MŰEGYETEM 1782

Steel buildings BMEEOHSA-A1

Continuous steel-concrete composite girder

HOMEWORK 2

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Input data:

span of secondary girders: $E := 11.4\ m$

number of spans of secondary girders: $n := 3$

distance between secondary girders: $a := 3.3\ m$

thickness of concrete slab: $h_c := 75\ mm$

steel grade: S275

concrete strength class: C35/45

characteristic value of permanent load: $g_k := 2.4\ \dfrac{kN}{m^2}$

characteristic value of variable load: $q_k := 4\ \dfrac{kN}{m^2}$

Trapezodial sheets: LTP45 0.7mm

task 4: unpropped construction

1.2 Specifications

MSZ EN 1990: 2004. Basics of structural design.
MSZ EN 1991: 2002. Actions on structures
MSZ EN 1992-1-1: 2004. Design of concrete structures: General rules and rules for buildings.
MSZ EN 1993-1-1: 2005. Design of steel structures: General rules and rules for buildings.
MSZ EN 1994-1-1:2004. Design of composite steel and concrete structures: General rules and rules for buildings.

1.3 Material parameters

$E_a := 210000 \frac{N}{mm^2}$ elastic modulus of steel

$\nu_a := 0.3$ Poisson ratio of steel

$G_a := \frac{E_a}{2 \cdot (1+\nu_a)} = (8.077 \cdot 10^4) \frac{N}{mm^2}$ shear modulus of steel

$E_{cm} := 32000 \frac{N}{mm^2}$ elastic modulus of concrete for short-term loads

$E_{c.eff} := \frac{E_{cm}}{2} = (1.6 \cdot 10^3) \frac{kN}{cm^2}$ elastic modulus of concrete for short- and long-term loads (simplified method)

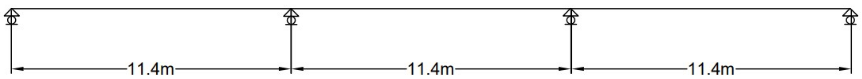
$n := \frac{E_a}{E_{c.eff}} = 13.125$ ratio of elastic moduli for short- and long-term loads

Structural steel : S275 $f_y := 27.5 \frac{kN}{cm^2}$ $\varepsilon := \sqrt[2]{\left(\frac{23.5}{f_y} \frac{kN}{cm^2}\right)} = 0.924$

Concrete: C35/45 $f_{ck} := 3.5 \frac{kN}{cm^2}$ $f_{cd} := \frac{f_{ck}}{1.5} = 2.333 \frac{kN}{cm^2}$

Rebars: B500B $f_{sk} := 50 \frac{kN}{cm^2}$ $f_{sd} := \frac{f_{sk}}{1.15} = 43.478 \frac{kN}{cm^2}$

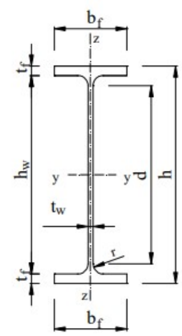
2. Design of secondary beams in construction phase
2.1. Statical system, cross-section, dimensions
Continuous girders



Span: $L := 11.4\ m$

Width of the load area: $a = 3.3\ m$

IPE 450 cross section:



$h := 450\ mm$ $\quad A_a := 98.8\ cm^2$

$b_f := 190\ mm$ $\quad A_v := 50.85\ cm^2$

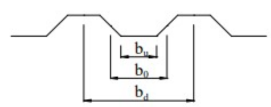
$t_f := 14.6\ mm$ $\quad W_{pl} := 1702\ cm^2$

$h_w := 420.8\ mm$ $\quad I_a := 33740\ cm^4$

$t_w := 9.4\ mm$ $\quad d := 378.8\ mm$

$r := 21\ mm$

LTP 45-0.7mm trapezoidal sheeting:

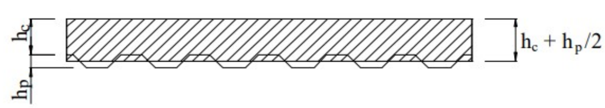


$h_p := 43\ mm$ $\quad b_u := 77\ mm$

$b_d := 180\ mm$ $\quad b_0 := 128\ mm$

Concrete slab:

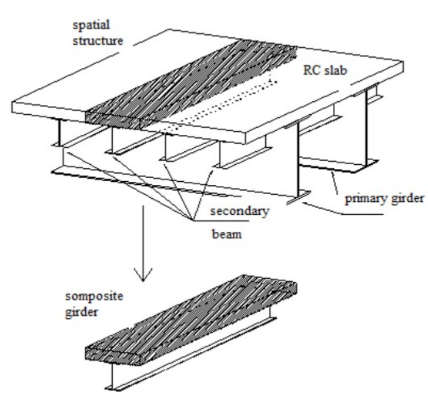
$h_c = 75\ mm$ $\quad \phi := 14\ mm$ $\quad s := 180\ mm$



$$h_s := h_c + \frac{h_p}{2} = 9.65\ cm$$

2.2. Loads in construction phase, partial factors

A composite girder will be treated as an individual structural element, so we have to transfer the surface loads to one girder. The width of the loading band of one girder is a 3.30m.



Permanent loads:

- concrete slab: $g_{vb} := 24 \frac{kN}{m^3}$ $\gamma_G := 1.35$

$$G_{vb} := g_{vb} \cdot a \cdot \left(h_c + \frac{h_p}{2}\right) = 7.643 \frac{kN}{m}$$

- trapezoidal sheeting: $g_{tr} := 0.069 \frac{kN}{m^2}$

$$G_{tr} := g_{tr} \cdot a = 0.228 \frac{kN}{m}$$

-steel secondary beam: $G_{ac} := 0.776 \frac{kN}{m}$

Extra load along the length of the beam:

$$G_e := g_k \cdot a = 7.92 \frac{kN}{m}$$

Total permanent load: $G_{tot} := G_{vb} + G_{tr} + G_{ac} + G_e = 16.567 \frac{kN}{m}$

Total variable load load the length of the beam: $Q := q_k \cdot a = 13.2 \frac{kN}{m}$

Partial factors for calculating the resistances:

$\gamma_{M0} := 1.0$ $\gamma_{M1} := 1.0$ $\gamma_{M2} := 1.25$

2.3. Checking in ultimate limit states
2.3.1. Cross-section classification
Based on tables: IPE 450 is Class 1 for bending.

Calculation of the steel beam - shear lag effect

For the side span:
Effective span: $L_{e.1} := 0.85 \cdot E = 9.69\ m$
Effective width without considering the spacing between the beams:

$b_{e.1} := \frac{L_{e.1}}{8} = 1.211\ m$

Effective width of the steel section: $b_{eff.1} := min\left(2 \cdot b_{e.1}, a\right) = 2.423\ m$

For the internal support region:
Effective span: $L_{e.2} := 0.25 \cdot 2 \cdot E = 5.7\ m$

Effective width without considering the spacing between the beams:

$$b_{e.2} := \frac{L_{e.2}}{8} = 0.713\ m$$

Effective width of the steel section: $b_{eff.2} := min\left(2 \cdot b_{e.2}, a\right) = 1.425\ m$

For the internal span:

Effective span: $L_{e.3} := 0.7 \cdot E = 7.98\ m$

Effective width without considering the spacing between the beams:

$$b_{e.3} := \frac{L_{e.3}}{8} = 0.998\ m$$

Effective width of the steel section: $b_{eff.3} := min\left(2 \cdot b_{e.3}, a\right) = 1.995\ m$

Distance between the top of the concrete slab and the centroid of the steel beam:

$$z_a := \frac{h}{2} + h_c + h_p = 34.3\ cm$$

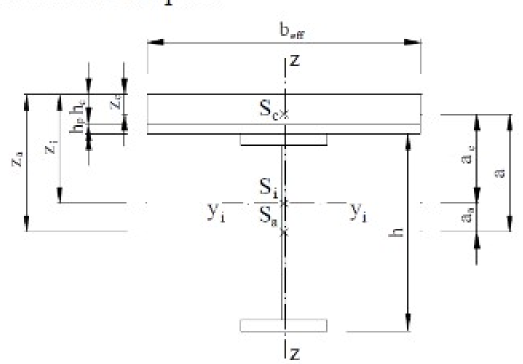
Distance between the top of the concrete slab and the centroid of the reinforcement:

$$z_s := 45\ mm = 4.5\ cm$$

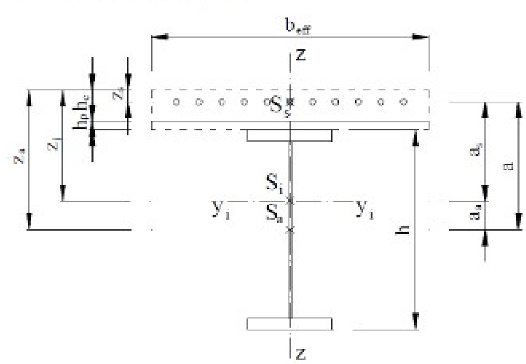
Distance between the top of the concrete slab and the centroid of the concrete slab:

$$z_c := \frac{h_s}{2} = 4.825\ cm$$

Notations in span:



Notations at support:



Calculation of the geometrical parameters for the side span

Cross-sectional area of the concrete slab:

$$A_{c.1} := h_s \cdot b_{eff.1} = 2337.713\ cm^2$$

Cross-sectional area of the steel section and concrete slab together:

$$A_{i.1} := \frac{A_{c.1}}{n} + A_a = 276.911\ cm^2$$

Distance between the top of the concrete slab and the centroid of the total section:

$$z_{i.1} := \frac{A_a \cdot z_a + \frac{A_{c.1}}{n} \cdot z_c}{A_{i.1}} = 15.341\ cm$$

Distance between the centroid of the total section and the centroid of the steel section:

$$a_{a.1} := z_a - z_{i.1} = 18.959\ cm$$

Distance between the centroid of the total section and the centroid of the concrete slab:

$$a_{c.1} := z_{i.1} - z_c = 10.516\ cm$$

Second moment of inertia of the concrete slab:

$$I_{c.1} := \frac{b_{eff.1} \cdot {h_s}^3}{12} = 18141.136\ cm^4$$

Second moment of inertia of the total section:

$$I_{i.1} := I_a + A_a \cdot {a_{a.1}}^2 + \frac{I_{c.1}}{n} + \frac{A_{c.1}}{n} \cdot {a_{c.1}}^2 = 90331.893\ cm^4$$

Calculation of the geometrical parameters for the internal support region:

Cross-sectional area of the reinforcement:

$$A_s := \frac{\pi}{4} \cdot \phi^2 \cdot \frac{b_{eff.2}}{s} = 12.187\ cm^2$$

Cross-sectional area of the steel section and the reinforcement together:

$$A_{i.2} := A_s + A_a = 110.987\ cm^2$$

Distance between the top of the concrete slab and the centroid of the total section:

$$z_{i.2} := \frac{A_a \cdot z_a + A_s \cdot z_s}{A_{i.2}} = 31.028\ cm$$

Distance between the centroid of the total section and the centroid of the steel section:

$$a_{a.2} := z_a - z_{i.2} = 3.272\ cm$$

Distance between the centroid of the total section and the centroid of the reinforcement:

$$a_{s.2} := z_{i.2} - z_s = 26.528\ cm$$

Second moment of inertia of the total section:

$$I_{i.2} := I_a + A_a \cdot {a_{a.2}}^2 + A_s \cdot {a_{s.2}}^2 = 43373.999\ cm^4$$

Calculation of the geometrical parameters for the internal span:

Distance between the centroid of the total section and the centroid of the concrete slab:

$$A_{c.3} := h_s \cdot b_{eff.3} = 1925.175\ cm^2$$

Cross-sectional area of the steel section and the steel section together:

$$A_{i.3} := \frac{A_{c.3}}{n} + A_a = 245.48\ cm^2$$

Distance between the top of the concrete slab and the centroid of the total section:

$$z_{i.3} := \frac{A_a \cdot z_a + \frac{A_{c.3}}{n} \cdot z_c}{A_{i.3}} = 16.688\ cm$$

Distance between the centroid of the total section and the centroid of the steel section:

$$a_{a.3} := z_a - z_{i.3} = 17.612\ cm$$

Distance between the centroid of the total section and the centroid of the concrete slab:

$$a_{c.3} := z_{i.3} - z_c = 11.863\ cm$$

Second moment of inertia of the concrete slab:

$$I_{c.3} := \frac{b_{eff.3} \cdot {h_s}^3}{12} = 14939.759\ cm^4$$

Second moment of inertia of the total section:

$$I_{i.3} := I_a + A_a \cdot {a_{a.3}}^2 + \frac{I_{c.3}}{n} + \frac{A_{c.3}}{n} \cdot {a_{c.3}}^2 = 86166.692\ cm^4$$

Internal forces on the girders:

Remark- AxisVM software is used to calculate internal forces. In the end of the homework, the documentation is included .

