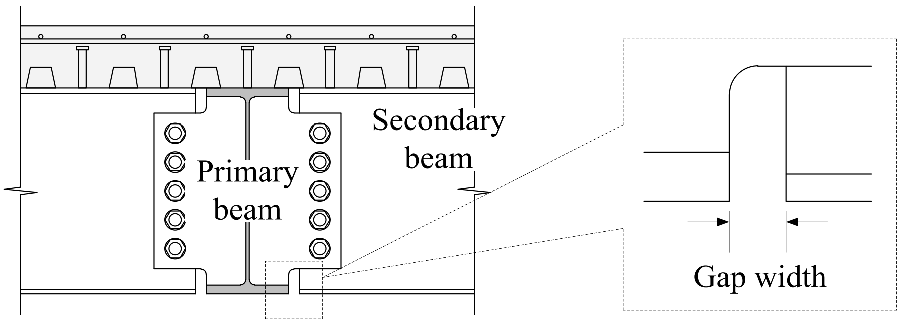
Commentary:

(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 secondary steel beam and stiffener or bottom flange of primary steel beam (see Figure 5.4) may fluctuate due to fabrication and construction tolerances. Therefore, it is necessary to provide contact plate that can be adjusted to fit the gap caused by these tolerances.



(a) Case for secondary beams of same depth



(b) Case for secondary beams of different depth

Figure 5.4: Gap width

(2) Quality assurance

When contact plates are installed before concrete casting as shown in Figure 5.1, the contact plate 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. Thus, some kinds of preventable measures should be in placed to ensure that contact plates will not be easily displaced and fallen off during the construction stage and throughout the service life of the structure.

5.2.2 Reinforcing Bars

(1) Concrete cover

The concrete cover for the reinforcing bars in concrete slab should be more than or equal to the minimum value specified in EN 1992-1-1.

(2) Lap length

When the longitudinal reinforcing bars in concrete slab are lapped over the hogging moment region, the arrangement of the lapped bars should comply with EN1992-1-1. The design lap length l_0 can be calculated by the following Eqs. unless the fire resistance design like membrane action is considered. Here, α_1, α_2, α_3, α_5, α_6, and $l_{b,\cancel{rqd}}$ should be referred to EN1992-1-1.

$$l_0 = \min\left(\alpha_1 \alpha_2 \alpha_3 \alpha_5 \alpha_6 l_b \; ; \; l_{0,min}\right) \tag{5.1}$$

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

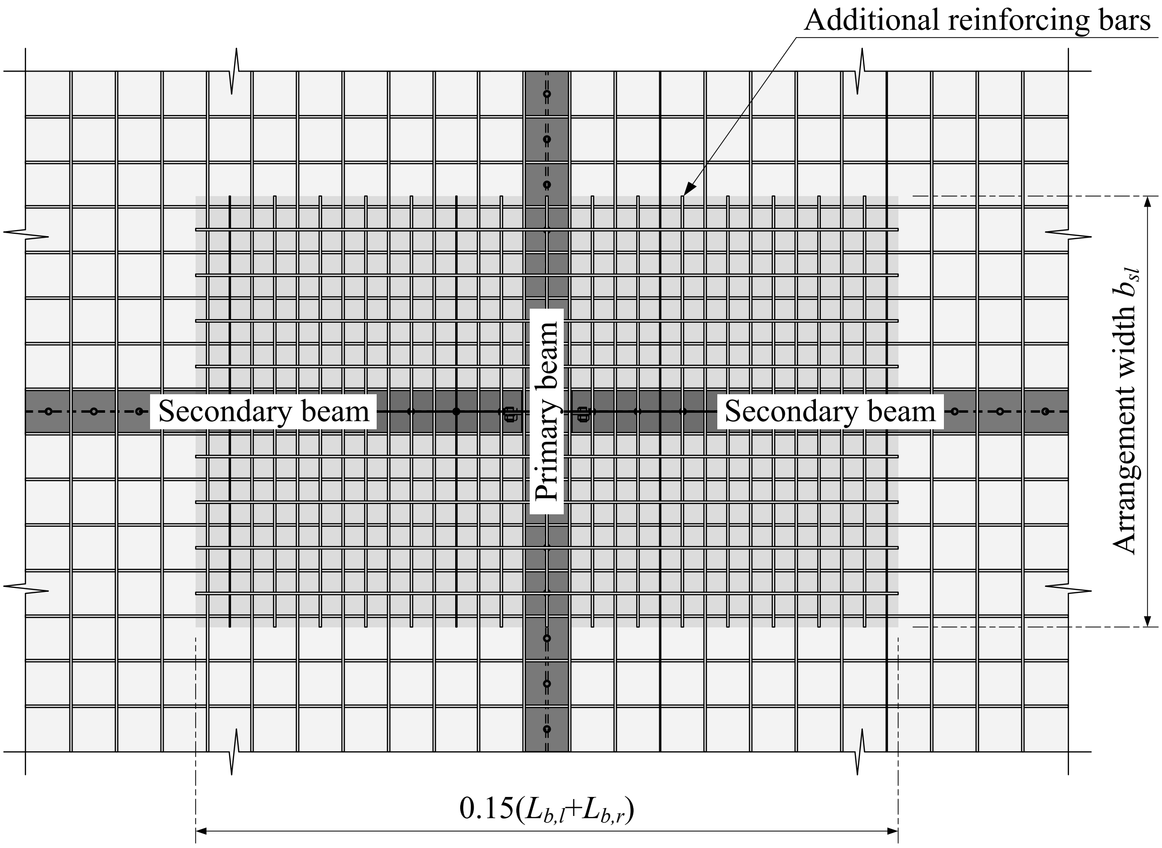
$$l_{0,min} = \max\left(0.3\alpha_6 l_b \; ; \; 15\phi_{sl} \; ; \; 200\right) \tag{5.2}$$

where

α_1	is the coefficient considering shape of bars
α_2	is the coefficient considering consrete 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	is the basic 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

(3) Additional reinforcing bars

Additional reinforcing bars can be arranged on beam-to-beam joints apart from anti-crack reinforcing bars. They should be arranged continuously in longitudinal direction of the secondary beam over the length, $0.15(L_{b,l}+L_{b,r})$, as shown in Figure 5.5. The arrangement width in transverse direction b_{sl} can be arbitrarily determined to be less than or equal to the secondary beam spacing. However, only the reinforcing bars within the effective width of beam-to-beam 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.



$L_{b,l}$: Beam length on left side

$L_{b,r}$: Beam length on right side

Figure 5.5: Arrangement of additional reinforcing bars

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 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 minimum value is determined by the requirements for both bond and environmental conditions. When the additional reinforcing bars mentioned in (3) are arranged on top of anti-crack reinforcing bars, it should be noted that the concrete cover is considered as the distance between the surface of the additional reinforcing bars closest to the nearest concrete surface.

(2) Lap length

As mentioned in Section 3.3, the longitudinal reinforcing bars in concrete slab should be continuous over the beam-to-beam composite joints. Generally, the longitudinal reinforcing bars are desirable 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 over the hogging moment region, the arrangement of the lapped bars should comply with EN1992-1-1.

(3) Additional longitudinal reinforcing bars

The rotational stiffness and the moment resistance of beam-to-beam 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 on beam-to-beam joints apart from anti-crack reinforcing bars to enhance the rotational stiffness and moment resistance of the composite joints.

Future Work

Although this design guide covers beam-to-beam composite joints with contact plates and secondary composite beams with composite joints, it is limited to the combination of primary composite beams and secondary composite beams as described in Chapter 3. This combination is often applied to the buildings with steel bracing frame system.

However, in the commercial buildings with center core wall, secondary composite beams may be connected to the peripheral concrete beams or concrete core wall, hence the secondary beam ends are less likely to be semi-rigid according to the scope of application in this design guide. Additionally, in recent years, the combination of primary precast concrete beams and secondary composite beams are widely used along with the spread of precast members. This combination is also out of scope in the current design guide.

Therefore, it is necessary to carry out the innovative research depending on the needs on modern buildings and expand the scope of application to the combination of concrete members and composite secondary beams in the future. Furthemore, it is desirable to achieve a more economical floor system by making it possible to apply semi-rigid joints to the primary beam ends (beam-to-column joints) as well.

Following the above, this publication will be revised in the future reflecting the latest research.

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Appendix I Simplified Analysis Method

This appendix presents the simplified analysis method for the design moment and deflection of composite beams with composite joints considering the effects of loading patterns. Step-by-step procedures utilizing the equations in subsection 4.3.2 (2) and the moment distribution method are described.

Typical composite floor plan shown in Figure AI.1 is considered. It should be noted that the joints located at the outer periphery or around the voids are designed as pinned joints because the tension forces cannot be transferred by the reinforcing bars, unless some special measures such as anchor reinforcing bars are taken. Here, the step-by-step procedures of the simplified analysis method are as follows.

