

SZIE-Ybl Miklós Építéstudományi Kar

SZAKDOLGOZAT

Nagy Erika

Mechanika és Tartószerkezeti Szakcsoport

Szerkezeti műszaki leírás

a diplomaterv tárgyát képező kéthajós acélszerkezetű daruzott
csarnok terveihez

Tárgy: Magasépítési szakirányú diplomamunka feladat
2015.

Tartalom:

- I. Bevezetés
- II. Acélszerkezetű csarnok

Készítette: Nagy Erika
SZIE-YMÉK, levelező hallgató
Neptun kód: [REDACTED]
Konzulens: Horváth Katalin

I. Bevezetés

A feladat tárgyát képező csarnokszerkezetek a megbízás szerint közvetlenül egymás mellett épülnek fel, egymással csuklós kapcsolattal. Méreteket és elrendezéseiket Megbízó, a SZIE-YMÉK ipari építői igények mentén határozták meg. A megbízáshoz építész szakági vázlattervet, műleírást és pontos szakági tervezői programot kaptam, mely elvárásaival minden szerkezettervezési részletre kiterjedt.

Terveimben ezért az általam tervezett szerkezetek tervezési határait feltüntettem, építészeti funkciói, esztétikai megjelenése, illetve kapcsolati kidolgozottsága minden esetben építész szakági részlettervhez igazodik, melyet szintén feltüntettem.

1. Acél csarnokszerkezet

- **Általános kialakítás:** kéthajós földszintes, daruzott hőszigetelt ipari csarnok méretezés szerinti anyagokból és szerkezeti kialakításokkal.

- 1.1. Méretelrendezés:

Csarnok hét kereteretállásból áll, tengelytávolsága: 6,60 méter.

A főhajó teljes belmagasság: 9,95 méter , vállmagasság: 7,4 méter

Mellék hajó teljes belmagasság: 4,85 méter , válmagasság: 4,4 méter

Főhajó oszlopainak tengelytávolsága 18 méter, a mellék hajó oszlopok tengelytávolsága 6,00 méter

Daru sínkorona magassága: 5,00 méter

Teherbírása 5,00 tonna (részletesen ld. Számítási melléklet III. – daru méretezés)

Tengelytávolság: 16,00 méter

- 1.2. Felhasznált anyagok, kapcsolatok

Keretlábak, gerendák, szelemenek: S 235

Felső trapézlemez: LTP 45/0,7

Alaptestek betonminősége: C20/25 – XC1 – 24 – F2

Szerkezeti laposacélok, csomólemezek: S235

Kapcsolatok:

- I. osztályú kétoldali sarokvarratok, gyártáskor illetőleg helyszíni kapcsolatoknál - $a \cong 0,7$ v gyökmérettel, a hegesztések végigmenő hegesztések (pontszerű hegesztéseket nem terveztünk).
- Csavarkapcsolatoknál a terveink szerinti méretű és darabszámú hatlapfejű nyers csavarokat alkalmaztunk (+ csavaranya és alátét). Furat minden esetben a d+1 méretkiíráshoz igazodik.
- Kapcsolóelemek: méretezés szerinti laposacél elemek, a rögzítő csavarkapcsolatok helyein $\varphi = d+1$ (mm) furattal.

- 1.3. Keretszerkezetek:

Keretoszlopok: HEA 400 –as melegen hengerelt acélszelvények

Keretgerendák: IPE 500

Falvázoszlopok: HEA 200

Mellékhajó oszlop: HEA 300

Mellékhajó gerenda: IPE400

Szelemenek: Z 200 vékonyfalú acélgerendák 1600 mm-es távolságokkal, mellékrajónál Z 200 2,5 mm vtg. acélgerendák 600 mm-es távolságokkal

Tetőburkolat: hőszigetelt LINDAB LTP 45 trapézlemez – ld. A-4 terven.

Falburkolat: hőszigetelt LINDAB LV20 trapézlemez – ld. A-4 terven.

Az oszlopok és gerendák hegesztett és csavarozott kapcsolatokkal csatlakozó, acélszelvények (A-3 terv).

A darupályatartó HEB 320 szelvényt tartó rövidkonzolok (IPE 400) kiékelve csatlakoznak rendre az acél keretoszlopokhoz (toldást ld. 6-os csomóponti részletrajzon).