The results are:

For the side span:

Maximum sagging moment: $M_{Ed.positive.1} := 491.396\ kN \cdot m$

Maximum hogging moment: $M_{Ed.negative.1} := 171.137\ kN \cdot m$

Maximum shear force: $V_{Ed.1} := 220.091\ kN$

For the internal support region:

Maximum sagging moment: $M_{Ed.positive.2} := 59.280\ kN \cdot m$

Maximum hogging moment: $M_{Ed.negative.2} := 590.922\ kN \cdot m$

Maximum shear force: $V_{Ed.2} := 292.204\ kN$ (considered for being the maximum in the girder)

For the internal span:
Maximum sagging moment: $M_{Ed.positive.3} := 265.669\ kN \cdot m$
Maximum hogging moment: $M_{Ed.negative.3} := 244.766\ kN \cdot m$
Maximum shear force: $V_{Ed.3} := 187.066\ kN$

Resistance of the parts:

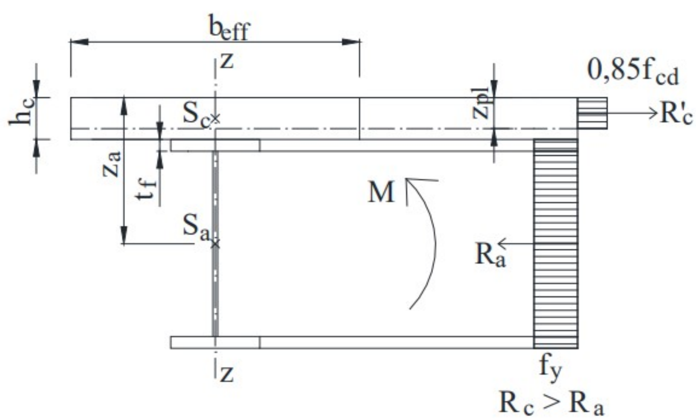
Resistance of the rebars: $R_s := A_s \cdot f_{sd} = 529.859\ kN$

Resistance of steel cross-section: $R_a := A_a \cdot f_y = 2717\ kN$

Resistance of each flange: $R_f := b_f \cdot t_f \cdot f_y = 762.85\ kN$

Resistance of the web with d height: $R_g := d \cdot t_w \cdot f_y = 979.198\ kN$

Resistance of concrete slab: $R_c := 0.85 \cdot b_{eff.1} \cdot h_c \cdot f_{cd} = 3603.469\ kN$
Resistance of the whole web (including the fillets)
$R_w := R_a - 2 \cdot R_f = 1191.3\ kN$



Moment resistance of side span region:

Resistance of the concrete slab $R_{c1} := 0.85 \cdot A_{c.1} \cdot f_{cd} = 4636.463\ kN$

$R_a = 2717\ kN$
$R_w = 1191.3\ kN$

$Plastic\_neutral\_axis :=$ if $(R_{c1} > R_a$, "is in the concrete", "is in the steel"$)$ = "is in the concrete"

$Plastic\_neutral\_axis := \text{if}\left(R_{c1} > R_w, \text{“is in the upper flange”}, \text{“is in the web”}\right) = \text{“is in the upper flange”}$

In the side span regions: $R_c > R_a,\ R_c > R_w$: The neutral axis is in the concrete.

$$z_{pl.a} := \frac{R_a}{b_{eff.1} \cdot 0.85 \cdot f_{cd}} = 5.655\ cm$$

Moment equilibrium:

$$M_{pl.Rd.1} := R_a \cdot \left(z_a - \frac{z_{pl.a}}{2}\right) = 855.108\ kN \cdot m$$

Calculation of the moment resistance in the internal support regions:

$Plastic\_neutral\_axis := \text{if}\left(R_s > R_a, \text{“is in the concrete”}, \text{“is in the steel”}\right) = \text{“is in the steel”}$
$Plastic\_neutral\_axis := \text{if}\left(R_s > R_w, \text{“is in the upper flange”}, \text{“is in the web”}\right) = \text{“is in the web”}$

$$\alpha := \frac{R_s + R_g}{2 \cdot R_g} = 0.771 \qquad \alpha = 0.771 \ > 0.5 \qquad d = 37.88\ cm$$

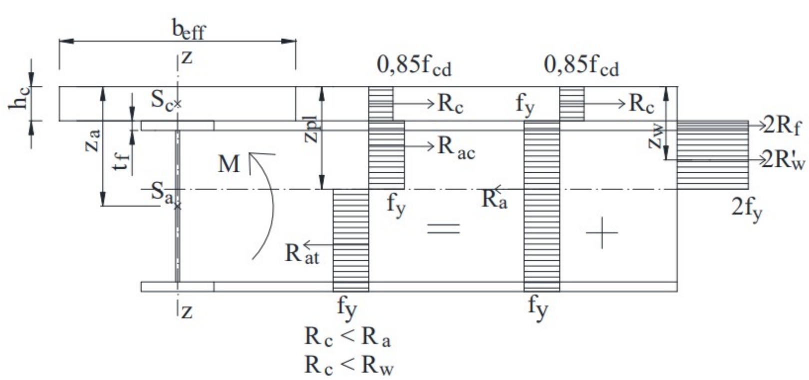
Height of the compressed zone: $\alpha \cdot d = 29.189\ cm$

$$Web := \text{if}\left(\frac{d}{t_w} < \frac{396 \cdot \varepsilon}{13 \cdot \alpha - 1}, \text{“is class 1”}, \text{“is not class 1”}\right) = \text{“is class 1”}$$

$$Web := \text{if}\left(\frac{d}{t_w} < \frac{456 \cdot \varepsilon}{13 \cdot \alpha - 1}, \text{“is class 2”}, \text{“is not class 2”}\right) = \text{“is class 2”}$$

$$Bottom\_flange := \text{if}\left(\frac{\frac{b_f}{2} - \frac{t_w}{2} - r}{t_f} < 9 \cdot \varepsilon, \text{“is class 1”}, \text{“is not class 1”}\right) = \text{“is class 1”}$$

Plastic moment resistance:
At internal supports: $R_s < R_a,\ R_s < R_w$: The neutral axis is in the web



$z_{pl.t} := 1\ cm$

$R_s + R_f + (z_{pl.t} - h_c - h_p - t_f) \cdot t_w \cdot f_y - (h + h_c + h_p - z_{pl.t} - t_f) \cdot t_w \cdot f_y - R_f = 0$

$z_{pl.t} := \mathbf{Find}\ (z_{pl.t}) = 24.051\ cm$

$R_{wc} := (z_{pl.t} - h_c - t_f) \cdot t_w \cdot f_y = 390.109\ kN$

$$M_{pl.Rd.2} := R_a \cdot \left(z_a - \frac{h_c}{2}\right) - 2 \cdot R_f \cdot \left(\frac{t_f}{2} + \frac{h_c}{2}\right) - \left(2 \cdot R_{wc} \cdot \left(\frac{z_{pl.t} - h_c - t_f}{2} + t_f + \frac{h_c}{2}\right)\right) = 662.17\ kN \cdot m$$

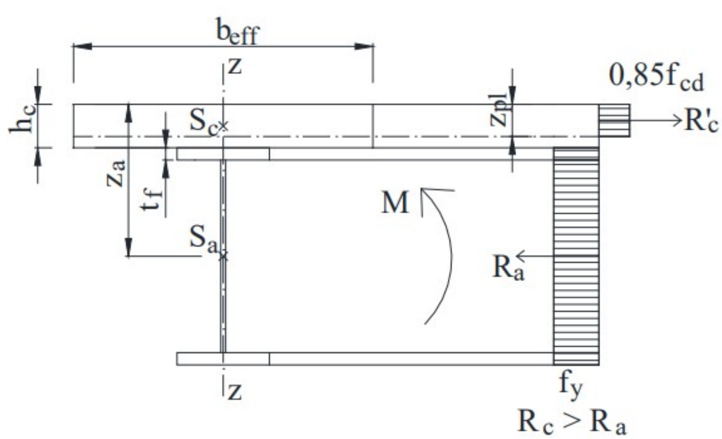
Calculation of the moment resistance in the internal span region:

Resistance of concrete slab $R_{c3} := 0.85 \cdot A_{c.3} \cdot f_{cd} = (3.818 \cdot 10^3)\ kN$

$R_a = (2.717 \cdot 10^3)\ kN$
$R_w = (1.191 \cdot 10^3)\ kN$

$Plastic\_neutral\_axis := \mathbf{if}\ (R_{c3} > R_a, \text{"is in the concrete"}, \text{"is in the steel"}) = \text{"is in the concrete"}$
$Plastic\_neutral\_axis := \mathbf{if}\ (R_{c3} > R_w, \text{"is in the upper flange"}, \text{"is in the web"}) = \text{"is in the upper flange"}$

In the internal spans: $R_c > R_a,\ R_c > R_w$: The neutral axis is in the "upper flange"



$z_{pl.1} := 1\ cm$

$R_a - z_{pl.1} \cdot b_{eff.1} \cdot 0.85 \cdot f_{cd} = 0$

$z_{pl.1} := \mathbf{find}\left(\mathbf{z_{pl.1}}\right) = 5.655\ cm$

$M_{pl.Rd.3} := R_a \cdot \left(z_a - \frac{z_{pl.1}}{2}\right) = 855.108\ kN \cdot m$

Calculation of the plastic shear resistance:

Partial factor for the resistance of the steel cross-sections independent of the class is: $\gamma_{M0} := 1.0$

The plastic shear resistance $V_{pl.Rd} := \frac{A_v \cdot f_y}{\sqrt{3} \cdot \gamma_{M0}} = 807.352\ kN$

Cross sectional resistances of the steel girder check:

Checking the bending resistance in the side span regions:
$M_{Ed} := M_{Ed.positive.1} = 491.396\ kN \cdot m$ $M_{pl.Rd.1} := 1113.156\ kN \cdot m$

$Bending\_resistance\_is := \text{if}\left(M_{pl.Rd.1} \ge M_{Ed}, \text{"adequate!"}, \text{"inadequate!"}\right) = \text{"adequate!"}$

Utilisation ratio= $\frac{M_{Ed}}{M_{pl.Rd.1}} = 44.14\%$

Checking the bending resistance in the internal support regions:
$M_{Ed} := M_{Ed.negative.2} = 590.922\ kN \cdot m$ $M_{pl.Rd.2} := 754.828\ kN \cdot m$

$Bending\_resistance\_is := \text{if}\left(M_{pl.Rd.2} \ge M_{Ed}, \text{"adequate!"}, \text{"inadequate!"}\right) = \text{"adequate!"}$

Utilisation ratio= $\frac{M_{Ed}}{M_{pl.Rd.2}} = 78.29\%$

Checking the bending resistance in the internal span regions:
$M_{Ed} := M_{Ed.positive.3} = 265.669\ kN \cdot m$ $M_{pl.Rd.3} := 1054.491\ kN \cdot m$

$Bending\_resistance\_is := \text{if}\left(M_{pl.Rd.3} \ge M_{Ed}, \text{"adequate!"}, \text{"inadequate!"}\right) = \text{"adequate!"}$

Utilisation ratio= $\frac{M_{Ed}}{M_{pl.Rd.3}} = 25.19\%$

Checking the shear resistance of the beam:

$V_{Rd} := V_{pl.Rd} = 807.352\ kN$ $\qquad$ $V_{Ed} := V_{Ed.2} = 292.204\ kN$

$Shear\_resistance\_is := \text{if}\left(V_{Rd} \geq V_{Ed}, \text{“adequate!”}, \text{“inadequate!”}\right) = \text{“adequate!”}$

Utilisation ratio= $\frac{V_{Ed}}{V_{Rd}} = 0.36$

Checking the Interaction of the Shear and the Bending Moment: -

$MV\_interaction := \text{if}\left(0.5 \cdot V_{Rd} < V_{Ed}, \text{“is needed!”}, \text{“is not needed!”}\right) = \text{“is not needed!”}$

The bending moment is not reduced due to the shear acing on the structural element because the maximum shear force is less than half of the shear resistance.