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

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

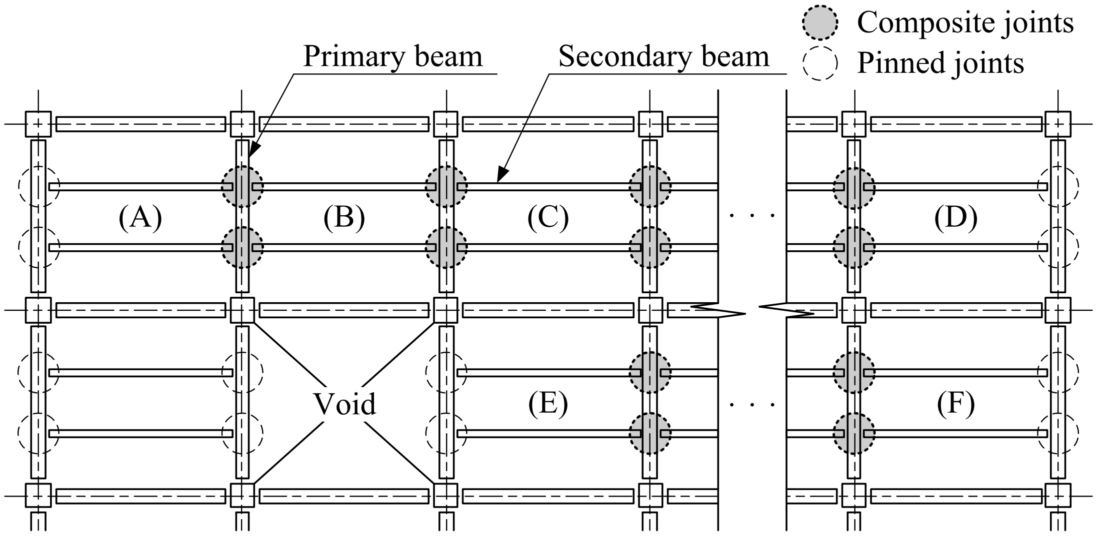


Figure AI.1: Typical composite floor plan

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

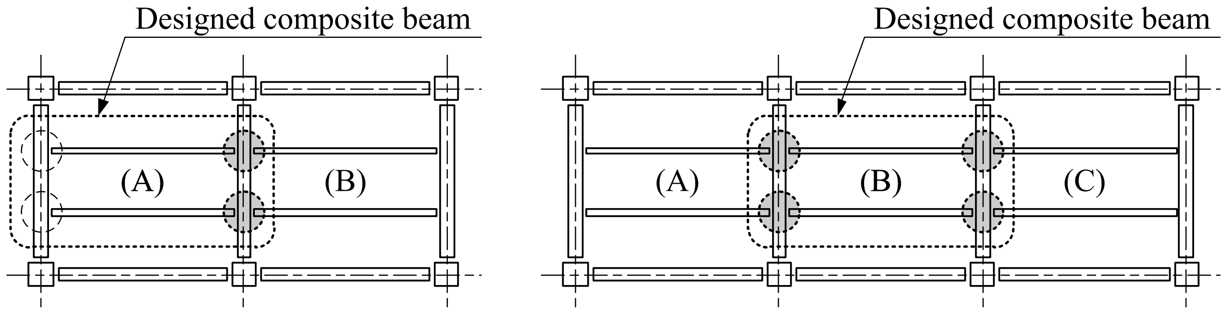
Firstly, all secondary composite beams are classified into the two types of beams, external beams or internal beams. Here, the external beams are the composite beams with composite joint at one end, and the internal beams are the composite beams with composite joints at the both ends. Accordingly, as regards the composite beams in the typical composite floor plan shown in Figure AI.1, the composite beam (A), (D), (E) and (F) are classified into the external beams, and the composite beam (B) and (C) are classified into the internal beams. Note that simply supported composite beams are not covered in this proposed analysis method.

(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 is designed, the virtual floor plan (continuous 3-span) including its adjacent beams on the 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 AI.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 AI.2 (b). Also for the other composite beams, their own virtual floor plans are extracted.

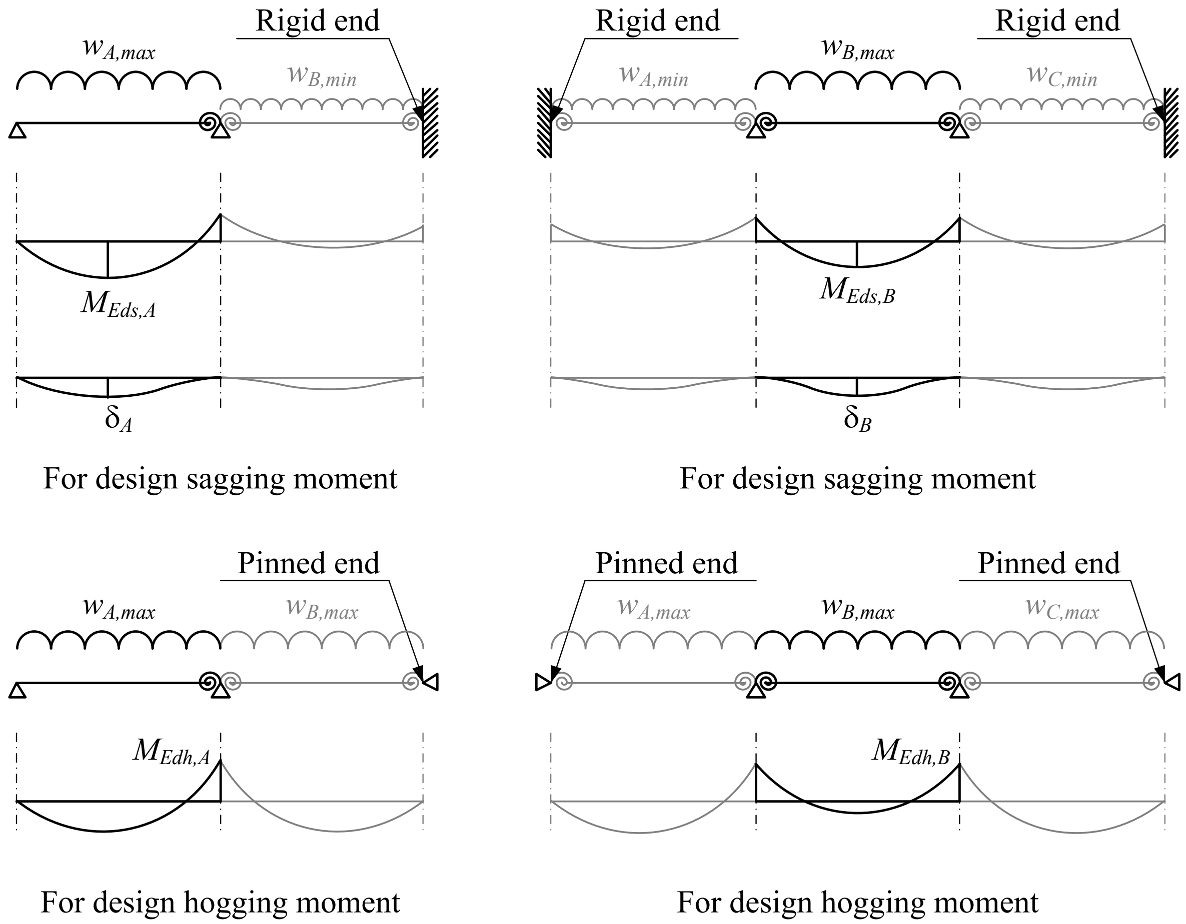
(3) Setting of end-restraint conditions and critical loading patterns

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 on the sagging moment region and on the hogging moment region may be different due to the effects 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 AI.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 the both ends are assumed to be rigid ends and the distributed loads on the composite beam (B) and the other composite beams are maximized and minimized respectively. Also, the hogging moment may be maximized when the 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 can maximize the sagging moment.



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

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



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

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

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

Finally, the design moment and deflection of secondary composite beams are analyzed in accordance with the above end-restraint conditions and the critical loading patterns. Note that they cannot be obtained only with the force equilibrium since the composite beams are statically indeterminate beams. However, the maximum design moment and deflection shown in Figure AI.3 can be calculated by utilizing the equations in subsection 4.3.2 (2) 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 AI.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 AI.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 subsection 4.3.2 (2). 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.(AI.1):

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

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.(AI.2).

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

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

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

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

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

In addition, the deflection δ_A can be also obtained from the equations in subsection 4.3.2 (2) by substituting $M_{h,A}$. Note that the yield 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 AI.3 can be also calculated by utilizing the equations in subsection 4.3.2 (2) and the moment distribution method.