A szélrács, illetve a hosszmerevítés szerkezeti kialakítása (A-5 rajz) tervünk szerint $\varnothing 20$ tömör köracél rudakkal történik majd, hegesztett és csavarozott, csomólemezt alkalmazó kapcsolatokkal.

Falpanelek: hőszigetelt LINDAB szendvicspanelek beépítése, oszlopokhoz történő rögzítése „U” vékonyfalú szelvényekkel és speciális profilokkal építész szakági részlettervek szerint történik.

- szakági részlettervek szerint kialakítandók.

- 1.4. Alapozási szerkezetek:

A Megbízó által rendelkezésemre bocsátott talajvizsgálati mellékletben (fúrásszelvény) – A-6 terv alapján az alapozási sík a fagyhatár alatt van a teherhordó talajon – $A_s = -1,50$ m (padlóvonal = $\pm 1,00$ m) lett felvéve, talajvízre számítani nem kell.

Az oszlopok alatti vasbeton tömbalapok geometriája méretezés szerint készült.

Alaptesteket talpgerendával kell összekötni a szüllyedés csökkentése miatt.

Betonminőség: C 20/25 Lehorgonyzási csavarok : $\varnothing 30/600$ betonacél, felső végén 100 mm hosszon M27 csavarkapcsolatra történő kialakítással.

Aljzatbeton: 5 cm vastagságú C 12/15 minőségű betonból készülhet a megfelelően tömörített ($T_{ry} \cong 90\%$) altalajon /kavicságyon.

Burkolati rétegződés, szigetelési utasítások a terveinkhez kapcsolódó építész

2.4. Felületvédelem

Felület előkészítés:	Sa 2,5 fokozat
Rétegfelépítés:	1.rtg. 75mm szürke Gehopon Ex-Zink 2.rtg. 75 mm vörösbarna Gehopon EXE1-20+EXE34 3.rtg. 50 mm Wierengen ACU M8+DX4
Acélszerkezet színe:	Brillians kék RAL 5007
Acélszerkezet tűzvédelme:	1 órás tűzállóság biztosítása Porotherm Steel tűzvédő bevonattal (vagy azzal egyenértékű anyaggal)

2. Felhasznált irodalom, szabványok

- EN 1990 EUROCODE 0	A tartószerkezeti tervezés alapjai
- EN 1991 EUROCODE 3	Acélszerkezetek tervezése
- EN 1992 EUROCODE 2	Betonszerkezetek tervezése

Munkám során több szakági méretező programmal készítettem rész- vagy teljes modellt, melyek segítették munkámat:

- Terveimet AUTOCAD 2010-es verziójával készítettem el – „AUTODESK Oktatói változattal” ellátott szoftverrel;
- Számításomban a tehermodellt és az erőjátékot, valamint a teherkombinációkat AXIS VM 12 Hallgatói kiterjesztésű statikai méretező programjával építettem fel és ellenőriztem. A mértékadó értékek kivétele csomóponti számításaim alapja.

- Tetőszerkezeteim méretellenőrzését DIMROOF 3.5 programmal végeztem (LINDAB Tervezői portálról jogtiszta formában) – modelleim a számításban megtalálhatóak;
- A tartószerkezeti modellt egy keretállásra ROBOT structural programcsomaggal végeztem.

Kecskemét, 2015. Június



YBL MIKLÓS ÉPÍTÉSTUDOMÁNYI KAR, BUDAPEST

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Részletes feladatkiírás

Nagy Erika

Levelező Építőmérnök
hallgató részére

Kéthajós, daruzott ipari csarnok tervezése

című szakdolgozatához

Feladat leírása

Megtervezendő egy kéthajós, daruzott, acélszerkezetű, hőszigetelt ipari csarnok.

Geometriai kialakítás, terhek

A csarnok hét keretállásból áll, a keretállások távolsága 6,60 m (1 – 7 tengelyek).

A csarnok főhajó oszlopok tengelytávolsága 18,00 m (A és H tengelyek), a mellékrajó oszlopok tengelytávolsága 6,00 m (H – K tengelyek).

A csarnokba telepítésre kerülő daru (A és H tengelyek):

- tengelytávolsága : 17,00 m,
- sínkorona magassága: ~5,00 m,
- teherbírása : 5 t.