Checking the Girder for the Lateral Torsional Buckling (LTB): -

M_{cr} determination:

Length between the lateral supports of the lower flange: $L := E = 11.4\ m$

Modulus of elasticity of structural steel: $E_a = 210\ GPa$

Shear modulus of structural steel: $G_a = 80.769\ GPa$

Inertia of the lower flange about the z-z axis of the I-section:

$I_{afz} := \frac{{b_f}^3 \cdot t_f}{12} = 834.512\ cm^4$

Pure torsional moment of inertia:

$I_{at} := \frac{1}{3} \cdot \left(2 \cdot b_f \cdot {t_f}^3 + h_w \cdot {t_w}^3\right) = 51.071\ cm^4$

For inner beams in a floor with four or more similar beams: $\alpha := 4$

Spacing between the parallel beams: $a = 3.3\ m$

Unit length of the composite slab: $b := 1\ m$

Ratio of steel to concrete cross-sectional areas: $\rho := \frac{A_s}{h_s \cdot b} = 0.013$

Second order moment of inertia of unit length concrete block about its own centroid axis:

$$I_2 := \frac{{h_s}^3 \cdot b}{12} = 7488.601\ cm^4$$

Cracked flexural stiffness per unit width of the concrete or composite slab:

$$EI_2 := E_a \cdot I_2 \cdot 6.5 \cdot \rho = \left(1.291 \cdot 10^7\right)\ kN \cdot cm^2$$

Flexural stiffness of the cracked concrete or composite slab in the transversal direction of the steel beam:

$$k_1 := \frac{\alpha \cdot EI_2}{a \cdot b} = 1564.734\ kN$$

Distance between the gravity centres of the two flanges:

$$h_f := h - t_f = 43.54\ cm$$

Flexural stiffness of the steel web:

$$k_2 := \frac{E_a \cdot {t_w}^3}{4 \cdot \left(1 - {\nu_a}^2\right) \cdot h_f} = 110.056\ kN$$

Rotational stiffness per unit length of the steel beam:

$$k_s := \frac{k_1 \cdot k_2}{k_1 + k_2} = 102.824\ kN$$

Inertia of the ideal cross-sectional at the internal support about the centroid axis of the steel section:

$$I_y := I_a + A_s \cdot \left(a_{s.2} + a_{a.2}\right)^2 = 44562.332\ cm^4$$

Inertia of the steel section about the y-y axis: $I_{ay} := I_a = 33740\ cm^4$

Inertia of the steel section about the z-z axis: $I_{az} := 1675.86\ cm^4$

Cross-sectional area of the ideal cross-section over the internal support:

$$A := A_{i.2} = 110.987\ cm^2$$

Cross-sectional area of the steel section:

$$A_a = 98.8\ cm^2$$

$$i_x := \sqrt{\frac{I_{ay} + I_{az}}{A_a}} = 18.933\ cm$$

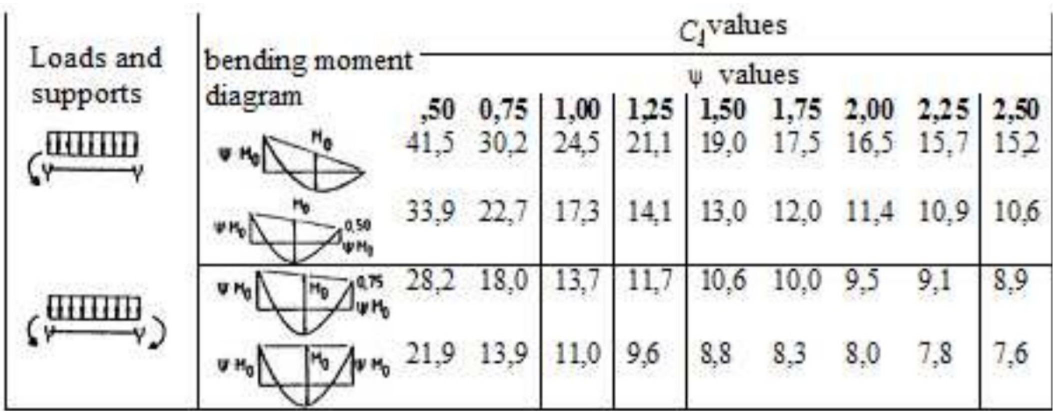
Distance between the centroid axis of the steel section and centroid axis of the concrete slab:

$$z_e := a_{a.1} + a_{c.1} = 29.475\ cm \qquad e := \frac{A \cdot I_{ay}}{A_a \cdot z_c \cdot (A - A_a)} = 644.575\ cm$$

For double symmetric steel section:

$$k_c := \frac{h_f \cdot \dfrac{I_y}{I_{ay}}}{\dfrac{\dfrac{h_f^{\ 2}}{4} + i_x^{\ 2}}{e} + h_f} = 1.283$$

$$M_0 := M_{small} + \frac{x_\eta}{E} \cdot (M_{big} - M_{small}) + |M_{middle}|$$



Loads and supports	bending moment diagram	C_4 values, ψ values: ,50	0,75	1,00	1,25	1,50	1,75	2,00	2,25	2,50
		41,5	30,2	24,5	21,1	19,0	17,5	16,5	15,7	15,2
	0,50 ψ M_0	33,9	22,7	17,3	14,1	13,0	12,0	11,4	10,9	10,6
	0,75 ψ M_0	28,2	18,0	13,7	11,7	10,6	10,0	9,5	9,1	8,9
	ψ M_0	21,9	13,9	11,0	9,6	8,8	8,3	8,0	7,8	7,6

load combination	M_big[kNm]	M_small[kNm]	M_middle[kNm]	x_n[m]	M0[kNm]	M_small/M_big	ψ	C4
1	462.262	94.662	-16.256	3.143	212.266	0.204779973	2.177749	15.93
2	419.375	70.002	-265.669	4	458.258	0.166919821	0.91515	26.43458
3	462.26	94.663	-16.256	4.807	265.9224	0.204783023	1.738327	17.57
4	590.922	59.28	-205.534	3.5	428.0374	0.100317808	1.380538	20
5	419.375	234.044	55.982	4	355.0544	0.558078092	1.181157	19.32468
6	590.921	59.281	-205.534	4.8	488.6634	0.10031967	1.20926	21.65406
7	548.035	198.662	-137.009	4	458.258	0.362498746	1.195909	21.8356
8	290.715	105.384	-72.678	4	243.0904	0.362499355	1.195913	21.8355

Table containing calculation of C4

$$C_4 := 15.93$$

$$M_{cr} := \frac{k_c \cdot C_4}{L} \cdot \sqrt{\left(G_a \cdot I_{at} + \frac{k_s \cdot L^2}{\pi^2}\right) \cdot E_a \cdot I_{afz}} = \left(2.803 \cdot 10^3\right)\ kN \cdot m$$

Calculation of resistance against lateral torsional buckling:

$$M_{Rk} := min\left(M_{pl.Rd.2}, M_{pl.Rd.3}\right) = 754.828\ kN \cdot m$$

$$\lambda_{LT} := \sqrt{\frac{M_{Rk}}{M_{cr}}} = 0.519 \qquad \alpha_{LT} := 0.34$$

$$\phi_{LT} := \frac{1 + \alpha_{LT} \cdot \left(\lambda_{LT} - 0.2\right) + {\lambda_{LT}}^2}{2} = 0.689$$

$$\chi_{LT} := \frac{1}{\phi_{LT} + \sqrt{{\phi_{LT}}^2 - {\lambda_{LT}}^2}} = 0.876$$

Resistance against LTB:

$M_{b.Rd} := \chi_{LT} \cdot M_{Rk} = 661.019\ kN \cdot m \qquad M_{Ed} := M_{Ed.negative.2} = 590.922\ kN \cdot m$

$LTB\_resistance := \text{if}\left(M_{b.Rd} \geq M_{Ed}, \text{“adequate!”}, \text{“inadequate!”}\right) = \text{“adequate!”}$

Utilisation ratio= $\dfrac{M_{Ed}}{M_{b.Rd}} = 89.4\%$

Design of the shear connectors:

Design resistance of the connectors

Diameter of shear connector: $d_{sc} := 20\ mm$

Length of shear connector: $h_{sc} := 95\ mm$ > $h_c + h_p = 118\ mm$ ok!

$\dfrac{h_{sc}}{d_{sc}} = 4.75$ > 4 $\qquad \alpha := 1.0 \qquad \gamma_V := 1.25$

$f_u := 430\ MPa$ (S275)

Design shear resistance from stud:

$$P_{Rd.1} := \frac{0.8 \cdot f_u \cdot \dfrac{{d_{sc}}^2 \cdot \pi}{4}}{\gamma_V} = 86.457\ kN$$

Design shear resistance from concrete:

$$P_{Rd.2} := \frac{0.29 \cdot \alpha \cdot {d_{sc}}^2 \cdot \sqrt{f_{ck} \cdot E_{cm}}}{\gamma_V} = 98.21\ kN$$

$b_0 = 128\ mm \qquad n_r := 2$

$$k'_t := \frac{0.7}{\sqrt{n_r}} \cdot \frac{b_0}{h_p} \cdot \left(\frac{h_{sc}}{h_p} - 1\right) = 1.782$$

$$k_t := \text{if}\left(k'_t > 0.7, 0.7, k'_t\right) = 0.7$$

Design shear resistance: $P_{Rd} := k_t \cdot min\left(P_{Rd.1}, P_{Rd.2}\right) = 60.52\ kN$

Check between the side support and the positive bending moment maximum

Design longitudinal shear force with full shear connection:

$$N_c := \frac{0.85 \cdot A_{c.1} \cdot f_{cd}}{\gamma_{M0}} = 4636.463\ kN$$

$$N_a := \frac{A_a \cdot f_y}{\gamma_{M0}} = 2717\ kN$$

$$V_l := min\left(N_a, N_c\right) = 2717\ kN$$

Number of studs: $n_{required} := \frac{V_l}{P_{Rd}} = 44.895$

$V_{Ed} = 292.204\ kN$ $x := 5.057\ m$

$$n_{wave} := \frac{x}{b_d} = 28.094 \quad < \quad n_{required}$$

Thus, three studs are needed in one wave of LTP45

Design longitudinal shear force with full shear connection

$N_c = \left(4.636 \cdot 10^3\right)\ kN$ $N_a = \left(2.717 \cdot 10^3\right)\ kN$

$$N_s := \frac{A_s \cdot f_{sd}}{\gamma_{M0}} = 529.859\ kN$$

$$V_l := min\left(N_a, N_c\right) + N_s = \left(3.247 \cdot 10^3\right)\ kN$$

Number of studs: $n_{required} := \frac{V_l}{P_{Rd}} = 53.65$

$L - x = 6.343\ m$

$$n_{wave} := \frac{L - x}{b_d} = 35.239 \quad < \quad n_{required}$$

Three studs are needed in one wave of LTP45

Transversal reinforcement:

Longitudinal shear force per unit length

$$v_{Ed} := \frac{P_{Rd}}{b_d} = 336.22 \ \frac{kN}{m}$$

Shear resistance per unit length:

$\phi = 14 \ mm$ $\qquad\qquad$ $s_f := s = 180 \ mm$

$$A_{sf} := \frac{\pi}{4} \cdot \phi^2 = 1.539 \ cm^2$$

$$Transversal\_reinforcement := \textbf{if}\left(\frac{2 \cdot A_{sf} \cdot f_{sd}}{s_f} > v_{Ed}, \text{"adequate"}, \text{"inadequate"}\right) = \text{"adequate"}$$

Unpropped Construction Technology:

The structure is checked during construction and in the final stages. The dominant internal forces for girders are bending moment and shear force.