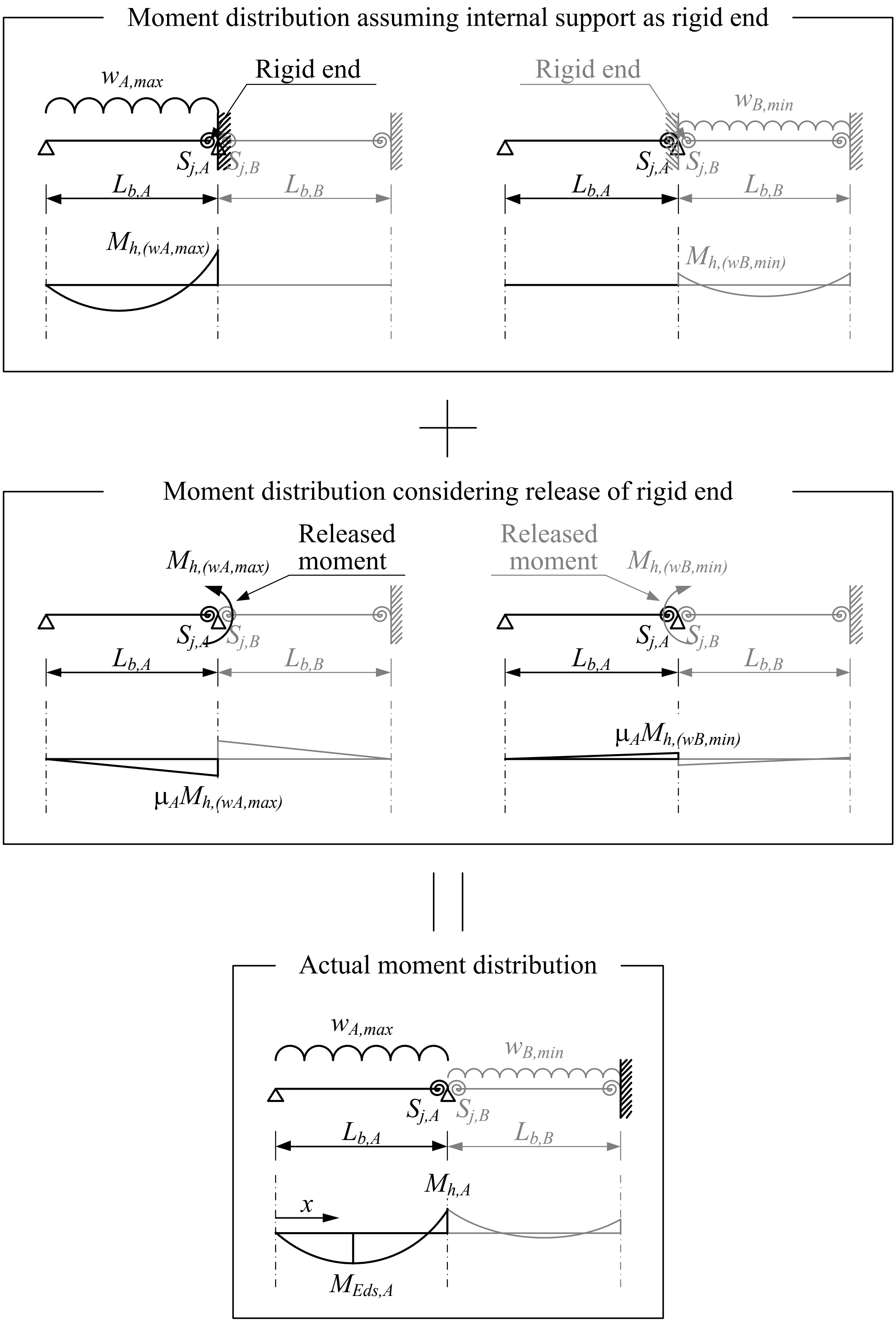


Figure AI.4: Principle of moment distribution method

Appendix II Design Example

This appendix presents the design example for a secondary composite beam with composite joints in accordance with the Eurocode approach. Figure AII.1 shows the composite floor plan in this design example. The designed beam member is an internal secondary composite beam supported by contact-type composite joints at both ends. 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 the 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 taken into account with the application of the simplified analysis method described in Appendix I.

In this design example, structural resistance checks at ultimate limit state and serviceability checks are mainly carried out. Based on Eurocode, serviceability limit state in buildings should take into account 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, $L_b/360$, $L_b/250$, and 4 [Hz] are considered as the limit of the deflection due to variable actions, the limit of the deflection due to permanent and variable actions, and the minimum natural frequency respectively. Besides, the limit of the crack width recommended in Eurocode 2, 0.3 [mm], is taken as the other serviceability criteria.

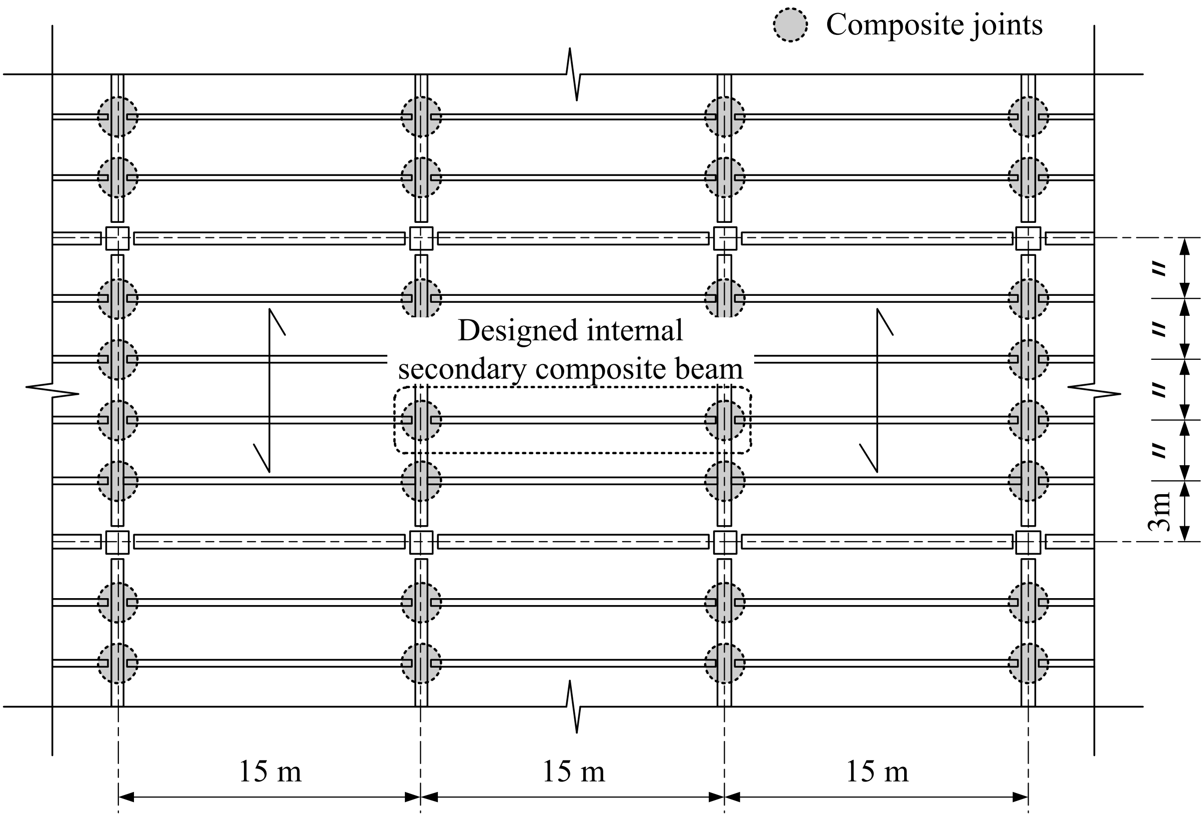


Figure AII.1: Composite floor plan

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

Design conditions

[Span and spacing]

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

[Steel beam]

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

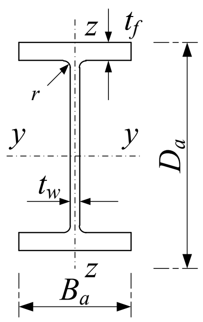


Figure AII.2: Cross-section of steel beam

[Profiled steel sheeting]

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

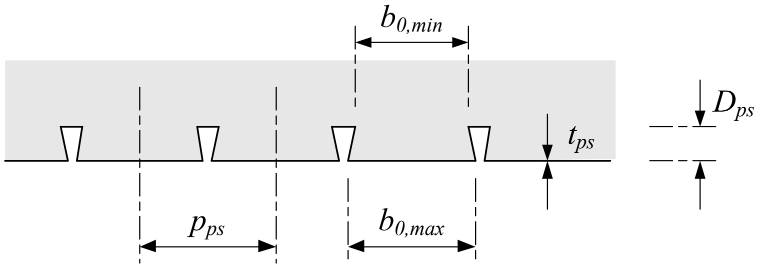


Figure AII.3: Cross-section of profiled steel sheeting

[Concrete slab]

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

[Reinforcing bar]

Diameter of anti-crack longitudinal reinforcing bars (row 1): $\phi_{sl,1}$ = 10 [mm]
Diameter of anti-crack transverse reinforcing bars (row 1): $\phi_{st,1}$ = 10 [mm]
Diameter of additional longitudinal reinforcing bars (row 2): $\phi_{sl,2}$ = 13 [mm]
Diameter of additional transverse reinforcing bars (row 2): $\phi_{st,2}$ = 13 [mm]
Pitch of anti-crack longitudinal reinforcing bars (row 1): $p_{sl,1}$ = 200 [mm]
Pitch of anti-crack transverse reinforcing bars (row 1): $p_{st,1}$ = 200 [mm]
Pitch of additional longitudinal reinforcing bars (row 2): $p_{sl,2}$ = 100 [mm]
Pitch of additional transverse reinforcing bars (row 2): $p_{st,2}$ = 100 [mm]
Covering depth of anti-crack longitudinal reinforcing bars (row 1): $z_{tcs-csl,1}$ = 45 [mm]
Covering depth of anti-crack transverse reinforcing bars (row 1): $z_{tcs-cst,1}$ = 35 [mm]
Covering depth of additional longitudinal reinforcing bars (row 2): $z_{tcs-csl,2}$ = 21 [mm]
Covering depth of additional transverse reinforcing bars (row 2): $z_{tcs-cst,2}$ = 34 [mm]

