3.3. Design of Primary Beam

3.3.1. Structural system & model

- **Structural system**
  - Transient design situation (construction stage): simple beam in and out of bending plane, with span $L=b$;
  - Persistent design situation (operational stage): simple beam in bending plane with uniform lateral supports at the top flange level. The flange in compression is laterally restrained by RC slab connected to the beam by welded bolt studs – type Nelson.

- **Structural model**
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3.3.2. Loads

**a) Operational stage**

- **Characteristic permanent loads (Self weight)**
  - Reaction force from SB (S.W. covering + RC slab+steel sheeting+installations (uniformly distributed)+SBeam):
    $$2R_{g_s,SB} = 2 \sum g_{k,i} \frac{d}{2}, [kN]$$
    (or taken as reaction force from previous calculations for statical model of the slab - simple beam)
  - S.W. Primary Beam
    $$g_{s,5} = 0.75 \text{ kN/m}$$ - approximate value or this self-weight should be calculated on the base of assumed initial cross-section of primary beam and self-weight of the steel per $m^3$.

- **Characteristic variable loads (Imposed loads + partition walls)**
Reaction force from SB (uniformly distributed)

\[ 2R_{q_{i,SB}} = 2\sum k q_{k,i} \frac{d}{2}, [kN] \]

**b) Construction stage - casting of concrete**

- **Permanent loads (Self weight)**
  - s.w. steel sheeting:
  - s.w. Primary Beam:
  - s.w. Secondary Beam:
  - s.w. RC slab (fresh concrete):
    
    \[ g_{k,\text{fresh concrete}} = 26 \text{ kN/m}^3 \]
    
    \[ q_{k} = g_{k,\text{fresh concrete}} h_{\text{equiv. flat slab}} \]

- **Variable loads**
  - s.w. lumped fresh concrete
    Inside the working area 3 m x 3 m (or the span length if less) -10% of the self-weight of the concrete but not less than 0,75 kN/m² and not more than 1,5 kN/m². Position of lumped fresh concrete determines should be the most unfavorable for the calculated member.
    
    \[ q_{k,\text{concrete}} = g_{k,\text{fresh concrete}} = 26 \text{ kN/m}^3 \]
    
    - Outside the working area - 0,75 kN/m²

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**3.3.3. Load combinations**

**a) Operational stage**

- **ULS design combinations**
  
  \[ 1.35G_{k} + 1.5Q_{k} = G_{d} + Q_{d} \]

- **SLS design combinations**
  
  \[ 1.0G_{k} + 1.0Q_{k} = G_{k} + Q_{k} \]
b) **Construction stage**

- **ULS design combinations**
  \[ 1.35G_k + 1.5Q_k = G_d + Q_d \]

- **SLS design combinations**
  \[ 1.0G_k + 1.0Q_k = G_k + Q_k \]

3.3.4. **Statistical calculations of the internal forces**

a) **Operational stage**

b) **Construction stage**
3.3.5. Cross-section composition, according design requirements for Operational stage

Having in mind relatively long span and intense forces the Primery beam will be designed with built-up plane section. The built-up cross-section is assembled by welding of independent steel plates. During the design procedure two processes run simultaneously - cross-section determining and fulfilling of design requirements. Beam will be designed only as a member of gravity forces bearing system and presented approach leads to the most economical solution.

a) Assumed initial proportions of cross-section
In this project the assumed approach is applying of fully effective cross-section. In such case could be used section class 3, composed directly for the current for ULS and SLS requirements.

- Assumed section class – 3
  - minimum requirements for outstand parts in compression:
    \[ 10\varepsilon < \frac{c_f}{t_f} \leq 14\varepsilon \]
  - minimum requirements for two sides supported parts in compression:
    \[ 83\varepsilon < \frac{c_w}{t_w} \leq 124\varepsilon \]

- Recommended section dimensions and proportions
  \[ t_w = 5\div10\ mm \]
  \[ t_f \geq 1.5t_w \] - requirement for reducing of deflections of heat affected flange due to welding;
  \[ b_f = (0.3 \div 0.5)h_w \] - relates to preferred ratio between self weight of beam flanges and web

b) Determination of cross-section dimensions, corresponding to the initial proportions and ULS and SLS design requirements