Composite beams are often designed under the assumption that the unpropped steel beam supports the weight of the structural steel and wet concrete plus construction loads. It may, therefore, be decided for reasons of economy to provide only sufficient connectors to develop enough composite action to support the loads applied afterwards. This approach results in many less connectors than are required to enable the maximum bending resistance of the composite beam to be reached. The use of such partial shear connection results in reduced resistance and stiffness.

During the construction phase, the steel section holds its self-weight, the self-weight of the concrete slab and the construction loads.

The steel girder is checked for:

Ultimate limit state checks:

1- Cross-sectional failure types

2- Stability failure check:

Serviceability limit state check:

3- Check for the maximum deformation of the steel girder.

The composite slab is checked for the following:

Ultimate limit state checks:

1- Cross-sectional failure types

2- Stability failure check.

3- The design of shear connectors.

Serviceability Limit state checks:

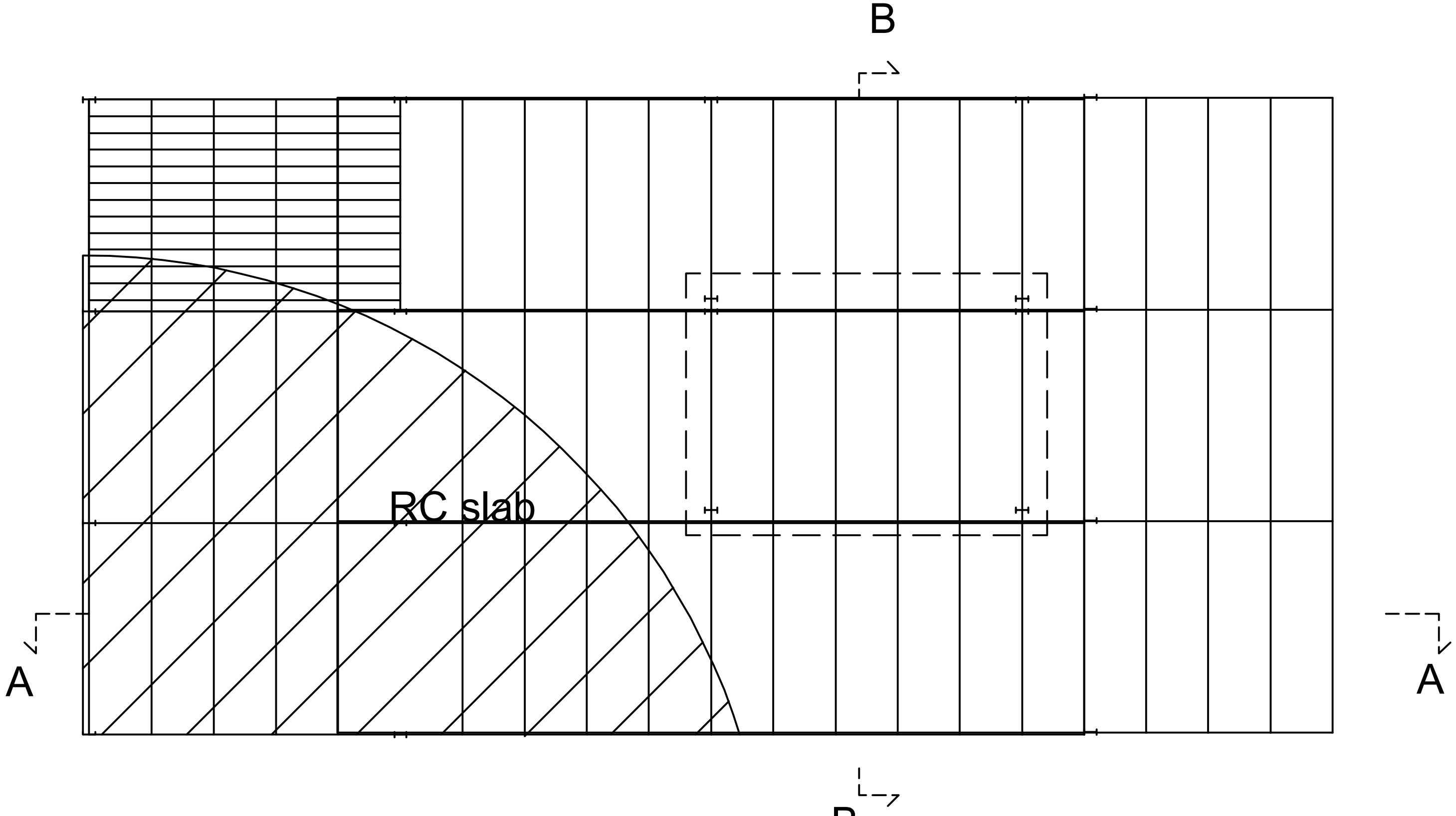
4- Check for the maximal crack width of the concrete slab.

5- Check for the minimal reinforcement of the concrete slab.

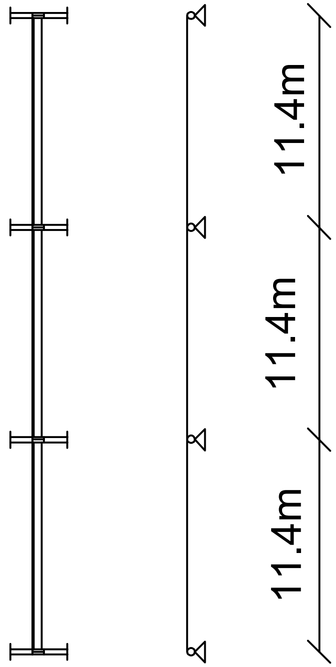
6- Check for the deformation of the composite slab.

In the construction phase, slab should be concreted as whole to ensure bonding because of the column is made from steel separately.

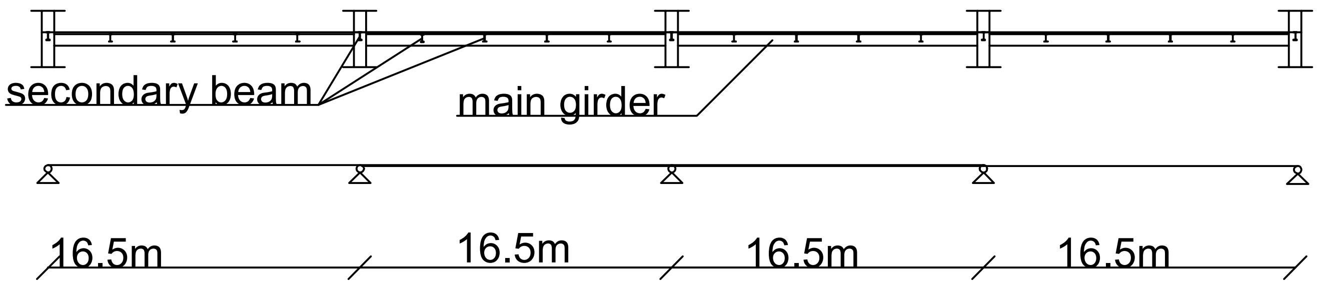
Top view



B-B section

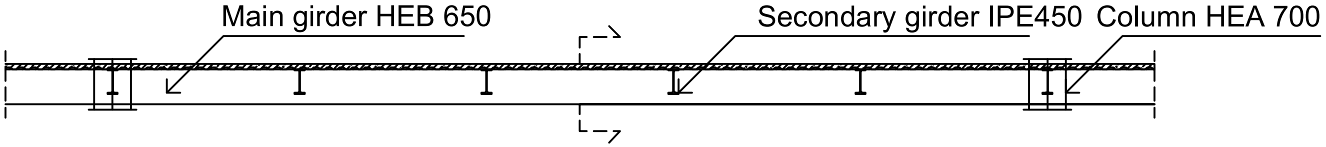


A-A section

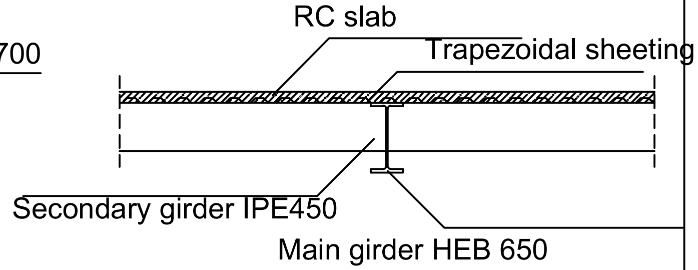


BME Department of Structural engineering	
Subject: Steel buildings Composite slab	
Preliminary drawing	
Student: Amgalantuul P.	Supervisor: Dr Noémi Seres

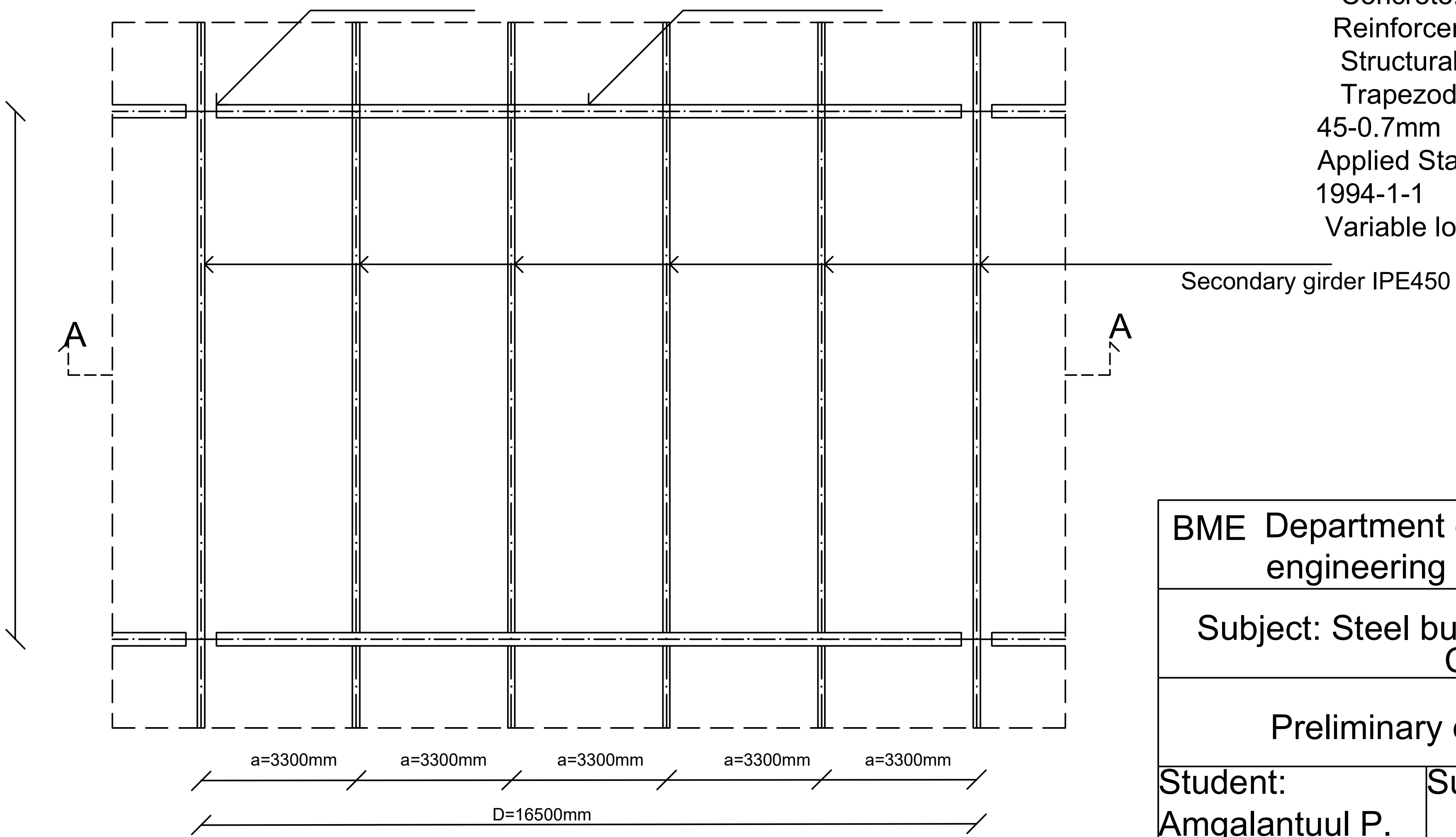
Section A-A S=1:100



Section A-A S=1:100



Top View, S=1:100



Material classes
Concrete: C35/45
Reinforcement : B500B
Structural steel: S275
Trapezodial sheeting : LTP 45-0.7mm
Applied Standard: MSZ-EN 1994-1-1
Variable load: 4kN/m^2