Arrangement width of additional longitudinal reinforcing bars (row 2): $b_{sl,2}$ = 1500 [mm]
Strength class: B500C
Characteristic yield strength: f_{sk} = 500 [N/mm^2]
Modulus of elasticity: E_s = 210000 [N/mm^2]
Partial factor: γ_s = 1.15
Design yield strength: $f_{sd} = f_{sk}/\gamma_s$ = 435 [N/mm^2]

[Headed stud]

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

[Fin plate]

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

[Contact plate]

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

[Bolt]

Size: M20 (d_b = 20 [mm])
Tensile stress area: A_b = 2.45 [cm^2]
Hole diameter: d_0 = 22 [mm]
Strength class: 8.8
Ultimate tensile strength: f_{bu} = 800 [N/mm^2]
Partial factor: γ_b = 1.25

Number on vertical line: $n_{b,v}$ = 7
Number on horizontal line: $n_{b,h}$ = 1
Pitch on vertical line: $p_{b,v}$ = 70 [mm]
Pitch on horizontal line: $p_{b,h}$ = 60 [mm]
Edge distance for fin plate on vertical line: $e_{b\text{-}fp,v}$ = 50 [mm]
Edge distance for fin plate on horizontal line: $e_{b\text{-}fp,h}$ = 50 [mm]
Edge distance for web of secondary beam on vertical line: $e_{b\text{-}bw,v}$ = 90 [mm]
Edge distance for web of secondary 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:

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

Weight per unit area of concrete slab and reinforcing bars:

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

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

Weight per unit area of profiled steel sheeting:

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

Weight per unit area of secondary 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): γ_Q = 1.50
Partial factor for variable actions (favourable): γ_{Qi} = 0.00

Design of semi-rigid composite joint

[Verifications of joint classification]

Check on initial rotational stiffness and yield moment resistance of beam-to-beam composite joint

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

$$z_{csl,1\text{-}cc} = D_a + D_{cs} - z_{tcs\text{-}csl,1} - \frac{t_f}{2} = 797 \text{ [mm]}$$

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

$$z_{csl,2\text{-}cc} = D_a + D_{cs} - z_{tcs\text{-}csl,2} - \frac{t_f}{2} = 822 \text{ [mm]}$$

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 reinforcing bars 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 reinforcing bars 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:

$$h_{eff,j} = 2h_{cj\text{-}fhs} = 400 \text{ [mm]}$$

Stiffness coefficient of longitudinal reinforcing bars for row 1:

$$k_{sl,1} = \frac{A_{sl,1}}{\left(\frac{h_{eff,j}}{2}\right)} = 2.4 \text{ [mm]}$$

Stiffness coefficient of longitudinal reinforcing bars for row 2:

$$k_{sl,2} = \frac{A_{sl,2}}{\left(\frac{h_{eff,j}}{2}\right)} = 8.0 \text{ [mm]}$$

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

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

Number of headed studs distributed over length l:

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

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

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

Equivalent vertical distance between longitudinal reinforcing bars and centre of contact part:

$$z_{sl,eq\text{-}cc} = D_a + D_{cs} - z_{tcs\text{-}sl,eq} - \frac{t_f}{2} = 816 \text{ [mm]}$$

Equivalent vertical distance between longitudinal reinforcing bars and centre of secondary steel beam:

$$z_{sl,eq\text{-}ca} = \frac{D_a}{2} + D_{cs} - z_{tcs\text{-}sl,eq} = 474 \text{ [mm]}$$

Parameter related to deformation of headed studs:

$$\xi = \frac{E_a I_{ay}}{z_{sl,eq\text{-}ca} E_s A_{sl,j}} = 2.16$$

$$\nu = \sqrt{\frac{(1+\xi) N k_{sc} l z_{sl,eq\text{-}ca}{}^2}{E_a I_{ay}}} = 4.09$$

Stiffness related to headed studs:

$$K_{sc} = \frac{N k_{sc}}{\nu - \left(\frac{\nu - 1}{1+\xi}\right)\left(\frac{z_{sl,eq\text{-}cc}}{z_{sl,eq\text{-}ca}}\right)} = 914228 \text{ [N/mm]}$$

Equivalent stiffness coefficient of longitudinal reinforcing bars:

$$k_{sl,eq} = \frac{A_{sl,1} + A_{sl,2}}{\left(\frac{h_{eff,j}}{2}\right)} = 10.3 \text{ [mm]}$$

Stiffness reduction factor due to deformation of headed studs:

$$k_{slip} = \frac{1}{1 + \left(\frac{E_s k_{sl,eq}}{K_{sc}}\right)} = 0.30$$

Initial rotational stiffness:

$$S_{j,ini} = E_s k_{slip} \sum k_{sl,r} z_{csl,r\text{-}cc}{}^2 = 428108 \text{ [kNm/rad]}$$

Effective width of secondary 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 secondary composite beam assuming simply supported condition:

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

Upper boundary of rotational stiffness for nominally pinned joint:

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

$$\therefore\ S_{j,ini} > \frac{0.5 E_a I_b}{L_b} \quad \text{OK} \left(\frac{\left(\frac{0.5 E_a I_b}{L_b}\right)}{S_{j,ini}} = 0.04\right)$$

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

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

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

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

Cross-sectional area of bottom flange of secondary steel beam:

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

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

$A_{cp} = 120.0$ [cm^2]

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

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

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

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

Yield moment resistance:

$M_{j,Rd} = z_{sl,eq\text{-}cc} \min\left(R_{sl,j}\ ;\ R_{con}\right) = 732.2$ [kNm]

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

$M_{pl,Rd} = 1289.8$ [kNm]

Upper boundary of moment resistance for nominally pinned joint:

$0.25 M_{pl,Rd} = 322.5$ [kNm]

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

[Verifications of structural resistance in composite stage]

Check on bolt group resistance at semi-rigid composite joint

Correction factor for bolt shear resistance:

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

Shear resistance of a single bolt:

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

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

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

α factor:

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

β factor:

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

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 secondary composite beam):

$V_{Ed} = 383.4$ [kN]

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

< In fin plate >

k_1 factor for vertical bolt bearing resistance:

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

Correction factor for vertical bolt bearing resistance:

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

Vertical bearing resistance of a single bolt:

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

k_1 factor for horizontal bolt bearing resistance:

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

Correction factor for horizontal bolt bearing resistance:

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

Horizontal bearing resistance of a single bolt:

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

Bolt bearing resistance:

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

< In web of secondary steel beam >

Nominal value of yield strength of web of secondary steel beam:

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

Ultimate tensile strength of web of secondary steel beam:

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

k_1 factor for vertical bolt bearing resistance:

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

Correction factor for vertical bolt bearing resistance:

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

Vertical bearing resistance of a single bolt:

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

k_1 factor for horizontal bolt bearing resistance:

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

Correction factor for horizontal bolt bearing resistance:

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

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:

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

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

Check on fin plate resistance at semi-rigid composite joint

Shear resistance for gross section:

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

Shear area for net section:

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

Shear resistance for net section:

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

Net area subjected to tension:

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

Net area subjected to shear:

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

Block shear resistance:

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

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

Elastic moment resistance:

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

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

Fin plate type:

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

Non-dimensional slenderness for lateral torsional buckling:

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

Value to determine reduction factor for lateral-torsional buckling:

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

Reduction factor for lateral-torsional buckling:

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

Lateral torsional buckling moment resistance:

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

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

Check on resistance of web of secondary steel beam at semi-rigid composite joint

Shear area for gross section:

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

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:

$$A_{bwV,n} = A_{bwV,g}\text{-}n_{b,v}d_0t_w = 56.2 \text{ [cm}^2\text{]}$$

Shear resistance for net section:

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

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

Elastic moment resistance on vertical line of bolts:

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

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

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

Design shear force on vertical line of bolts:

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

Plastic shear resistance on vertical line of bolts:

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

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

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

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

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

Check on fillet weld of fin plate at semi-rigid composite joint

Effective throat thickness of fillet weld:

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

Required minimum throat thickness of fillet weld:

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

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

[Verification of serviceability in composite stage]

Analysis of design moment of semi-rigid composite joint

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

Maximum design distributed load:

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

Design hogging moment (Load-case 2):

$M_{Edh} = 405.9$ [kNm]

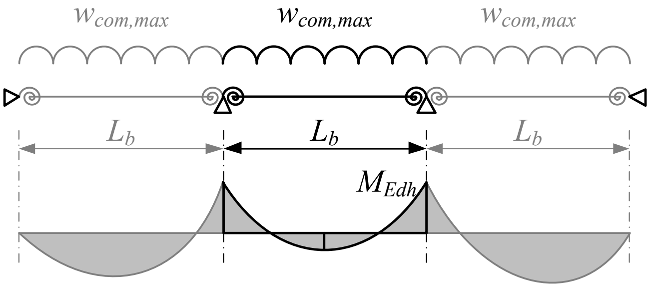


Figure AII.4: Design load with corresponding moment (Load-case 2)