A csarnok főhajó belmagasságát a telepítésre kerülő daru alapján kell meghatározni.

A csarnok főhajó váll tengely magassága ~7,40 m, a gerincmagasság ~9,10 m, a tető lejtése ~7°.

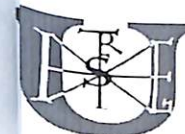
A mellékrajók váll tengely magassága ~4,40 m, a tető lejtése ~7°.

A csarnok hőszigetelt.

A terheket a vonatkozó szabványok, előírások és irányelvek alapján kell felvenni.

Budapest, 2015. május 28.

Horváth Katalin
Horváth Katalin
belső konzulens



SZENT ISTVÁN
EGYETEM

Dr. Dombay Gábor PhD.
intézetigazgató, főiskolai tanár



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SZAKDOLGOZAT KIÍRÁS
2015

Nagy Erika

Levelező Építőmérnök szakos
hallgató részére

A feladat címe: Kéthajós, daruzott ipari csarnok tervezése

Belső konzulens: Horváth Katalin mérnöktanár

Külső konzulens: -

A szakdolgozat beadási határideje: 2015. június 05., péntek 12⁰⁰

Budapest, 2015. május 28.

Dr. Dombay Gábor
intézetigazgató



SZENT ISTVÁN
EGYETEM



YBL MIKLÓS ÉPÍTÉSTUDOMÁNYI KAR, BUDAPEST

Mechanika és Tartószerkezeti Szakcsoport

Számítási melléklet I.

Kéthajós gyártó-bemutató, hőszigetelt csarnok kiviteli terve
Tartószerkezeti számítási melléklet

Első rész: acélszerkezetű, daruzott gyártó-üzemrész

Tartalom:

- I. Főtartó méretezése**
- II. „Z” szelemen méretezés**
- III. Trapézlemez méretezése**
- IV. Főtartó keret vizsgálatai**
- V. Falvázoszlop méretezése**
- VI. Merevítő szélrács
méretezése**
- VII. Csomópontok méretezése**
- VIII. Kiegészítő számítás: alapok**

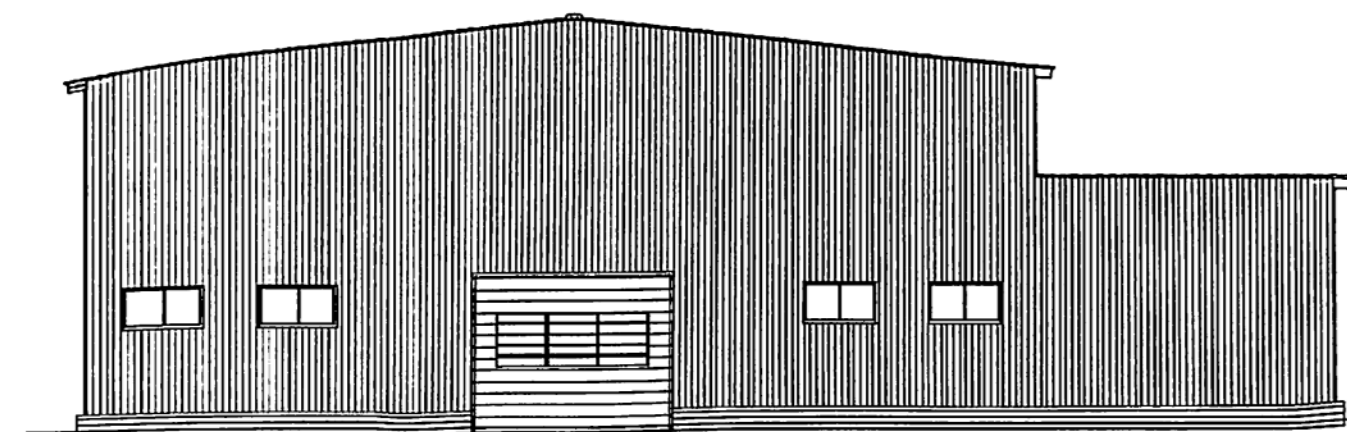
Készítette: Nagy Erika

SZIE-YMÉK, levelezőtagozat

Neptun kód: D8WK20

Konzulens: Horváth Katalin

ACÉLSZERKEZETŰ CSARNOK



I. FŐTARTÓ MÉRETEZÉSE

Keret terhek

Állandó terhek

axis-ból

Rétegrend

mérték.egy $\frac{kg}{m^2}$

külső trapézlemez LTP45/0.7
z szelemen Z200
hőszigetelés 15 cm
párázáró fólia 0.1 cm
belső trapézlemez LTP45/0.7