BME Department of Structural engineering	
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Project

Analysis by Amgalantuul Purevsuren

AxisVM X6 R1t · Registered to Amgalantuul Purevsuren

Model 3.axs

Report

Report, Table of contents

Materials

	Name	*Type*	*National design code*	*Material code*	*Model*	*E_x [N/mm²]*	*E_y [N/mm²]*	*ν*	*α_T [1/°C]*	*ρ [kg/m³]*
1	C35/45	Concrete	Eurocode-H	EN 206	Linear	34100	34100	0.20	1E-5	2500
2	S 275	Steel	Eurocode-H	10025-2	Linear	210000	210000	0.30	1.2E-5	7850

	Name	*Material color*	*Contour color*	*Texture*	*P_1*	*P_2*	*P_3*
1	C35/45			Concrete A	f_{ck}[N/mm²] = 35.00	γ_c= 1.500	α_{cc}= 1.00
2	S 275			Steel	f_y[N/mm²] = 275.00	f_u[N/mm²] = 430.00	f_y^*[N/mm²] = 255.00

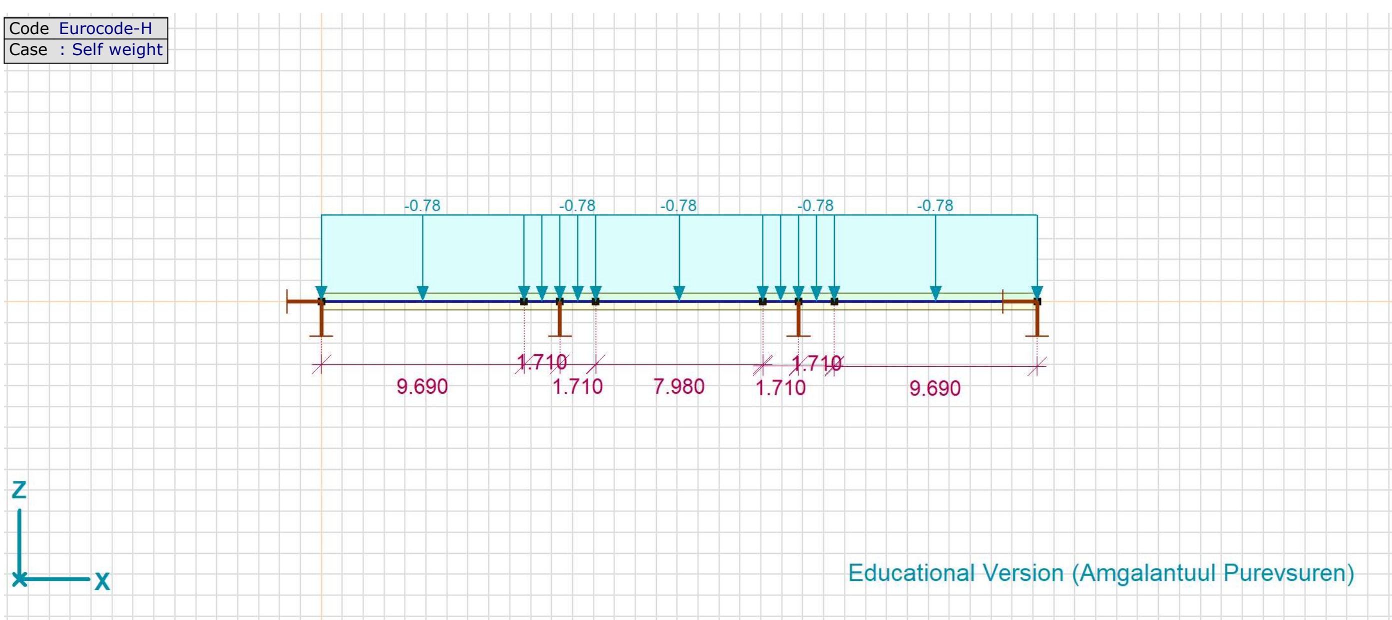
	Name	*P_4*	*P_5*	*P_6*	*P_7*	*P_8*	*P_9*	*P_{10}*	*P_{11}*	*P_{12}*	*P_{13}*	*P_{14}*
1	C35/45	ϕ_t= 2.00										
2	S 275	f_u^*[N/mm²] = 410.00										

Name: Material name; **Type:** Type of material; **Model:** Material model; **E_x:** Young's modulus of elasticity in local x direction; **E_y:** Young's modulus of elasticity in local y direction; **ν:** Poisson's ratio; **α_T:** Thermal expansion coefficient; **ρ:** Density; **Contour color:** Material outline color; **P_1, P_2, P_3, P_4, P_5, P_6, P_7, P_8, P_9, P_{10}, P_{11}, P_{12}, P_{13}, P_{14}:** Design parameter;

Custom load combinations by load cases

	Name	*Type*	*ST1*	*Self weight*	*Permanent load without steel*	*Variable 1*	*Variable 2*	*Variable 3*	*Comment*
1	Co #1	ULS	0	1.35	1.35	1.50	0	0	
2	Co #2	ULS	0	1.35	1.35	0	1.50	0	
3	Co #3	ULS	0	1.35	1.35	0	0	1.50	
4	Co #4	ULS	0	1.35	1.35	1.50	1.50	0	
5	Co #5	ULS	0	1.35	1.35	1.50	0	1.50	
6	Co #6	ULS	0	1.35	1.35	0	1.50	1.50	
7	Co #7	ULS	0	1.35	1.35	1.50	1.50	1.50	
8	Co #8	ULS	0	1.35	1.35	0	0	0	

Name: Load combination name; **Type:** Load combination type; **ST1, Self weight, Permanent load without steel, Variable 1, Variable 2, Variable 3:** Factor;