Check on elastic moment resistance of semi-rigid composite joint

Elastic moment resistance:

$$\frac{2}{3}M_{j,Rd} = 488.1 \text{ [kNm]}$$

$$\therefore \ \frac{2}{3}M_{j,Rd} > M_{Edh} \quad \text{OK} \left(\frac{M_{Edh}}{\left(\frac{2}{3}M_{j,Rd}\right)} = 0.83 \right)$$

Design of internal beam

[Verifications of structural resistance in construction stage]

Analysis of design moment and shear force of steel beam

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:

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

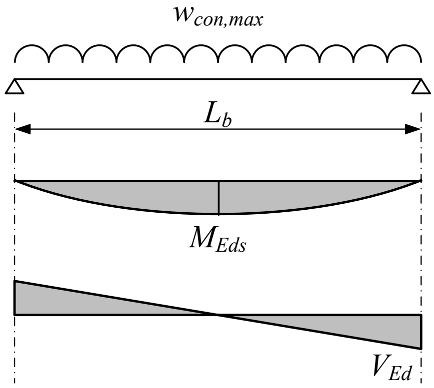


Figure AII.5: Design load with corresponding moment and shear force

Check on classification of steel beam

Classification of steel flange:

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

Classification of steel web:

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

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

Check on shear resistance and moment resistance of steel beam

Shear area:

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

Plastic shear resistance:

$$V_{pl,a,Rd} = \frac{A_V f_{ayd}}{\sqrt{3}} = 1478.7 \text{ [kN]}$$

Nominal value of yield strength of web:

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

Minimum shear buckling coefficient:

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

Modified slenderness of web:

$$\lambda_w = 0.76 \sqrt{\frac{f_{wy}}{k_{\tau,min}\left\{190000\left(\frac{t_w}{B_a}\right)^2\right\}}} = 1.06$$

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:

$$V_{b,a,Rd} = \min\left\{\frac{\chi_w f_{wy}\left(D_a\text{-}2t_f\text{-}2r\right)t_w}{\sqrt{3}\gamma_a} \; ; \; \frac{1.2 f_{wy}\left(D_a\text{-}2t_f\text{-}2r\right)t_w}{\sqrt{3}\gamma_a}\right\} = 969.3 \text{ [kN]}$$

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

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

Effective plastic section modulus:

$$W_{eff,pl,a} = \left\{\begin{array}{c} B_a t_f\left(D_a\text{-}t_f\right) + 0.4292r^2\left(D_a\text{-}2t_f\text{-}0.4467r\right) + t_w r\left(D_a\text{-}2t_f\text{-}r\right) \\ +\frac{t_w}{2}\left(40t_w\sqrt{\frac{235}{f_{ay}}}\right)^2 + \frac{t_w}{2}\left(20t_w\sqrt{\frac{235}{f_{ay}}}\right)^2 \\ +t_w\left(20t_w\sqrt{\frac{235}{f_{ay}}}\right)\left(D_a\text{-}t_f\text{-}r\text{-}40t_w\sqrt{\frac{235}{f_{ay}}}\text{-}t_f\text{-}r\text{-}10t_w\sqrt{\frac{235}{f_{ay}}}\right) \end{array}\right\} = 3219 \text{ [cm}^3\text{]}$$

Plastic moment resistance:

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

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

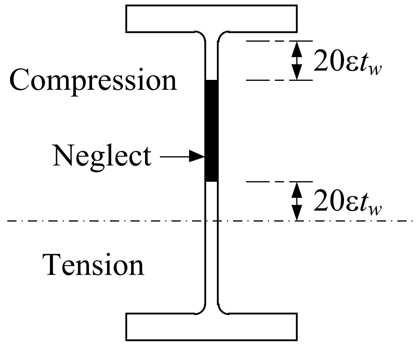


Figure AII.6: Effective cross-section for effective Class 2

[Verifications of structural resistance in composite stage]

Analysis of design moment and shear force of composite beam

Design moment and shear force in composite stage can be calculated by the simplified analysis method described in Appendix I.

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} = 945.3 [kNm]

Design hogging moment (Load-case 2):

M_{Edh} = 732.2 [kNm]

Design shear force:

V_{Ed} = 383.4 [kN]

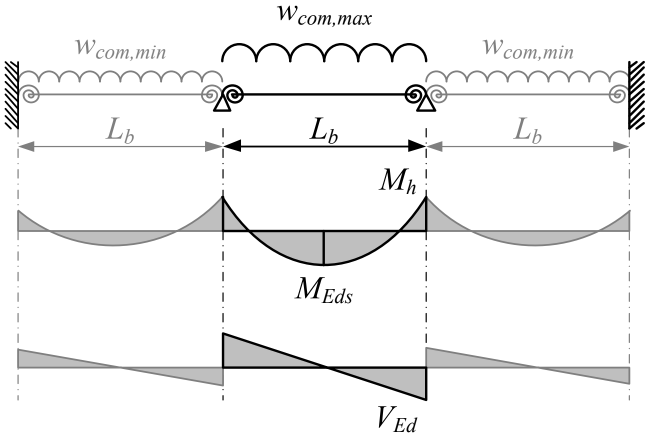


Figure AII.7: Design load with corresponding moment and shear force (Load-case 1)

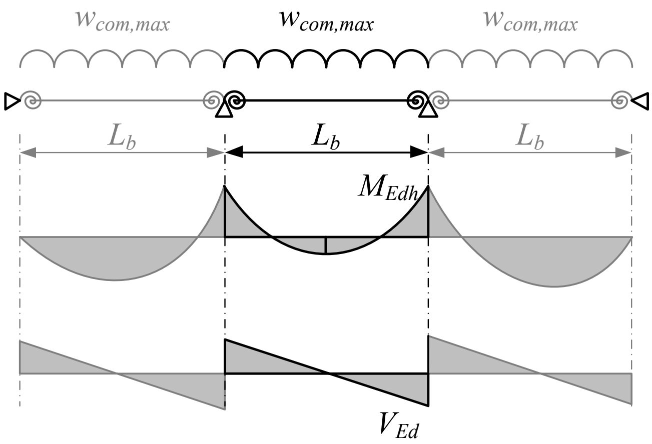


Figure AII.8: Design load with corresponding moment and shear force (Load-case 2)

Check on classification of composite beam

< Load-case 1 for maximizing sagging moment >

Classification of steel flange on sagging moment region:

Bottom flange is in tension $\rightarrow$ Class 1

x-coordinate at inflection point:

$$x_0 = \frac{L_b}{2}\left(1-\sqrt{1-\frac{8M_h}{w_{com,max}L_b^2}}\right) = 1419 \text{ [mm]}$$

Effective width on sagging moment region:

$$b_{effs} = b_{0s}+\min\left(\frac{L_b-2x_0}{4}\ ;\ B_b-b_{0s}\right) = 3000.0 \text{ [mm]}$$

Compression resistance of composite slab within b_{effs}:

$$R_{cs} = b_{effs}h_{cs}\left(0.85f_{cd}\right) = 4207.5 \text{ [kN]}$$

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 on sagging moment region:

Full web is in tension $\rightarrow$ Class 1

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

< Load-case 2 for maximizing hogging moment >

Classification of steel flange on hogging moment region:

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

x-coordinate at inflection point:

$$x_0 = \frac{L_b}{2}\left(1-\sqrt{1-\frac{8M_{Edh}}{w_{com,max}L_b^2}}\right) = 2246 \text{ [mm]}$$

Effective width on hogging moment region:

$$b_{effh} = b_{0h} + \min\left(\frac{2x_0}{4}\ ;\ B_b - b_{0h}\right) = 1223.0\ [\text{mm}]$$

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

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

Tension resistance of longitudinal reinforcing bars within b_{effh}:

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

Portion of part of cross-section in compression

$$\alpha = \frac{\left(\frac{D_a - 2t_f - 2r}{2}\right) + \left(\frac{R_{sl}}{2t_w f_{ayd}}\right)}{D_a - 2t_f - 2r} = 0.66$$

Classification of steel web on hogging moment region:

$$\frac{D_a - 2t_f - 2r}{t_w} = 70.2 > \frac{456\sqrt{\frac{235}{f_{ay}}}}{13\alpha - 1} = 67.5 \quad \rightarrow \text{Class 3}$$

∴ Class 1 steel flange & Class 3 steel web → Effective Class 2 OK

Modular ratio for short-term loading:

$$n_0 = \frac{E_a}{E_{cm}} = 6.77$$

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

$$z_0 = \frac{(A_a + A_{sl})\left(0.5D_a + D_{ps} + 0.5h_{cs}\right)}{A_a + A_{sl} + \left(\frac{h_{cs} b_{effh}}{n_0}\right)} = 203.7\ [\text{mm}]$$

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:

$$\rho_{sl,req} = \frac{f_{ay}}{235}\frac{f_{ctm}}{f_{sk}}\sqrt{k_c} = 0.79\%\ \text{(for Class 2 cross-section)}$$