- Determination of the optimum height - \( h_{opt} \)
This height meets requirement for minimum self weight of the steel beam with I shaped cross section and enough bending resistance. Formula corresponds to the assumed ratio between area of flanges and web. According to the researches the best ratio is \( \sum A_f = A_w \).

\[ h_{opt} = 1.05 \sqrt{\frac{\lambda_w W_{required,y}}{f_y \gamma M_0}} \]
\[ W_{required,y} = \frac{M_{max,y}}{f_y \gamma M_0} \]
\[ 83\varepsilon < \lambda_w \leq 124\varepsilon \]
\[ \lambda_w \approx 100 \div 120 \]

- Determination of the minimum height – \( h_{min} \)
This height meets simultaneously the ULS requirement for enough bending resistance and SLS requirement for enough stiffness.
\[ M_{\text{Ed},y,\text{max}} = \frac{p_d l^2}{8}; \quad W_{el,y} = \frac{I_y}{h^2} \]

\[ \sigma_d = \frac{M_{\text{Ed},y,\text{max}}}{W_{el,y}} \leq \frac{f_y}{\gamma_{M0}} \]

\[ w_{\text{max}} = \frac{5}{384} \frac{p_d l^4}{E I_y} \leq w_u \]

\[ \sigma_k = \frac{p_k f_y}{p_d \gamma_{M0}} \]

\[ \text{if } w_{\text{max}} = w_u (G + Q) = \frac{l}{350}, \text{then} \]

\[ h_{\text{min}} = \frac{5}{24} \frac{\sigma_k l^2}{E w_u} \]

- **Assumed cross section properties**
  - \[ h_{\text{min}} \leq h \approx h_{\text{opt}}, \text{ rounded to } 5 \text{mm}; \]
  - \[ t_w \geq \frac{h}{\lambda_w \epsilon} (5; 6; 7; 8; 9; 10) \text{mm} \]

- **Proving of assumed web dimensions by shear resistance design check**
  Let’s assume shear force is beared only by the web:

\[ \tau_{\text{max}} = \frac{V_{\text{Ed},z,\text{max}} S_y}{I_y t_w} \approx \frac{1.5 V_{\text{Ed},z}}{h_w t_w} \leq \frac{f_y}{\sqrt{3} \gamma_{M0}}, \text{ then} \]

\[ t_w \geq \frac{1.5 V_{\text{Ed},z}}{h_w f_y} \sqrt{3} \gamma_{M_k} \]

If slenderness of stiffened web (stiffened by transverse intermediate and end-post stiffeners) is beyond the limits below

\[ \frac{h_w}{t_w} \geq \frac{72 \epsilon}{\eta} \quad \text{– for unstiffened or} \]

\[ \frac{h_w}{t_w} \geq \frac{31 \epsilon \sqrt{k_i}}{\eta} \eta = 1.2 \text{ for } S235 \div S355 \]

\[ \eta = 1.0 \text{ for } S460 \text{ and higher} \]

the web should be checked for resistance to shear buckling as well.

- **Shear resistance of cross-section**
  In difference with the shear resistance, shear buckling resistance should be proved for the maximum shear force at distance \( h_w/2 \) from the stiffener (transverse rib) in the considered web panel.

\[ \frac{V_{\text{Ed},z}}{V_{\text{bw,Rd}}} \leq 1 \]

\[ V'_{\text{bw,Rd}} = \frac{V_{bw,Rd} h_w f_{wy}}{\sqrt{3} \gamma_{M_k}} \leq \frac{\eta t_w h_w f_{wy}}{\sqrt{3} \gamma_{M_k}} \]
\( \chi_w = f\left( \lambda_w \right) \) – contribution of the web to shear buckling resistance

\( \lambda_w = \frac{h_w}{37.4 A_t \varepsilon \sqrt{k_w}} \) – for stiffened web

\( \lambda_w = \sqrt{\frac{A_t f_y / \sqrt{3}}{A_t \tau_{cr}}} \) – modified slenderness

\( \tau_{cr} = \sigma_E k_r \) – critical shear buckling stress

\( \sigma_E = \frac{\pi^2 E}{12\left(1-\nu^2\right)} \left( \frac{h_w}{t_w} \right)^2 \) – elastic critical plate buckling stress

\( k_r = 5.34 + \frac{4}{\alpha^2} \), for \( \alpha = \frac{a}{h_w} \geq 1 \)

\( k_r = 4.0 + \frac{5.34}{\alpha^2} \), for \( \alpha = \frac{a}{h_w} < 1 \)

\( \nu \) – Poisson’s ratio

\( \chi_w \) – depends on the stiffness of end post (end transversal rib)

see 5.2 and 5.3 - EN 1993-1-5

The given beam to column joint detail is flexible, so non-rigid end post.