$g_{ktl} := 0.07$
 $g_z := 0.072$
 $g_{hsz} := 0.12$
 $g_{sz} := 0.014$
 $g_{btl} := 0.07$

Burkolat súlya

$t := 6.6 \text{ m}$ $t_z := 1.6 \text{ m}$

$$P_{burkolat} := t \cdot (g_{ktl} + g_{hsz} + g_{sz} + g_{btl}) + \frac{(t \cdot g_z)}{t_z} = 2.105 \frac{kN}{m}$$

Állandó jellegű hasznos teher gépészet

$$F_g := 5 \text{ kN}$$

Hóteher (vetületi teher)

$$A := 400 \text{ m}$$

$$s_k := 1.25 + \frac{A - 400}{400} = 1.25$$

$$\mu := 0.8 \quad 0^\circ < \alpha < 30^\circ$$

$$C_e := 0.8 \text{ széles}$$

$$C_t := 1.0$$

$$s := C_e \cdot C_t \cdot \mu \cdot s_k = 0.8$$

$$P_{sk1} := t \cdot s = 5.28 \frac{\text{kN}}{\text{m}}$$

$$P_{sk2} := \frac{P_{sk1}}{2} = 2.64 \frac{\text{kN}}{\text{m}}$$

Szélteher

Terepkategória IV

$$z := 7400 + 250 + 200 + 45 + 1100 = 8995 \text{ mm}$$

$$\text{szél torlónyomás} \quad qpz := 0.2955$$

$$cpe := 0.2$$

$$cpi := -0.3$$

$$wek := qpz \cdot cpe = 0.059$$

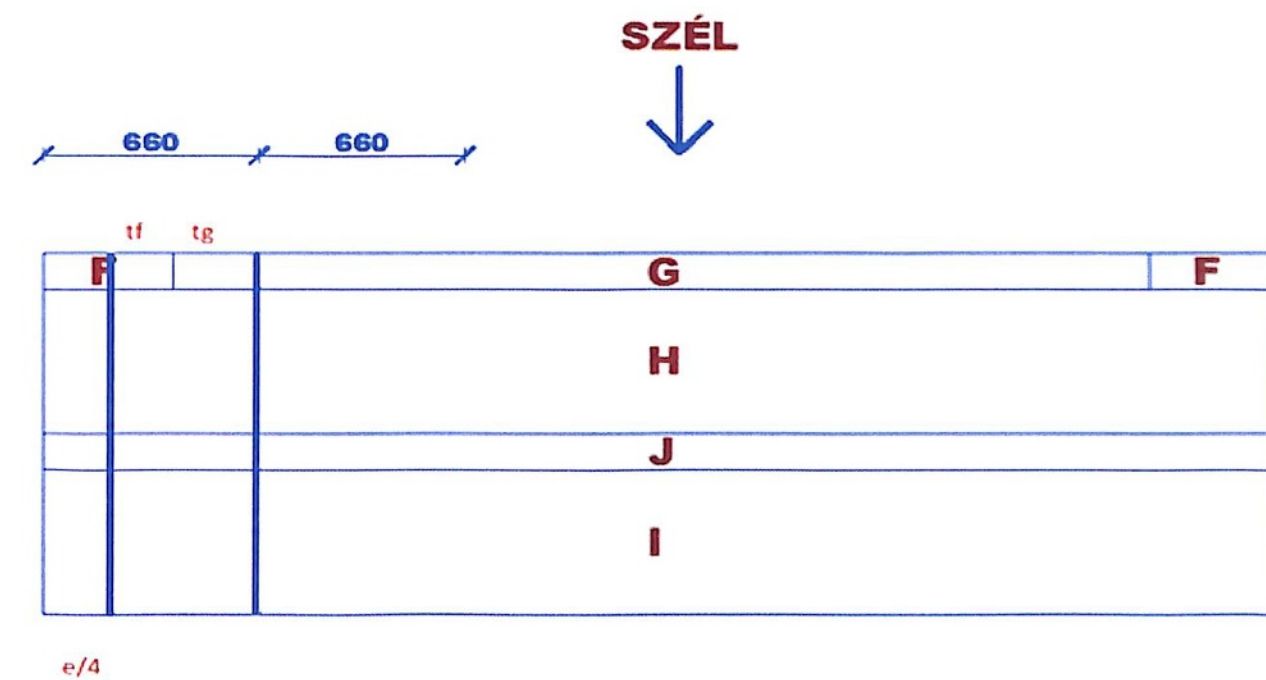
$$wik := qpz \cdot cpi = -0.089$$

15 m-nél kisebb épületnél

$$cs := 1$$

$$cd := 1$$

Keresztirányú Külső



$$e := z \cdot 2 = 17990 \text{ mm}$$

$$b := 39600 \text{ mm}$$

$$\frac{e}{10} = 1799 \text{ mm}$$

$$\frac{e}{4} = 4497.5 \text{ mm}$$

$$\frac{e}{2} = 8995 \text{ mm}$$

"I" zóna

$$\alpha := 7 \quad t := 6.6 \text{ m}$$

$$5^\circ \quad cpe10 := -0.6$$

$$15^\circ \quad cpe10' := -0.4$$

$$cpe1 := \left(\frac{(cpe10' - cpe10)}{15 - 5} \right) \cdot (\alpha - 5) + (cpe10) = -0.56$$

$$5^\circ \quad cpe.10 := -0.6$$

$$15^\circ \quad cpe.10' := 0.0$$