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

$$A_{cs} = b_{effh} h_{cs} = 1210.8\ [\text{cm}^2]$$

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

$$A_{sl,req} = \rho_{sl,req} A_{cs} = 9.51\ [\text{cm}^2]$$

$$\therefore A_{sl} > A_{sl,req} \quad \text{OK} \quad \left(\frac{A_{sl,req}}{A_{sl}} = 0.46\right)$$

Check on minimum degree of shear connection of composite beam

< Load-case 1 for maximizing sagging moment >

Distance between inflection points on sagging moment region:

$L_{es} = L_b\text{-}2x_0 = 12163$ [mm]

Correction factor of headed stud taking into account h_{hs}/d_{hs}:

$$\alpha_{hs} = 1.00 \left(\text{for } \frac{h_{hs}}{d_{hs}} > 4\right)$$

Shear resistance of a headed stud:

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

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

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

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

$k_{ts,max} = 0.70$ (for $n_{hss} = 2$, $t_{ps} \leq 1$, $d_{hs} \leq 20$, and welded through)

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

$$k_{ts} = \min\left\{\frac{0.7}{\sqrt{\min(n_{hss} \ ; 2)}} \frac{b_{0,min}}{D_{ps}}\left(\frac{h_{hs}}{D_{ps}}\text{-}1\right) \ ; k_{ts,max}\right\} = 0.70$$

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

$R_{qs} = N_{hss}k_{ts}P_{Rd} = 3543.9$ [kN]

Tension (Compression) resistance of secondary steel beam:

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

Degree of shear connection on sagging moment region:

$$\eta_s = \frac{R_{qs}}{\min(R_a \ ; R_{cs})} = 0.84$$

Required minimum degree of shear connection on sagging moment region:

$$\eta_{s,req} = \max\left\{1\text{-}\left(\frac{355}{f_{ayd}}\right)(0.75\text{-}0.03L_{es}); 0.4\right\} = 0.61 \ (\text{for } L_{es} \leq 25)$$

$$\therefore \ \eta_s > \eta_{s,req} \quad \text{OK} \ \left(\frac{\eta_{s,req}}{\eta_s} = 0.73\right)$$

< Load-case 2 for maximizing hogging moment >

Half of distance between inflection points on hogging moment region:

$$\frac{L_{eh}}{2} = x_0 = 2246 \text{ [mm]}$$

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

$$N_{hsh} = \left\lfloor \frac{\left(\frac{L_{eh}}{2}\right)\text{-}h_{cj\text{-}fhs}}{p_{ps}} \right\rfloor n_{hsh} = 22$$

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

$k_{th,max} = 0.70$ (for $n_{hsh} = 1$, $t_{ps} \leq 1$, $d_{hs} \leq 20$, and welded through)

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

$$k_{th} = \min\left\{\frac{0.7}{\sqrt{\min(n_{hsh}\ ;\ 2)}}\frac{b_{0,min}}{D_{ps}}\left(\frac{h_{hs}}{D_{ps}}-1\right)\ ;\ k_{th,max}\right\} = 0.70$$

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

$R_{qh} = N_{hsh}k_{th}P_{Rd} = 1257.5$ [kN]

Degree of shear connection on hogging moment region:

$$\eta_h = \frac{R_{qh}}{\min(R_a\ ;\ R_{sl})} = 1.40$$

Required minimum degree of shear connection on hogging moment region:

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

$$\therefore\ \eta_h > \eta_{h,req}\quad \text{OK}\ \left(\frac{\eta_{h,req}}{\eta_h} = 0.71\right)$$

Check on shear resistance and moment resistance of composite beam

Plastic shear resistance:

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

Shear buckling resistance:

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

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

< Load-case 1 for maximizing sagging moment >

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

$R_w = R_a\text{-}2B_at_ff_{ayd} = 2233.0$ [kN]

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

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

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

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

Location of plastic neutral axis for full shear connection:

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

Plastic sagging moment resistance with full shear connection:

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

Location of plastic neutral axis for partial shear connection:

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

Plastic sagging moment resistance with partial shear connection:

$$M_{plp,Rds} = R_a\frac{D_a}{2}+R_{qs}\left(D_{cs}\text{-}\frac{R_{qs}}{R_{cs}}\frac{D_{cs}\text{-}D_{ps}}{2}\right)-\frac{\left(R_a\text{-}R_{qs}\right)^2}{4B_af_{ayd}} = 1957.3 \text{ [kNm]}$$

$$\therefore\ \min\left(M_{plf,Rds}\ ;\ M_{plp,Rds}\right) > M_{Eds}\quad \text{OK}\ \left(\frac{M_{Eds}}{\min\left(M_{plf,Rds}\ ;\ M_{plp,Rds}\right)} = 0.48\right)$$

< Load-case 2 for maximizing hogging moment >

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

$z_{sl,eq\text{-}tf} = D_{cs}\text{-}z_{tcs\text{-}sl,eq} = 123.9$ [mm]

Tension (Compression) resistance of effective steel beam:

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

Location of plastic neutral axis for with full shear connection:

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

Plastic hogging moment resistance with full shear connection:

$$M_{plf,Rdh} = \left\{ \begin{array}{c} W_{pl,a}f_{ayd}+R_{sl}\left(\dfrac{D_a}{2}+z_{sl,eq\text{-}tf}\right) \\ -\dfrac{R_{sl}^2+(R_v+R_{sl})\left(R_v+R_{sl}\text{-}2R_{eff,v}\right)}{4t_wf_{ayd}} \end{array} \right\} = 1289.8 \text{ [kNm]}$$

Tension (Compression) resistance of flange of steel beam:

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

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

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

Plastic moment resistance after deducting shear area:

$$M_{pl,f,Rd} = R_f z_{ctf\text{-}cbf}+R_{sl}\left(\frac{D_a}{2}+z_{sl,eq\text{-}tf}\right) = 1202.3 \text{ [kNm]}$$

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

$$M_{y,v,Rdh} = M_{plf,Rdh} = 1289.8 \text{ [kNm]} \left(\text{for } V_{Ed} \leq \frac{V_{pl,Rd}}{2}\right)$$

$$\therefore \min\left(M_{plf,Rdh}\,;\,M_{y,v,Rdh}\right) > M_{Edh} \quad \text{OK} \left(\frac{M_{Edh}}{\min\left(M_{plf,Rdh}\,;\,M_{y,v,Rdh}\right)} = 0.57\right)$$

<u>Check on lateral-torsional buckling of composite beam</u>

< 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:

$$e \ = \frac{(A_a+A_{sl})I_{ay}}{A_a z_{ccs\text{-}ca} A_{sl}} = 1328.8$$

Second moment of area on hogging moment region:

$$I_h = I_{ay}+\frac{A_a A_{sl}\left\{D_a+2\left(D_{ps}+h_{cs}\text{-}z_{tcs\text{-}sl,eq}\right)\right\}^2}{4(A_a+A_{sl})} = 140126 \text{ [cm}^4\text{]}$$

k_c factor:

$$k_c = \frac{\frac{z_{ctf\text{-}cbf} I_h}{I_{ay}}}{\left(\frac{z_{ctf\text{-}cbf}^2}{4}+i_{ax}^2\right)/e+z_{ctf\text{-}cbf}} = 1.15$$

Property of distribution of moment:

$$C_4 = 21.6 \left(\text{for } 0.50 < \psi = \frac{M_{Edh}}{\left(\frac{w_{com,max} L_b^2}{8}\right)} < 0.75 \right)$$

Length between points at which bottom flange of steel beam is laterally restrained:

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

Cross-sectional area of transverse reinforcing bars per unit length:

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

Area per unit length of concrete slab in compression:

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

Equivalent vertical distance between transverse reinforcing bars and concrete slab in compression:

$$z_{st,eq\text{-}ccs,c} = D_{cs}\text{-}z_{tcs\text{-}st,eq}\text{-}\frac{D_{ps}}{2} = 90.7 \text{ [mm]}$$

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

$$z_{st,eq\text{-}na} = \frac{z_{st,eq\text{-}ccs,c}}{\left(1+\frac{A_{st}}{A_{c,c}}\right)} = 72.1 \text{ [mm]}$$

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

$z_{na\text{-}ccs,c} = z_{st,eq\text{-}ccs,c}\text{-}z_{st,eq\text{-}na} = 18.6$ [mm]

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

$$I_{cs2} = A_{st} z_{st,eq\text{-}na}^2 + A_{c,c}\left(z_{na\text{-}ccs,c}^2+\frac{D_{ps}^2}{12}\right) = 1269.1 \text{ [cm}^4\text{]}$$

Cracked flexural stiffness per unit length of composite slab:

$E_a I_{cs2} = 2665.0$ [kNm^2/m]

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

$$k_1 = \frac{4E_a I_{cs2}}{B_b} = 3553.4 \text{ [kN/rad] (for continuous slab)}$$

Flexural stiffness of web of steel beam:

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

Transverse (rotational) stiffness per unit length:

$$k_s = \frac{k_1 k_2}{k_1+k_2} = 60.4 \text{ [kN/rad]}$$

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

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

Elastic critical moment for lateral-torsional buckling:

$$M_{cr} = \frac{k_c C_4}{L_{cr}} \sqrt{\left[\left\{\frac{E_a}{2(1+0.3)}\right\} I_{T,a} + \frac{k_s L_{cr}{}^2}{\pi^2}\right] E_a I_{bfz}} = 2973.8 \text{ [kNm]}$$