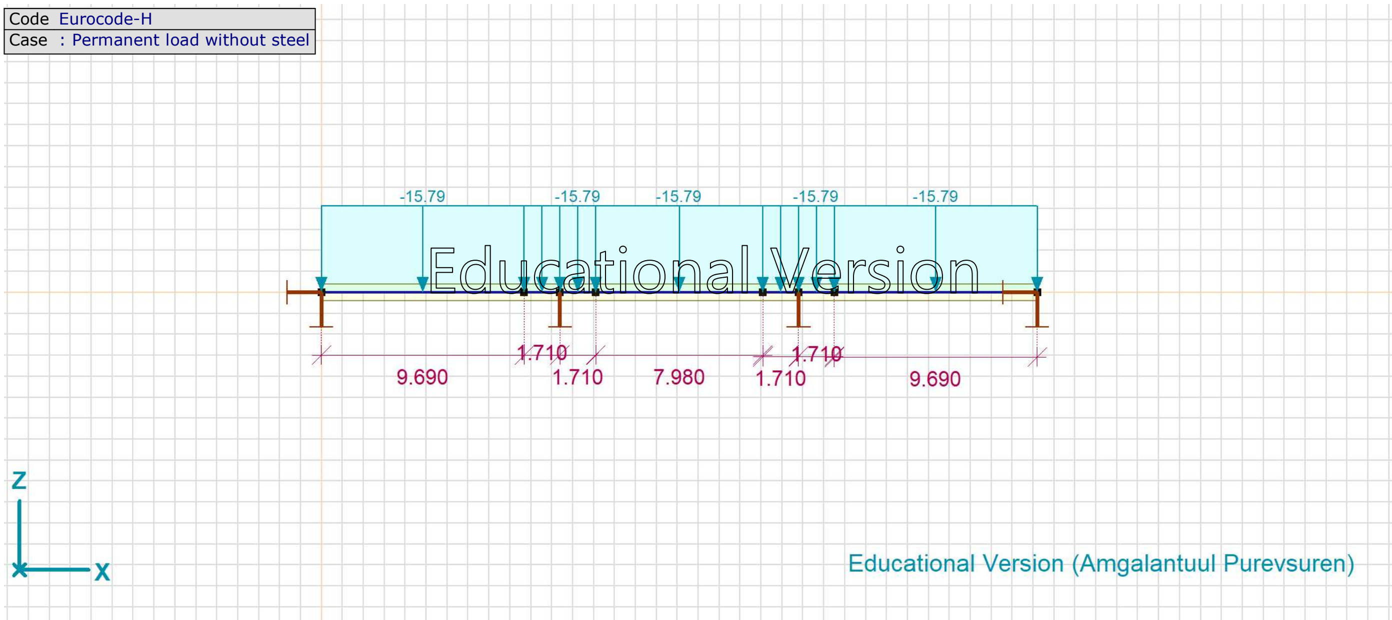


Self weight

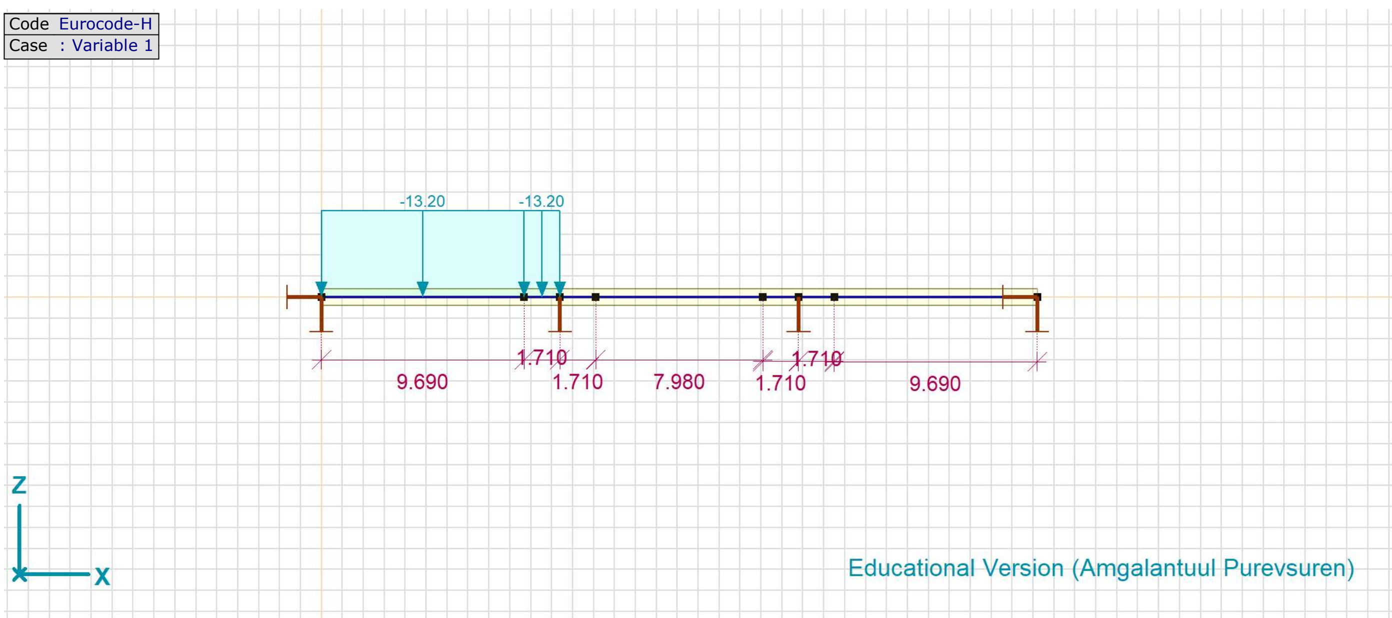
Project

Analysis by Amgalantuul Purevsuren

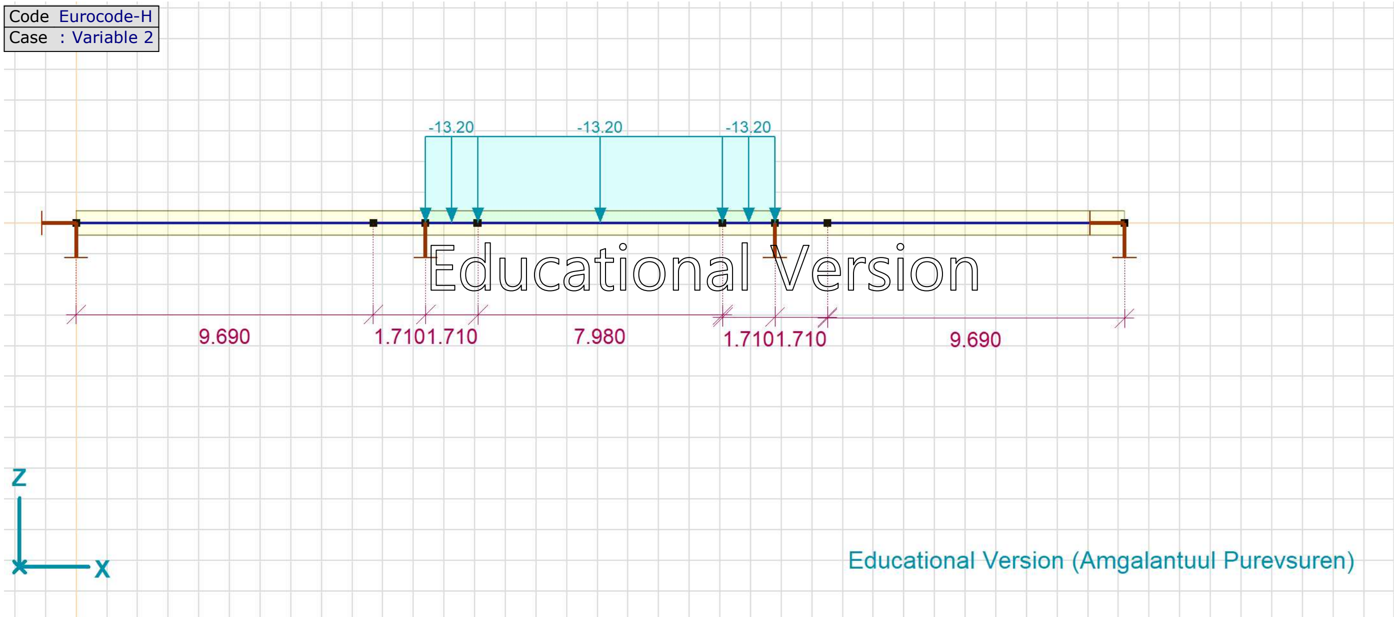
Model: **Model 3.axs** 4/24/2022



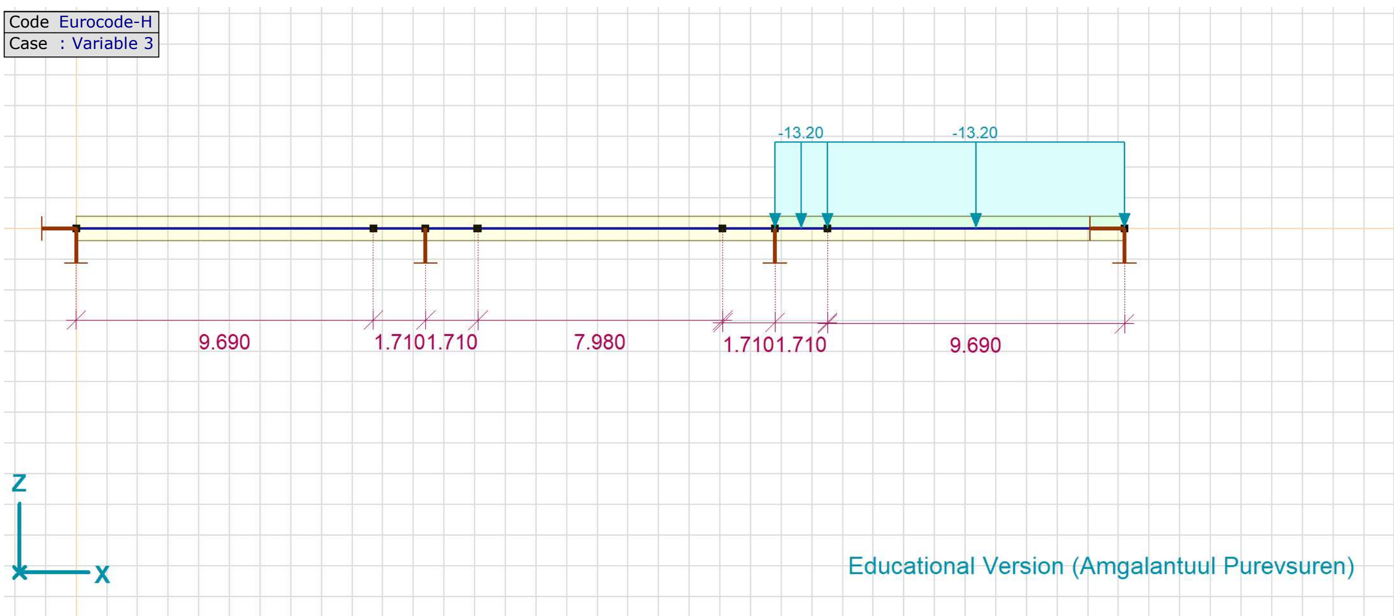
Permanent load without steel



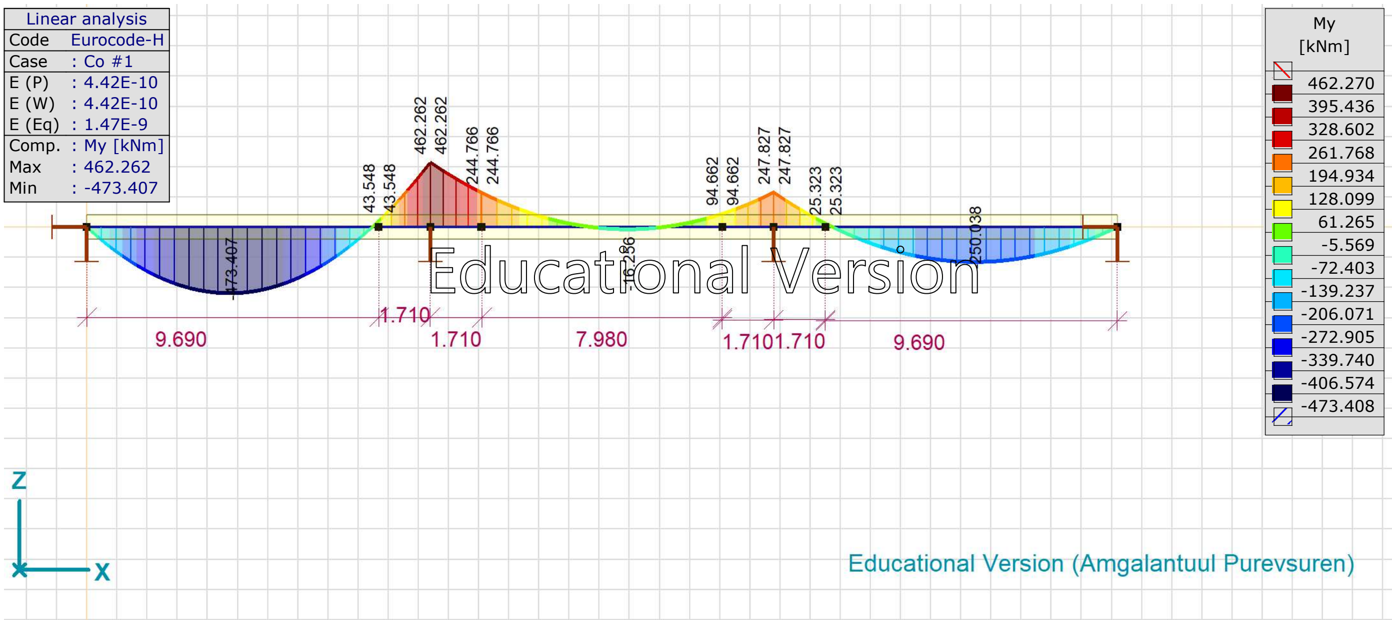
Variable 1