$$P_{wesz0Ik'} := t \cdot qpz \cdot cpe1 = -1.092 \quad \frac{kN}{m}$$

$$cpe2 := \left(\frac{(cpe.10' - cpe.10)}{15 - 5} \right) \cdot (\alpha - 5) + (cpe.10) = -0.48$$

$$P_{wesz0Ik} := t \cdot qpz \cdot cpe2 = -0.936 \quad \frac{kN}{m}$$

"J" zóna

$$\alpha := 7 \quad t := 6.6 \text{ m}$$

$$5^\circ \quad cpe10 := 0.2$$

$$15^\circ \quad cpe10' := -1$$

$$cpe1 := \left(\frac{(cpe10' - cpe10)}{15 - 5} \right) \cdot (\alpha - 5) + (cpe10) = -0.04$$

$$5^\circ \quad cpe.10 := -0.6$$

$$15^\circ \quad cpe.10' := 0.0$$

$$P_{wesz0Jk'} := t \cdot qpz \cdot cpe1 = -0.078 \quad \frac{kN}{m}$$

$$cpe2 := \left(\frac{(cpe.10' - cpe.10)}{15 - 5} \right) \cdot (\alpha - 5) + (cpe.10) = -0.48$$

$$P_{wesz0Jk} := t \cdot qpz \cdot cpe2 = -0.936 \quad \frac{kN}{m}$$

"H" zóna

$$\alpha := 7 \quad t := 6.6 \text{ m}$$

$$5^\circ \quad cpe10 := -0.6$$

$$15^\circ \quad cpe10' := -0.3$$

$$cpe1 := \left(\frac{(cpe10' - cpe10)}{15 - 5} \right) \cdot (\alpha - 5) + (cpe10) = -0.54$$

$$5^\circ \quad cpe.10 := 0$$

$$15^\circ \quad cpe.10' := 0.2$$

$$P_{wesz0Hk'} := t \cdot qpz \cdot cpe1 = -1.053 \quad \frac{kN}{m}$$

$$cpe2 := \left(\frac{(cpe.10' - cpe.10)}{15 - 5} \right) \cdot (\alpha - 5) + (cpe.10) = 0.04$$

$$Pwesz0Hk := t \cdot qpz \cdot cpe2 = 0.078 \frac{kN}{m}$$

"F" zóna

$$\alpha := 7 \quad tf := 1.1975m$$

$$5^\circ \quad cpe.10 := -1.7$$

$$15^\circ \quad cpe.10' := -0.9$$

$$cpe1 := \left(\frac{(cpe.10' - cpe.10)}{15 - 5} \right) \cdot (\alpha - 5) + (cpe.10) = -1.54$$