Location of plastic neutral axis for $M_{pl,Rkh}$:

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

Characteristic value of plastic hogging moment resistance:

$$M_{pl,Rkh} = \left\{ \begin{array}{c} W_{pl,a} f_{ayd} + A_{sl} f_{sk} \left(\frac{D_a}{2} + z_{sl,eq\text{-}tf}\right) \\ -\dfrac{\left(A_{sl} f_{sk}\right)^2 + \left(R_v + A_{sl} f_{sk}\right)\left(R_v + A_{sl} f_{sk} - 2R_{eff,v}\right)}{4 t_w f_{ayd}} \end{array} \right\} = 1305.3 \text{ [kNm]}$$

Non-dimensional slenderness for lateral-torsional buckling:

$$\lambda_{LT} = \sqrt{\frac{M_{pl,Rkh}}{M_{cr}}} = 0.66$$

Imperfection factor corresponding to appropriate lateral-torsional buckling curve:

$$\alpha_{LT} = 0.76 \left(\text{for } \frac{D_a}{B_a} = 3.5 > 2.0\right)$$

Value to determine reduction factor for lateral-torsional buckling:

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

Reduction factor for lateral-torsional buckling:

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

Buckling moment resistance of laterally unrestrained composite beam:

$$M_{b,Rd} = \chi_{LT} M_{plf,Rdh} = 1016.1 \text{ [kNm]}$$

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

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

$$\therefore M_{b,Rd} > M_{Edh} \quad \text{OK} \left(\frac{M_{Edh}}{M_{b,Rd}} = 0.72\right)$$

Check on longitudinal shear resistance of composite beam:

< Load-case 1 for maximizing sagging moment >

Effective width on hogging moment region:

$$b_{effh} = b_{0h} + \min\left(\frac{2x_0}{4} \; ; \; B_b - b_{0h}\right) = 809.3 \text{ [mm]}$$

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

$$A_{sl} = \pi\left(\frac{\phi_{sl,1}}{2}\right)^2 \left\lfloor\frac{b_{effh}}{p_{sl,1}}\right\rfloor + \pi\left(\frac{\phi_{sl,2}}{2}\right)^2 \min\left(\left\lfloor\frac{b_{effh}}{p_{sl,2}}\right\rfloor ; \left\lfloor\frac{b_{sl,2}}{p_{sl,2}}\right\rfloor\right) = 13.76\ [\mathrm{cm}^2]$$

Tension resistance of longitudinal reinforcing bars within b_{effh}:

$$R_{sl} = A_{sl}f_{sd} = 598.3\ [\mathrm{kN}]$$

Change of longitudinal force in composite slab:

$$\Delta N_L = \left\{\begin{matrix} \min(R_a ; R_{cs} ; N_{hss}P_{Rd}) \\ +\min\left(\left\lceil\frac{x_0\text{-}h_{cj\text{-}fhs}}{p_{ps}}\right\rceil n_{hsh}P_{Rd} ; R_{sl}\right)\end{matrix}\right\} = 4806\ [\mathrm{kN}]$$

Design longitudinal shear stress in composite slab:

$$v_{L,Ed} = \frac{\Delta N_L}{2h_{cs}\left(\frac{L_b}{2}\right)} = 3.24\ [\mathrm{N/mm}^2]$$

Minimum angle to minimize cross-sectional area of transverse reinforcing bars:

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

Required tension resistance of transverse reinforcement per unit length:

$$R_{tr,req} = \frac{1000h_{cs}v_{L,Ed}}{\cot\theta_{min}} = 255.8\ [\mathrm{kN/m}]$$

< Load-case 2 for maximizing hogging moment >

Effective width on sagging moment region:

$$b_{effs} = b_{0s}+\min\left\{\frac{(L_b\text{-}2x_0)}{4} ; B_b\text{-}b_{0s}\right\} = 2727.0\ [\mathrm{mm}]$$

Compression resistance of composite slab within b_{effs}:

$$R_{cs} = b_{effs}h_{cs}\left(0.85f_{cd}\right) = 3824.6\ [\mathrm{kN}]$$

Distance between inflection points on sagging moment region:

$$L_{es} = L_b\text{-}2x_0 = 10508\ [\mathrm{mm}]$$

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

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

Change of longitudinal force in composite slab:

$$\Delta N_L = \left\{\begin{matrix} \min(R_a ; R_{cs} ; N_{hss}P_{Rd}) \\ +\min\left(\left\lceil\frac{x_0\text{-}h_{cj\text{-}fhs}}{p_{ps}}\right\rceil n_{hsh}P_{Rd} ; R_{sl}\right)\end{matrix}\right\} = 4722\ [\mathrm{kN}]$$

Design longitudinal shear stress in composite slab:

$$v_{L,Ed} = \frac{\Delta N_L}{2h_{cs}\left(\frac{L_b}{2}\right)} = 3.18\ [\mathrm{N/mm}^2]$$

Minimum angle to minimize cross-sectional area of transverse reinforcing bars:

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

Required tension resistance of transverse reinforcement per unit length:

$$R_{tr,req} = \frac{1000h_{cs}v_{L,Ed}}{\cot\theta_{min}} = 251.3 \text{ [kN/m]}$$

Tension resistance of transverse reinforcement per unit length:

$$R_{st}+R_{pse} = A_{st}f_{sd}+A_{pse}f_{psd} = 1245.6 \text{ [kN/m]}$$

$$\therefore\ R_{st}+R_{pse} > R_{tr,req} \quad \text{OK} \left(\frac{R_{tr,req}}{R_{st}+R_{pse}} = 0.21\right)$$

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

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

$$\therefore\ A_{st} > A_{st,req} \quad \text{OK} \left(\frac{A_{st,req}}{A_{st}} = 0.05\right)$$

Crushing shear stress of concrete slab:

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

$$\therefore\ v_{Rd} > v_{L,Ed} \quad \text{OK} \left(\frac{v_{L,Ed}}{v_{Rd}} = 0.74\right)$$

[Verifications of serviceability in construction stage]

<u>Analysis of deflection of steel beam</u>

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 and live 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”:

$$\delta_P = \frac{w_{con,P}L_b^{\,4}}{384E_aI_{ay}} = 39.6 \text{ [mm]}$$

Deflection due to “live loads”:

$$\delta_V = \frac{w_{con,V}L_b^{\,4}}{384E_aI_{ay}} = 4.7 \text{ [mm]}$$

Deflection due to “dead loads and live loads”:

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

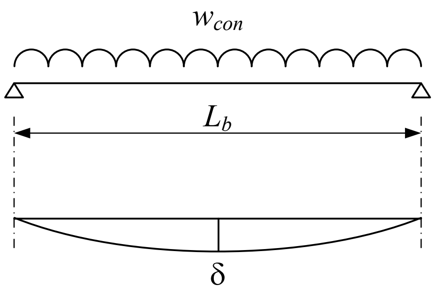


Figure AII.9: Design load with corresponding deflection

Check on deflection of steel beam

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 of composite beam

Deflection in composite stage can be calculated by the simplified analysis method described in Appendix I.

Design distributed load due to “superimposed dead loads”:

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

Maximum design distributed load due to “live loads”:

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

Minimum design distributed load due to “live loads”:

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

Deflection due to “superimposed dead loads”:

$\delta_P = 4.0$ [mm]

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

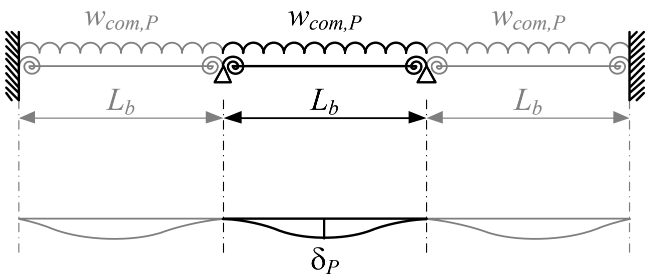
$\delta_V = 12.2$ [mm]

Deflection due to “dead loads and superimposed dead loads”:

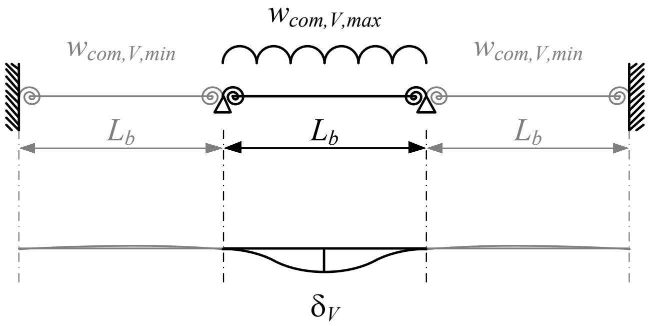
$\delta_{tP} = 43.5$ [mm]

Deflection due to “dead loads, superimposed dead loads, and live loads”:

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



(a) Superimposed dead loads



(b) Live loads (Load-case 1)

Figure AII.10: Design load with corresponding deflection

<u>Check on deflection of composite beam</u>

Limit of deflection due to “live loads”:

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

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

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

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

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

<u>Analysis of natural frequency of composite beam</u>

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 I.