Cross section notations

\[ \text{Figure 5.1: End supports} \]

5.3 Contribution from the web

(1) For webs with transverse stiffeners at supports only and for webs with either intermediate transverse stiffeners or longitudinal stiffeners or both, the factor \( \chi_w \) for the contribution of the web to the shear buckling resistance should be obtained from Table 5.1 or Figure 5.2.

| Table 5.1: Contribution from the web \( \chi_w \) to shear buckling resistance |
|-----------------|-----------------|-----------------|
| \( \lambda_w \) | Rigid end post | Non-rigid end post |
| \( \lambda_w < 0.83/\eta \) | \( \eta \) | \( \eta \) |
| \( 0.83/\eta \leq \lambda_w < 1.08 \) | \( \frac{0.83}{\lambda_w} \) | \( \frac{0.83}{\lambda_w} \) |
| \( \lambda_w \geq 1.08 \) | \( 1.37\left\{0.7 + \lambda_w \right\} \) | \( \frac{0.83}{\lambda_w} \) |

NOTE: See 6.2.6 in EN 1993-1-1.

In conclusion, accepted dimensions of the web are..............
• Calculation of the required flange area and the flanges dimensions

\[ A_{f, req} = \frac{W_{req}}{b_j} - \frac{t_j h_w}{6}, \]

\[ b_j \geq 180 \text{ mm} \quad \text{width should be enough for welding of bolted studs and bearing of corrugated steel sheets} \]

\[ b_f = (0.3 \times 0.5) h_w \]

\[ t_f \geq \frac{b_f - t_w}{2 \times 14\varepsilon} \]

In conclusion, accepted dimensions of the flange are.............

• Assumed cross section properties

...................

3.3.6. Final bending resistance

\[ I_y = \]

\[ W_y = \]

\[ \frac{M_{Ed,y}}{M_{pl,Rd,y}} \leq 1 \]

3.3.7. Design verification for bending-shear interaction, in case of significant bending moments and shear forces at the same beam cross section

If \( \frac{V_{Ed,z}}{V_{bw,Rd,z}} > 0.5 \), then interaction should be verified, but if

\[ \frac{M_{Ed,y}}{M_{f,Rd,y}} \leq 1, \text{where } M_{f,Rd,y} = \frac{A_f f_y}{\gamma_{M0}} h_0, \]

it means the flanges can bear whole bending moments independently and web—whole shear.

Otherwise combined design check:

\[ \eta_1 = \left( 1 - \frac{M_{f,Rd,y}}{M_{pl,Rd,y}} \right) \left( 2\eta_3 - 1 \right)^2 \leq 1 \]

\[ \eta_1 = \frac{M_{Ed,y}}{M_{pl,Rd,y}} \]

\[ M_{pl,Rd,y} = \frac{A_f f_y}{\gamma_{M0}} h_0 + \left( \frac{h_w}{2} \right)^2 \frac{t_w f_y}{\gamma_{M0}} \]

\[ \eta_3 = \frac{V_{Ed,z}}{V_{bw,Rd,z}} \]
3.3.8. Lateral torsional buckling resistance verification for Construction Stage

\[
\frac{M_{Ed,y,\text{max}}}{M_{p,Rd,y}} \leq 1
\]

\[
M_{b,Rd,y} = \chi_{LT} W_y \frac{f_y}{\gamma_{M_0}}
\]

where buckling length corresponds to the spacing between lateral supports. Initially, this length could be equal to the beam length, having in mind presence of vertical braced frames in the longitudinal direction between columns. If the design ratio isn’t fulfilled, the length could be reduced by applying of horizontal braced frames between secondary beams only for construction stage.