$$5^\circ \quad cpe.10 := 0$$

$$15^\circ \quad cpe.10' := 0.2$$

$$Pwesz0Fk' := tf \cdot qpz \cdot cpe1 = -0.545 \frac{kN}{m}$$

$$cpe2 := \left(\frac{(cpe.10' - cpe.10)}{15 - 5} \right) \cdot (\alpha - 5) + (cpe.10) = 0.04$$

$$Pweny0Fk := tf \cdot qpz \cdot cpe2 = 0.014 \frac{kN}{m}$$

"G" zóna

$$\alpha := 7 \quad tg := 5.4025m$$

$$5^\circ \quad cpe.10 := -1.2$$

$$15^\circ \quad cpe.10' := -0.8$$

$$cpe1 := \left(\frac{(cpe.10' - cpe.10)}{15 - 5} \right) \cdot (\alpha - 5) + (cpe.10) = -1.12$$

$$5^\circ \quad cpe.10 := 0$$

$$15^\circ \quad cpe.10' := 0.2$$

$$Pwesz0Gk' := tg \cdot qpz \cdot cpe1 = -1.788 \frac{kN}{m}$$

$$cpe2 := \left(\frac{(cpe.10' - cpe.10)}{15 - 5} \right) \cdot (\alpha - 5) + (cpe.10) = 0.04$$

$$Pweny0Gk := tg \cdot qpz \cdot cpe2 = 0.064 \frac{kN}{m}$$

Fal

$$z = 8995 \text{ mm}$$

$$d := 18000 \text{ mm}$$

$$\frac{z}{d} = 0.4997 \quad t := 6.6 \text{ m}$$

"D" zóna

$$1 \quad cpe10 := 0.8$$

$$0.25 \quad cpe10' := 0.7$$

$$cpe1 := \left(\frac{(cpe10 - cpe10')}{1 - 0.25} \right) \cdot \left(\frac{z}{d} - 0.25 \right) + (cpe10') = 0.7333$$

$$P_{wesz0Dk'} := t \cdot qpz \cdot cpe1 = 1.43 \quad \frac{kN}{m}$$

"E" zóna

$$1 \quad cpe10 := -0.5$$

$$0.25 \quad cpe10' := -0.3$$

$$cpe1 := \left(\frac{(cpe10 - cpe10')}{1 - 0.25} \right) \cdot \left(\frac{z}{d} - 0.25 \right) + (cpe10') = -0.367$$

