1. input report

**Steel grade**
Steel grade S235

**Column parameters**
Section HE300A

**Beam parameters**
Section IPE270

**Verification parameters**
Welds at the connection point:
- Beam flange top: fillet weld, weld thickness a = 6.0 mm
- Beam web: fillet weld, weld thickness a = 4.0 mm
- Beam flange bottom: fillet weld, weld thickness a = 4.0 mm

**Internal forces and moments at the joint periphery referring to the system axes**
Lk 1: $M_{b,Ed} = 75.00 \text{ kNm}$  $V_{b,Ed} = 80.00 \text{ kN}$
$N_{c1,Ed} = 300.00 \text{ kN}$  $M_{c1,Ed} = 55.00 \text{ kNm}$ (bottom)

**Partial safety factors for material**
Resistance of cross-sections $\gamma_{M0} = 1.00$
Resistance of members in stability failure $\gamma_{M1} = 1.10$
Resistance of bolts, welds, plates in bearing $\gamma_{M2} = 1.25$
check of data
ok

notes
there are several basic components selected which perhaps do not ensure the total loading capacity of the joint.
no verification for cross-sections.
no verification for column web area.
no verification for welds within the connection.

2. Lk 1

2.1. design values

periphery connection ⊥ zur connection plane partial internal forces and moments

slope angle: αB = αV = α = 0°
distance: e1 = 145.0 mm, e3 = 129.9 mm, e2 = 129.9 mm

internal forces and moments perpendicular to the connection planes

periphery beam
M_d = 75.00 kNm, V_d = 80.00 kN
periphery column (bottom)
N_c = 300.00 kN, M_c = 55.00 kNm

calculation of internal forces and moments at periphery column (top)
N_c = N_e - V_d = 220.00 kN
M_c = M_e + V_eo e3 - M_d - V_d e1 - N_e (e3 - e5) = -31.60 kNm

partial internal forces and moments
N_b,t = -N_d z/zb + M_d/zb = 288.68 kN, z_b = 259.8 mm, zbw = 129.9 mm
N_b,c = N_d z/zb + M_d/zb = 288.68 kN, z_b = 259.8 mm, zbo = 129.9 mm
V_b,w = V_d = 80.00 kN

2.2. basic components

2.2.1. Gk 1: Column web panel in shear

transformation parameter (EC 3-1-8, 5.3(9)) \( \beta_1 = 1.00 \) for \( M_{11} = 75.00 \) kNm \( (M_{12} = 0) \)

Only the essential sizes are sketched to scale.
The connection geometry is only hinted.

slenderness of column web \( d_{de} / h = 24.47 < 69.00 \) ⇒ method applicable
plastic shear resistance \( V_{wp,rd} = (0.9 f_{yw} A_v) / (3^{1/2} \gamma_{MO}) = 455.2 \) kN

2.2.2. Gk 2: column web in transverse compression

transformation parameter (EC 3-1-8, 5.3(9)) \( \beta_1 = 1.00 \) for \( M_{11} = 75.00 \) kNm \( (M_{12} = 0) \)
longitudinal compressive stress in column web \( \sigma_{com,Ed} = 57.98 \) N/mm²
effective width of column web in transverse compression $b_{eff,c} = t_{b} + 2 \cdot 2^{1/2} \cdot a_{b} + 5 \cdot (t_{c} + s_{c}) = 226.5 \text{ mm}$

reduction factor $k_{w} = 1.0$ for $\alpha_{com, Ed} = 58.0 \text{ N/mm}^2 \leq 0.7 \cdot f_{y, w} = 164.5 \text{ N/mm}^2$

plate slenderness $\lambda_{p} = 0.932 \cdot \left( \frac{b_{eff,c} \cdot d_{w} \cdot f_{y}}{(E \cdot t_{w})^{1/2}} \right)$

reduction factor for web buckling $p = (\lambda_{p} - 0.2) / \lambda_{p}^2 = 0.941$

reduction factor for interaction with shear stress $\beta = 1 \Rightarrow \omega = 0.862$

resistance of an unstiffened web in transverse compression:

$$F_{c,w, Rd} = \omega \cdot (k_{w} \cdot b_{eff,c} \cdot t_{w} \cdot f_{y, w}) / \gamma_{M0} = 389.88 \text{ kN}$$

$$F_{c,w, Rd} = \omega \cdot (k_{w} \cdot p \cdot b_{eff,c} \cdot t_{w} \cdot f_{y, w}) / \gamma_{M1} = 333.35 \text{ kN (decisive)}$$

resistance of upper beam flange:

effective width of column web in transverse compression $b_{eff,c} = t_{b} + 2 \cdot 2^{1/2} \cdot a_{b} + 5 \cdot (t_{c} + s_{c}) = 232.2 \text{ mm}$

reduction factor $k_{w} = 1.0$ for $\alpha_{com, Ed} = 58.0 \text{ N/mm}^2 \leq 0.7 \cdot f_{y, w} = 164.5 \text{ N/mm}^2$

plate slenderness $\lambda_{p} = 0.932 \cdot \left( \frac{b_{eff,c} \cdot d_{w} \cdot f_{y}}{(E \cdot t_{w})^{1/2}} \right)$

reduction factor for web buckling $p = (\lambda_{p} - 0.2) / \lambda_{p}^2 = 0.933$

reduction factor for interaction with shear stress $\beta = 1 \Rightarrow \omega = 0.856$

resistance of an unstiffened web in transverse compression:

$$F_{c,w, Rd} = \omega \cdot (k_{w} \cdot b_{eff,c} \cdot t_{w} \cdot f_{y, w}) / \gamma_{M0} = 397.04 \text{ kN}$$

$$F_{c,w, Rd} = \omega \cdot (k_{w} \cdot p \cdot b_{eff,c} \cdot t_{w} \cdot f_{y, w}) / \gamma_{M1} = 336.69 \text{ kN (decisive)}$$