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”:

$$\delta_{P+0.1V} = 10.1 \text{ [mm]}$$

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

$$f_{P+0.1V} = \frac{18}{\sqrt{\delta_{P+0.1V}}} = 5.7 \text{ [Hz]}$$

Check on vibration of composite beam

Required minimum natural frequency:

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

$$\therefore f_{P+0.1V} > f_{req} \quad \text{OK} \left(\frac{f_{req}}{f_{P+0.1V}} = 0.70\right)$$

Control of crack width of composite beam

Maximum diameter of longitudinal reinforcing bars:

$$\phi_{sl}^{\ *} = \phi_{sl,max}\left(\frac{2.9}{f_{ctm}}\right) = 14.5 \text{ [mm]}$$

Limit of stress permitted in longitudinal reinforcing bars immediately after cracking:

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

< Load-case 2 for maximizing hogging moment >

x-coordinate at inflection point:

$$x_0 = \frac{L_b}{2}\left(1\text{-}\sqrt{1\text{-}\frac{8M_{Eds}}{w_{com,max}L_b^{\ 2}}}\right) = 2764 \text{ [mm]}$$

Effective width on hogging moment region:

$$b_{effh} = b_{0h}+\min\left(\frac{2x_0}{4}\ ;\ B_b\text{-}b_{0h}\right) = 1482.1 \text{ [mm]}$$

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

$$A_{sl} = \pi\left(\frac{\phi_{sl,1}}{2}\right)^2 \left\lfloor\frac{b_{effh}}{p_{sl,1}}\right\rfloor+\pi\left(\frac{\phi_{sl,2}}{2}\right)^2 \min\left(\left\lfloor\frac{b_{effh}}{p_{sl,2}}\right\rfloor\ ;\ \left\lfloor\frac{b_{sl,2}}{p_{sl,2}}\right\rfloor\right) = 24.08 \text{ [cm}^2\text{]}$$

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

$$z_0 = \frac{(A_a+A_{sl})\left(0.5D_a+D_{ps}+0.5h_{cs}\right)}{A_a+A_{sl}+\left(\frac{h_{cs}b_{effh}}{n_0}\right)} = 185.0 \text{ [mm]}$$

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:

$$A_{cs} = b_{effh}h_{cs} = 1467.3 \text{ [cm}^2\text{]}$$

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

$$A_{sl,req} = \frac{0.72 k_c f_{ctm} A_{cs}}{\sigma_{sl,lim}} = 11.44 \text{ [cm}^2\text{]}$$

$$\therefore\ A_{sl} > A_{sl,req} \quad \text{OK} \left(\frac{A_{sl,req}}{A_{sl}} = 0.48\right)$$

Equivalent vertical distance between longitudinal reinforcing bars and neutral axis:

$$z_{sl,eq\text{-}na} = \frac{A_{sl} z_{tcs\text{-}sl,eq} + A_a \left(h_{cs} + D_{ps} + 0.5 D_a\right)}{A_{sl} + A_a} - z_{tcs\text{-}sl,eq} = 398.3 \text{ [mm]}$$

Second moment of area on hogging moment region:

$$I_h = I_{ay} + \frac{A_a A_{sl} \left\{D_a + 2\left(D_{ps} + h_{cs} - z_{tcs\text{-}sl,eq}\right)\right\}^2}{4\left(A_a + A_{sl}\right)} = 145711 \text{ [cm}^4\text{]}$$

Stress in longitudinal reinforcing bars caused by M_{Edh}:

$$\sigma_{sl,0} = \frac{M_{Edh}}{I_h} z_{sl,eq\text{-}na} = 111 \text{ [N/mm}^2\text{]}$$

Correction of stress in longitudinal reinforcing bars for tension stiffening:

$$\Delta\sigma_{sl} = \frac{0.4 f_{ctm}}{\frac{(A_a + A_{sl}) I_h}{A_a I_{ay}} \left(\frac{A_{sl}}{A_{cs}}\right)} z_{csl\text{-}na} = 37 \text{ [N/mm}^2\text{]}$$

Tensile stress in longitudinal reinforcing bars due to direct loading:

$$\sigma_{sl} = \sigma_{sl,0} + \Delta\sigma_{sl} = 148 \text{ [N/mm}^2\text{]}$$

$$\therefore\ \sigma_{sl} < \sigma_{sl,lim} \quad \text{OK} \left(\frac{\sigma_{sl}}{\sigma_{sl,lim}} = 0.61\right)$$

Appendix III Comparison of Pinned Joint and Semi-rigid Joint

This appendix presents the comparison of nominally pinned joints and semi-rigid joints in term of weight saving for the beam sizing. As introduced in the Forward, the significant advantage of using semi-rigid joints is lighter weight of steel beams compared to beams using pinned joints. Although only secondary beam ends can be semi-rigid according to the current design guide, lighter weight of secondary steel beam has a big impact on reduction of material cost and improvement of construction productivity because secondary beams often make up 20% to 30% of total steel tonnage.

The degree of steel weight reduction depends on the floor beam layout, especially the beam span. In general, the reduction ratio of steel weight 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 AIII.1 and Table AIII.2 show the comparison of pinned joints and semi-rigid joints on weight of steel beam under 12 m beam span and 15 m beam span, respectively. Other design conditions such as beam spacing and design loads are the same. In both comparisons, pinned joints are used for Case 1 and semi-rigid joints are used for Case 2, thus the advantages of semi-rigid joints for pinned joints can be evaluated. In addition, Case 3 in which the cross-section of Case 2 is changed from UB to JIS is also provided to clarify the advantage of using JIS cross- sections. Case 3 in Table AIII.2 corresponds to the design example in Appendix II.

Comparing Case 1 and Case 2 in both tables, it can be observed that semi-rigid joints can contribute to about 15% reduction in weight of steel beam. In other words, an economical long span beam layout can be achieved without increasing the beam weight, which may lead to the efficient use of floor space. Besides, it can be observed from the comparison of Case 2 and Case 3 that the combination of JIS cross-sections and semi-rigid joints enables a greater weight reduction because JIS has a larger number of I-sections with high cross-sectional efficiency due to narrow flange width and thinner web plate.

Table AIII.1: Comparison of pinned joints and semi-rigid joints on weight of steel beam under 12 m beam span

<table>
<tr><th colspan="2">Case</th><th>Case 1</th><th>Case 2</th><th>Case 3</th></tr>
<tr><td colspan="2">Beam span L_b</td><td colspan="3">12.0 m</td></tr>
<tr><td colspan="2">Beam spacing B_b</td><td colspan="3">3.0 m</td></tr>
<tr><td rowspan="2">Design load</td><td>SDL $g_{k,3}$</td><td colspan="3">3.0 kN/m^2</td></tr>
<tr><td>IL $q_{k,2}$</td><td colspan="3">5.0 kN/m^2</td></tr>
<tr><td colspan="2">Joint classification</td><td>Pinned</td><td>Semi-rigid</td><td>Semi-rigid</td></tr>
<tr><td rowspan="2">Steel beam</td><td>Cross-section</td><td>UB533x210x101</td><td>UB533x210x82</td><td>JIS550x200x9x12</td></tr>
<tr><td>Mass per metre</td><td>101.0 kg/m</td><td>82.2 kg/m
(▲18.6%)</td><td>76.0 kg/m
(▲24.8%)</td></tr>
</table>

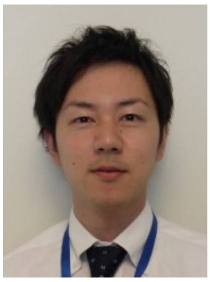
Table AIII.2: Comparison of pinned joints and semi-rigid joints on weight of steel beam under 15 m beam span

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

About the Authors

Dr J Y Richard Liew is a Professor and acting head of the Department of Civil & Environmental Engineering at the National University of Singapore. He is a Chartered Engineer in UK, a Professional Engineer in Singapore and a Chartered Professional Engineer of the Association of Southeast Asian Nations. He is a Fellow of the Academy of Engineering Singapore, an Honorary Fellow and the Past President of Singapore Structural Steel Society and Honorary Fellow of Hong Kong Institute of Steel Construction.

He has been in involved in research and practice in steel concrete composite structures covering a wide spectrum of interests, including light-weight and high strength materials for applications in offshore, marine, defense, buildings, and civil infrastructural works. Arising from this work, he has co-authored 8 books and generated more than 400 technical publications. He serves on the editorial boards of 8 international journals. He interacts with the industry in the Asia Pacific region serving as an expert and technical advisor and has been involved in numerous iconic projects. He chairs several international and national committees related to standards and specifications of steel and composite structures. He is a key person responsible for the development of Singapore's national annexes for the design of steel and composite structures using Eurocodes 3 and 4.



Mr. Yuichi Nishida is a Senior Manager in the Construction Products Development Div. at the Nippon Steel Corporation. He obtained his B.Eng. and M.Eng. degree from Kyushu University in Japan and is currently working as a structural engineer to develop new construction products and technologies. He joined the National University of Singapore taking advantage of the companys studying -abroad programme and obtained his M.Eng. degree with a research project on semi-rigid composite joints and beams.

He has been working in Prof. Liew's steel and concrete composite group on series of research projects of composite structures covering the design of innovative joints for long span structures.

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