$$P_{weny0Ek'} := t \cdot qpz \cdot cpe1 = -0.715 \quad \frac{kN}{m}$$

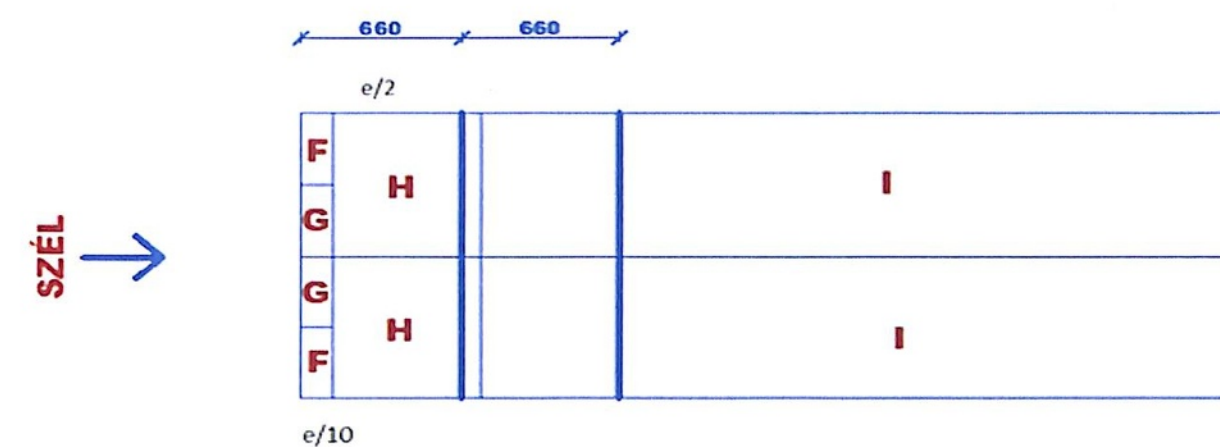
Hosszírányú Külső 3HK

$$e := z \cdot 2 = 17990 \text{ mm}$$

$$\frac{e}{10} = 1799 \text{ mm}$$

$$\frac{e}{4} = 4497.5 \text{ mm}$$

$$\frac{e}{2} = 8995 \text{ mm}$$



"H" zóna

$$\alpha := 7 \quad t := 0.905 \quad m$$

$$5^\circ \quad cpe10 := -0.7$$

$$15^\circ \quad cpe10' := -0.6$$

$$cpe1 := \left(\frac{(cpe10' - cpe10)}{15 - 5} \right) \cdot (\alpha - 5) + (cpe10) = -0.68$$

$$P_{wesz0Hk'} := t \cdot qpz \cdot cpe1 = -0.182 \quad \frac{kN}{m}$$

"I" zóna

$$\alpha := 7 \quad t := 5.695 \quad m$$

$$5^\circ \quad cpe10 := -0.6$$

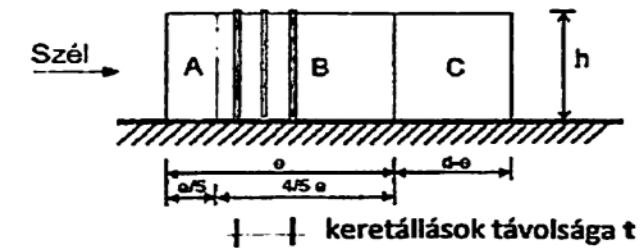
$$15^\circ \quad cpe10' := -0.5$$

$$cpe1 := \left(\frac{(cpe10' - cpe10)}{15 - 5} \right) \cdot (\alpha - 5) + (cpe10) = -0.58$$

$$P_{weszIk'} := t \cdot qpz \cdot cpe1 = -0.976 \quad \frac{kN}{m}$$

Fal

Oldalnézet $e < d$ esetén



$$e := z \cdot 2 = 17990 \text{ mm}$$

$$b := 18132 \text{ mm}$$

$$\frac{e}{5} = 3598 \text{ mm}$$

$$ta := 0.298 \text{ m}$$

"A" zóna

$$cpe10 := -1.2$$

$$cpe10' := -1.2$$

$$cpe1 := \left(\frac{(cpe10' - cpe10)}{15 - 5} \right) \cdot (\alpha - 5) + (cpe10) = -1.2$$

$$P_{weszAk'} := ta \cdot qpz \cdot cpe1 = -0.106 \quad \frac{kN}{m}$$

"B" zóna

$$cpe10 := -0.8$$

$$cpe10' := -0.8$$

$$tb := 6.302 \text{ m}$$

$$cpe1 := \left(\frac{(cpe10' - cpe10)}{15 - 5} \right) \cdot (\alpha - 5) + (cpe10) = -0.8$$

$$P_{weszBk'} := tb \cdot qpz \cdot cpe1 = -1.49 \quad \frac{kN}{m}$$

Belső nyomás

Nincs információ, hogy hova kerülnek a nyílások

$$cpei := 0.2$$

$$t := 6.6 \text{ m}$$

$$P_{winjk} := t \cdot qpz \cdot cpei = 0.39 \quad \frac{kN}{m}$$

Belső szívás

$$c_{pei} := -0.3$$

$$t := 6.6 \text{ m}$$

$$P_{wisk} := t \cdot q_{pz} \cdot c_{pei} = -0.585 \frac{kN}{m}$$

II. "Z" SZELEMEN MÉRETEZÉS**Z szelemen terhek****Rétegrend**

$\alpha := 7$

$t := 1.6 \text{ m}$

külső trapézlemez LTP45/0.7

$g_{ktl} := 0.07$

z szelemen Z200

$g_z := 0.072$

hőszigetelés 15 cm

$g_{hsz} := 0.12$

párazáró fólia 0.1 cm

$g_{sz} := 0.014$

belső trapézlemez LTP45/0.7

$g_{btl} := 0.07$

Állandó terher

$$P_{gk} := (t \cdot (g_{ktl} + g_{btl} + g_{hsz} + g_{sz})) + (g_z) = 0.5104 \frac{kN}{m}$$