2.2.3. Gk 4: column flange in bending

effective width of unstiffened connections to flanges $b_{eff} = t_{w} + 2 \cdot 2^{1/2} \cdot a_{b} + 7 \cdot k \cdot t_{f} = 160.5 \text{ mm}$, $s = 27.0 \text{ mm}$, $k = 1.00$

$b_{eff} > b_{p} \Rightarrow b_{eff} = b_{p} = 135.0 \text{ mm}$

resistance of column flange in bending

$$F_{f,c, Rd} = (b_{eff,b,c} \cdot f_{c} \cdot f_{b}) / \gamma_{M0} = 323.6 \text{ kN}$$

2.2.4. Gk 3: column web in transverse tension

transformation parameter (EC 3-1-8, 5.3(9)) $\beta_{l} = 1.00$ for $M_{1} = 75.00 \text{ kNm}$ ($M_{2} = 0$)

Only the essential sizes are sketched to scale.
The connection geometry is only hinted.

effective width of column web in transverse tension $b_{eff,t} = t_{b} + 2 \cdot 2^{1/2} \cdot a_{b} + 5 \cdot (t_{c} + s_{c}) = 232.2 \text{ mm}$

reduction factor for interaction with shear stress $\beta = 1 \Rightarrow \omega = 0.856$

resistance of a column web with transverse tension

$$F_{t,w, Rd} = \omega \cdot (b_{eff,t} \cdot w \cdot f_{y, t}) / \gamma_{M0} = 397.0 \text{ kN}$$

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2.2.5. Gk 7: beam flange and web in compression

flange bottom: section class for $\phi(c-t) = 4.82$: 1
web: section class for $\phi = 0.50$ and $\phi(c-t) = 33.27$: 1
section class of beam: 1

taking into account the moment-shear force-interaction $V_{Ed} = 80.0$ kN

stress due to bending with shear force: $V_{Ed} = 80.0$ kN $\leq 150.2$ kN $= V_{pl,Rd}^2$ $\Rightarrow$ no effect

resistance $M_{c,Rd} = M_{pl,Rd} = (W_{pl} f_y) / \gamma_M = 113.74$ kNm, $W_{pl} = 484.00$ cm$^3$

resistance of a flange (and web) with compression $F_{c,t,Rd} = M_{c,Rd} / (h - t) = 437.80$ kN

resistance of upper beam flange:

stress due to bending with shear force: $V_{Ed} = 80.0$ kN $\leq 150.2$ kN $= V_{pl,Rd}^2$ $\Rightarrow$ no effect

resistance $M_{c,Rd} = M_{pl,Rd} = (W_{pl} f_y) / \gamma_M = 113.74$ kNm, $W_{pl} = 484.00$ cm$^3$

resistance of a flange (and web) with compression $F_{c,t,Rd} = M_{c,Rd} / (h - t) = 437.80$ kN

2.3. shear resistance

shear resistance of column web $V_{wp,Rd} / \beta_j = 455.2$ kN

2.4. verifications

2.4.1. verification of the connection capacity with partial internal forces and moments

shear force in column web:
$V_{c,w,Ed} = M_{d,w} / z \cdot (V_{c1} - V_{c2}) / 2 = 292.99$ kN, $M_{d,w} = 76.1$ kNm, $z = 259.8$ mm

Gk 1: $F_{Rd} = V_{wp,Rd} / \beta_j = 455.2$ kN, $F_{Ed} = \sqrt{V_{c,w,Ed}} = 292.99$ kN
$F_{Ed} = 293.0$ kN $< F_{Rd} = 455.2$ kN $\Rightarrow$ $U = 0.644 < 1$ ok

Gk 2: $F_{Rd} = F_{c,w,Rd} = 333.4$ kN, $F_{Ed} = N_{bc} = 288.68$ kN
$F_{Ed} = 288.7$ kN $< F_{Rd} = 333.4$ kN $\Rightarrow$ $U = 0.866 < 1$ ok

Gk 4: $F_{Rd} = F_{c,t,Rd} = 323.6$ kN, $F_{Ed} = N_{bt} = 288.68$ kN
$F_{Ed} = 288.7$ kN $< F_{Rd} = 323.6$ kN $\Rightarrow$ $U = 0.892 < 1$ ok

Gk 3: $F_{Rd} = F_{c,t,Rd} = 397.0$ kN, $F_{Ed} = N_{bt} = 288.68$ kN
$F_{Ed} = 288.7$ kN $< F_{Rd} = 397.0$ kN $\Rightarrow$ $U = 0.727 < 1$ ok

Gk 7: flange: $F_{Rd} = F_{c,t,Rd} = 437.8$ kN, $F_{Ed} = N_{bc} = 288.68$ kN
$F_{Ed} = 288.7$ kN $< F_{Rd} = 437.8$ kN $\Rightarrow$ $U = 0.659 < 1$ ok

utilization partial internal forces and moments $U_{Gk} = 0.892 < 1$ ok

2.4.2. verification result

maximum utilization: max $U = 0.892 < 1$ ok

3. final result

maximum utilization: max $U = 0.892 < 1$ ok

verification succeeded