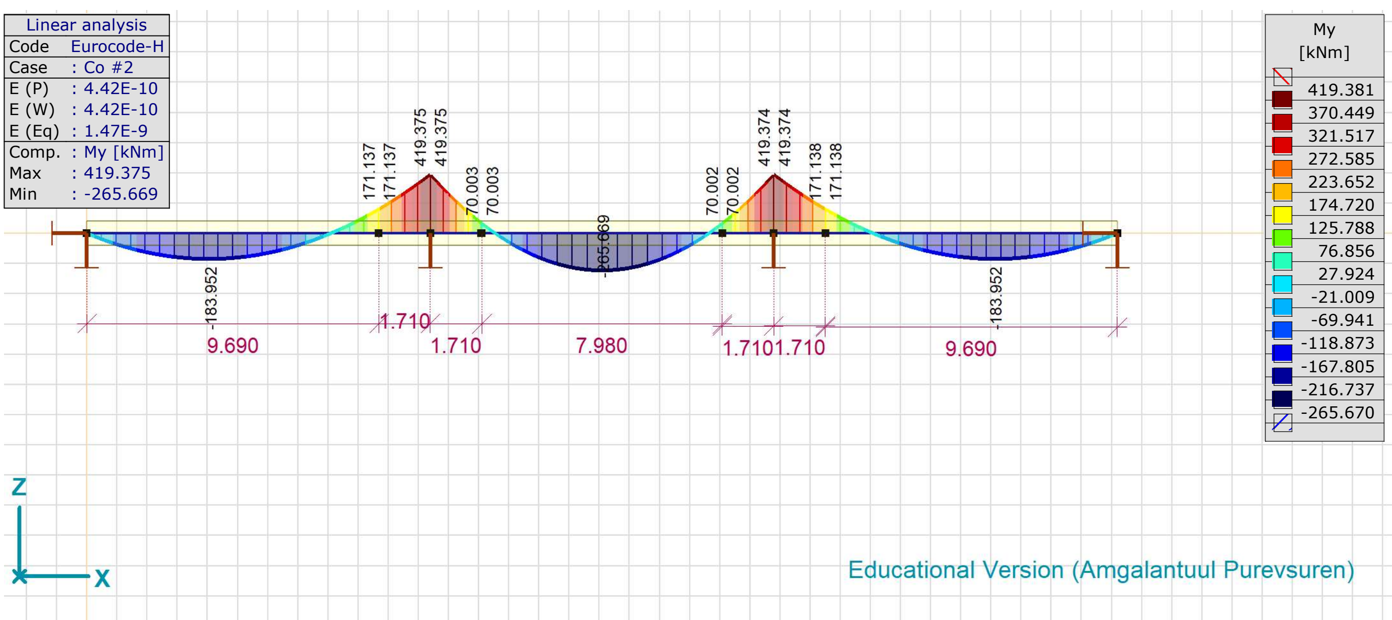
Variable 2, Front view



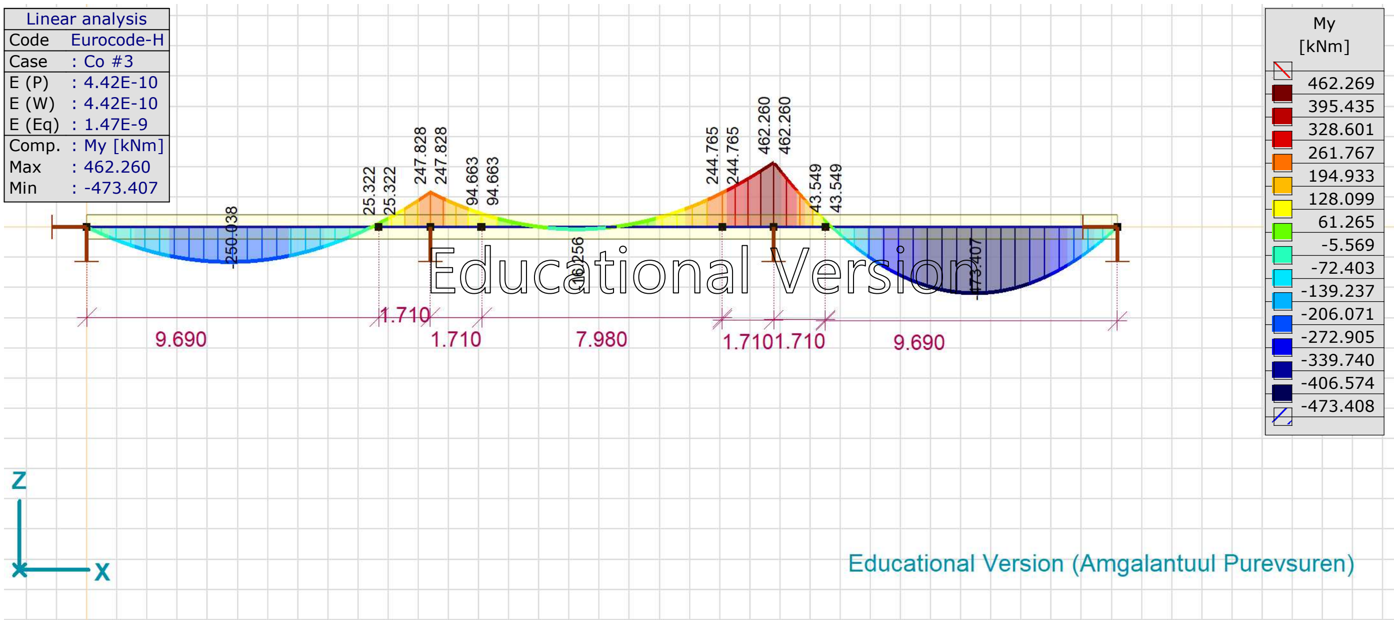
Variable 3, Front view



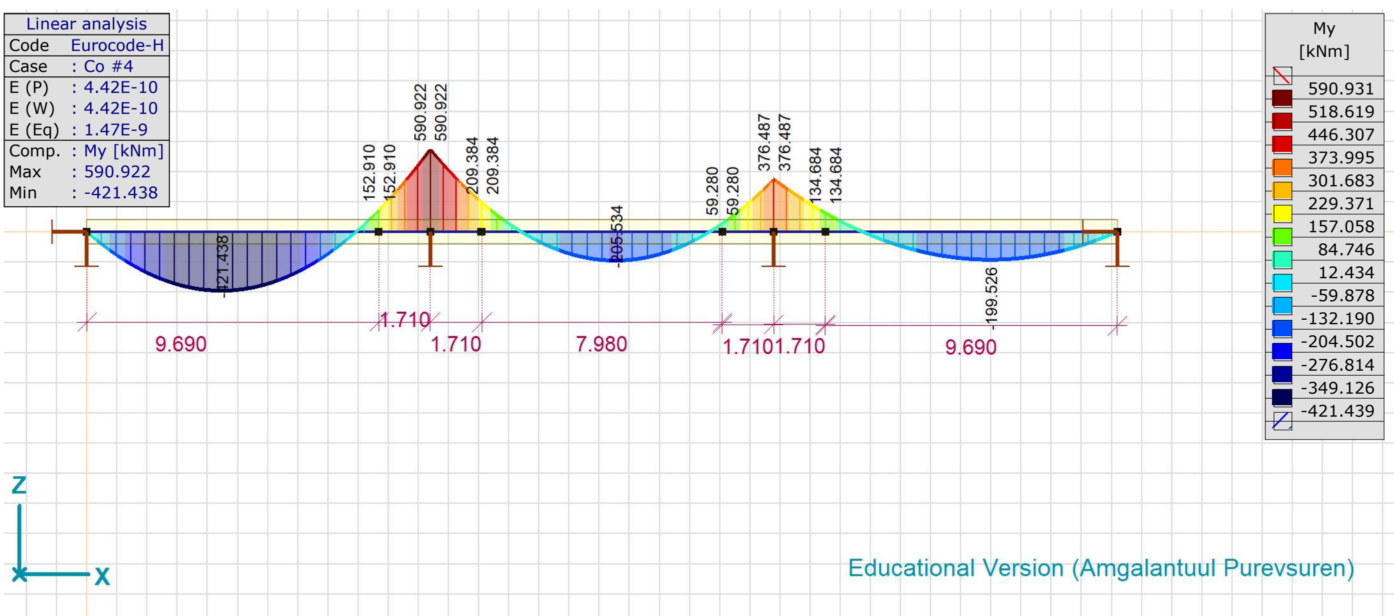
[I], Linear, Co #1 (ULS)



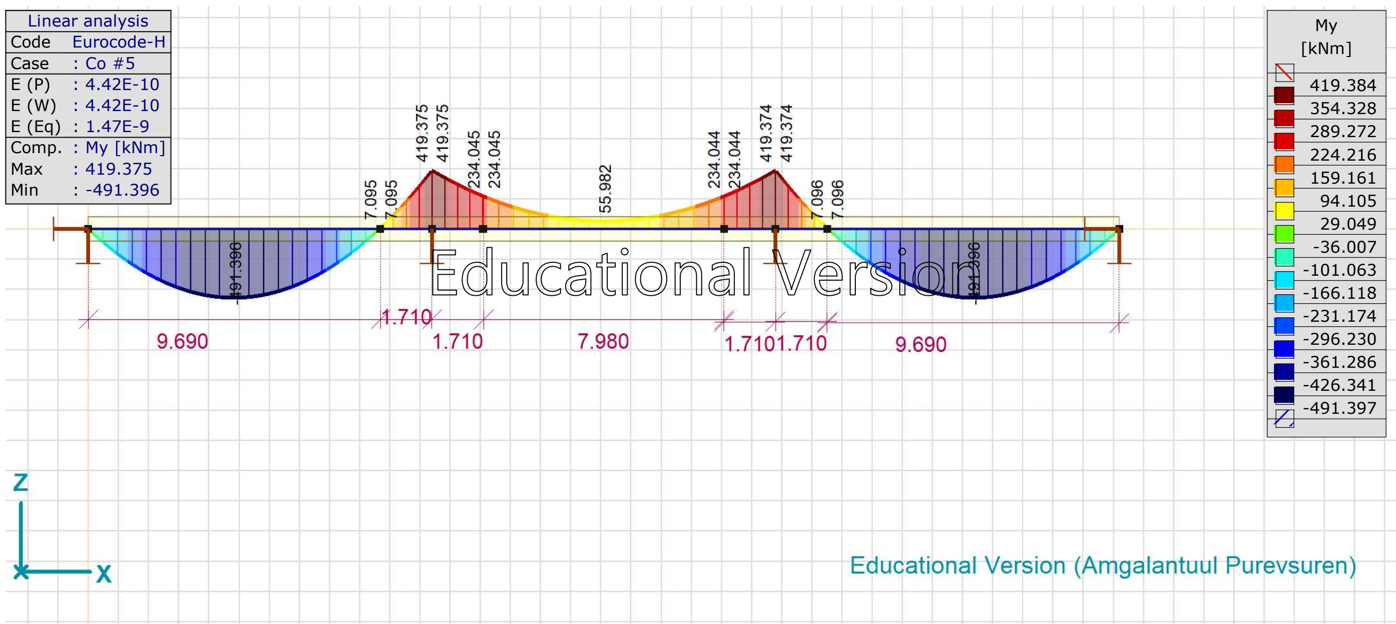
[I], Linear, Co #2 (ULS)



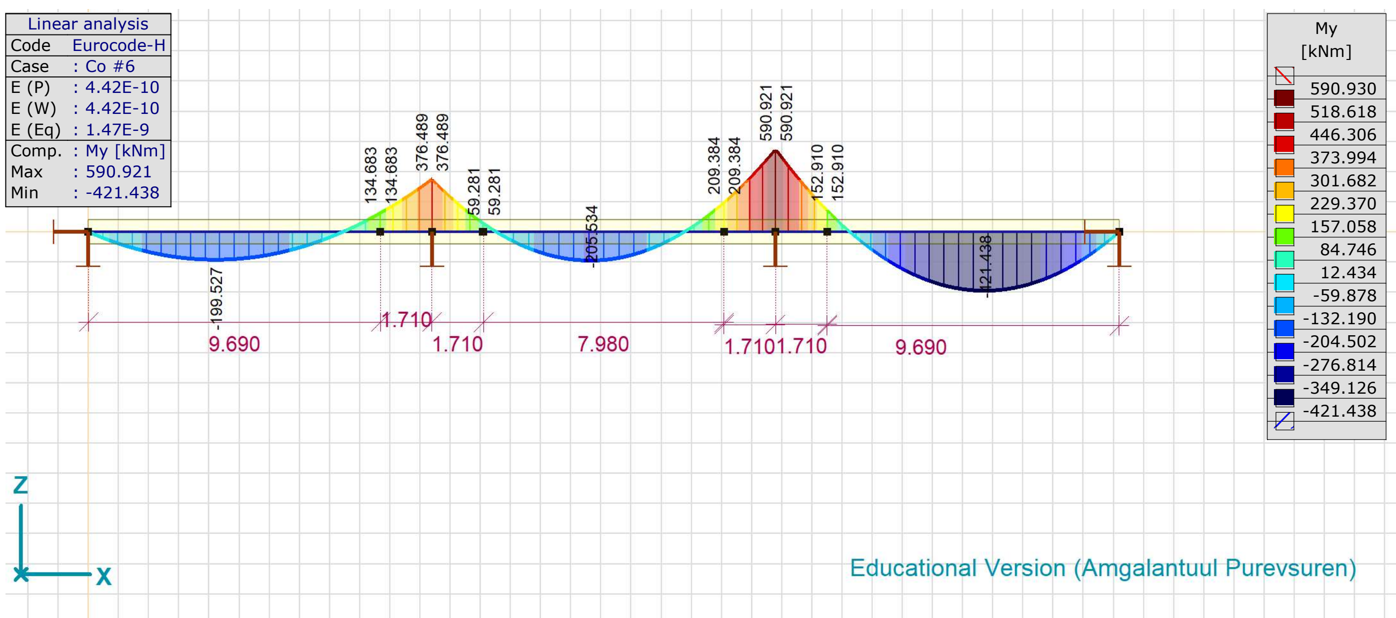
[I], Linear, Co #3 (ULS)



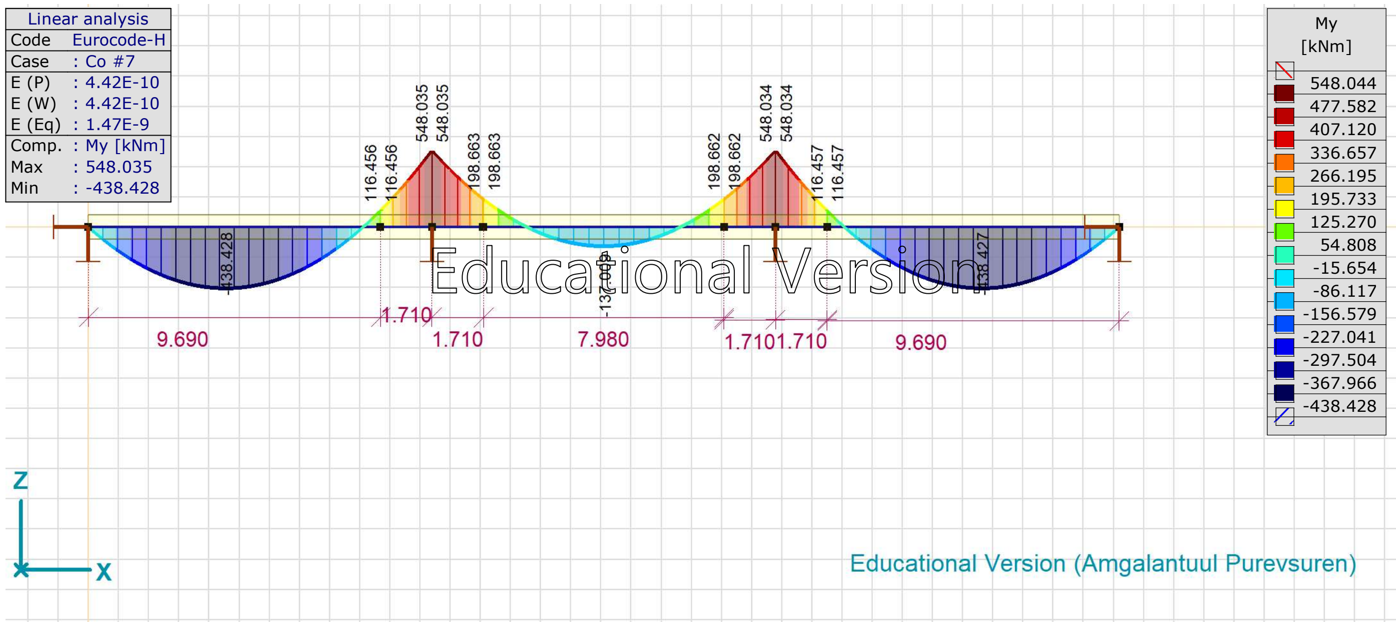
[I], Linear, Co #4 (ULS)



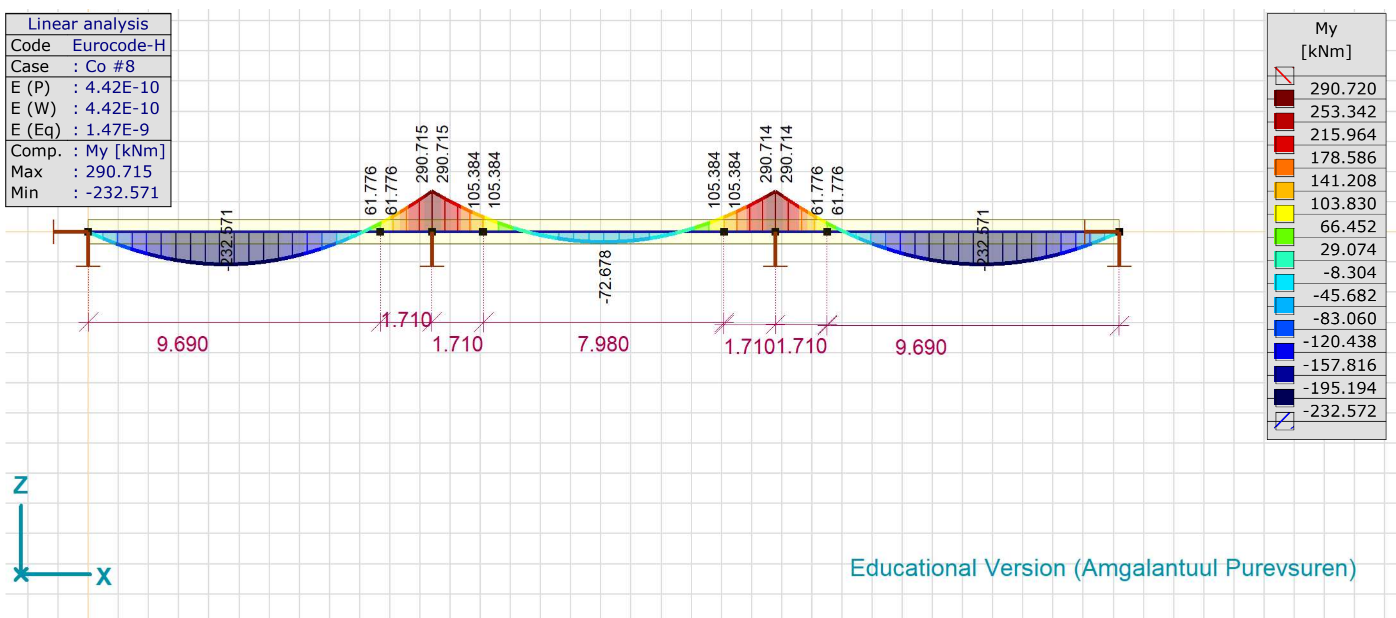
[I], Linear, Co #5 (ULS)



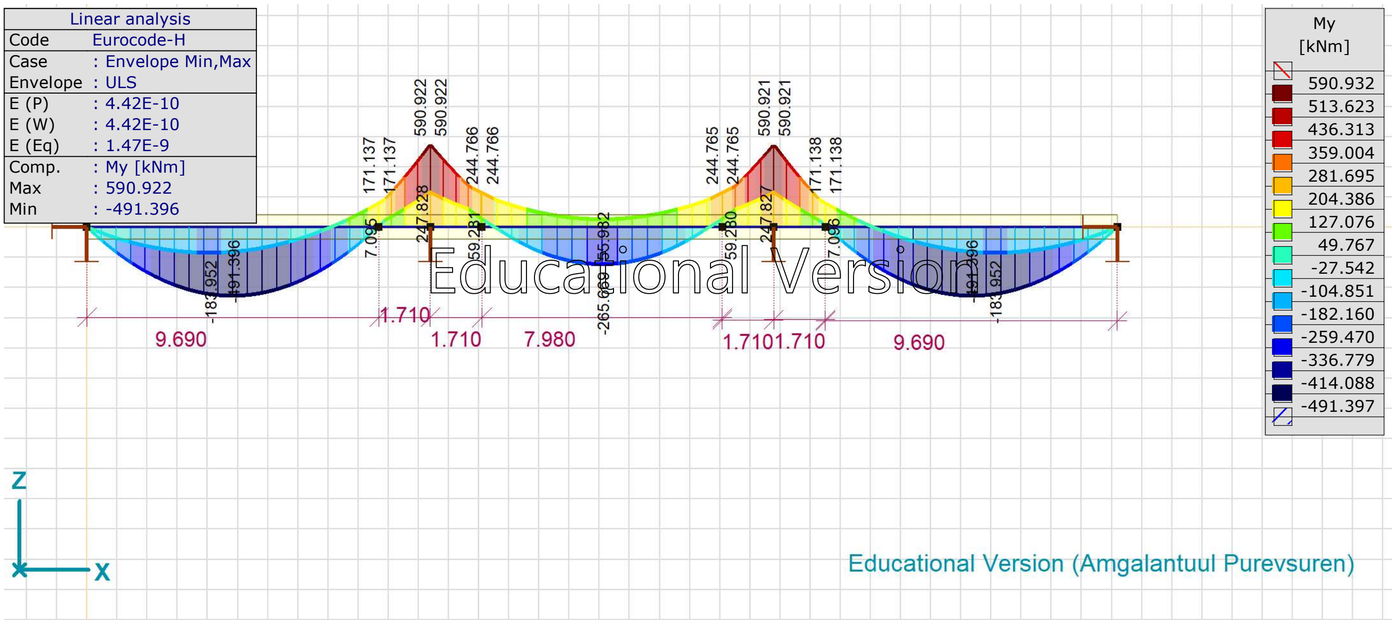
[I], Linear, Co #6 (ULS)



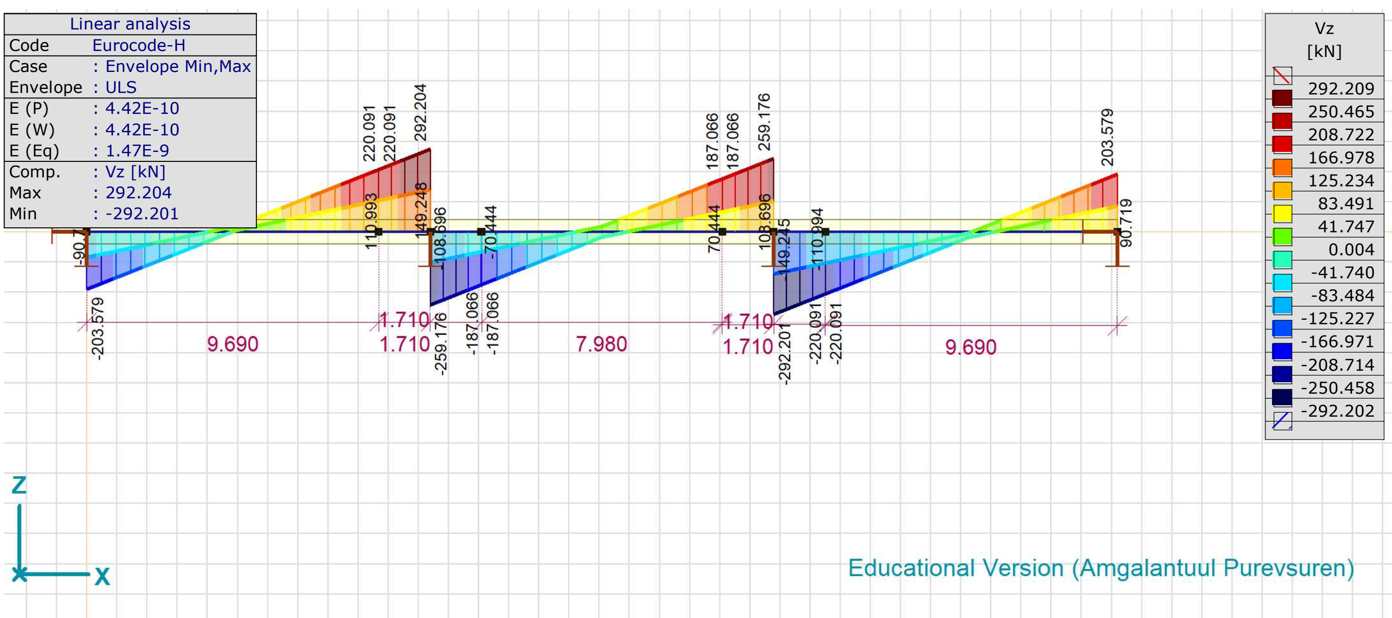
[I], Linear, Co #7 (ULS)



[I], Linear, Co #8 (ULS)



[I], Linear, Envelope (ULS)



[I], Linear, Envelope (ULS) Vz