$\alpha < 10^\circ \quad \text{nem kell redukálni}$

Hóteher (vetületi terher)

$A := 400 \text{ m}$

$t := 1.6 \text{ m}$

$$s_k := 1.25 + \frac{A - 400}{400} = 1.25$$

$\mu := 0.8$

$0^\circ < \alpha < 30^\circ$

$ce := 0.8 \quad \text{szeles}$

$\alpha := 7$

$ct := 1.0$

$s := ce \cdot ct \cdot \mu \cdot s_k = 0.8$

$$P_{sk} := t \cdot s = 1.28 \frac{kN}{m}$$

Szélteher**Keresztírányú - Külső**

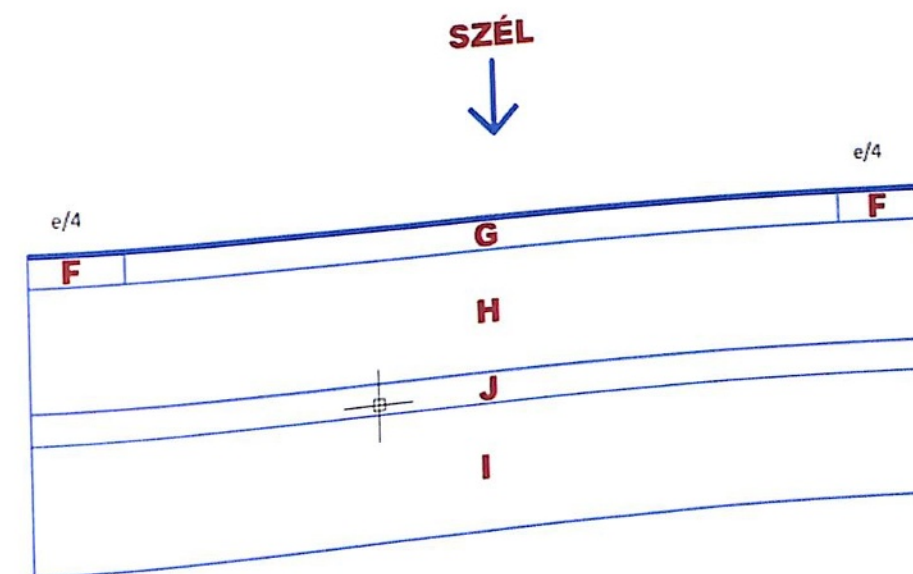
$e := z \cdot 2 = 17990 \text{ mm}$

$$\frac{e}{10} = 1799 \text{ mm}$$

$$\frac{e}{4} = 4497.5 \text{ mm}$$

szél torlónyomás

$q_{pz} := 0.2955$



$$\alpha := 7 \quad t := 1.6 \text{ m}$$

"F" zóna

$$\begin{aligned} 5^\circ \quad cpe_{10} &:= -1.7 \\ 15^\circ \quad cpe_{10'} &:= -0.9 \end{aligned}$$

$$cpe_1 := \left(\frac{(cpe_{10'} - cpe_{10})}{15 - 5} \right) \cdot (\alpha - 5) + (cpe_{10}) = -1.54$$

$$P_{weszFk'} := t \cdot qpz \cdot cpe_1 = -0.728 \quad \frac{kN}{m}$$

$$\begin{aligned} 5^\circ \quad cpe_{.10} &:= 0 \\ 15^\circ \quad cpe_{.10'} &:= 0.2 \end{aligned}$$

$$cpe_2 := \left(\frac{(cpe_{.10'} - cpe_{.10})}{15 - 5} \right) \cdot (\alpha - 5) + (cpe_{.10}) = 0.04$$

$$P_{wenyFk} := t \cdot qpz \cdot cpe_2 = 0.019 \quad \frac{kN}{m}$$

$$\alpha := 7 \quad t := 1.6 \text{ m}$$

"G" zóna

$$\begin{aligned} 5^\circ \quad cpe_{10} &:= -1.2 \\ 15^\circ \quad cpe_{10'} &:= -0.8 \end{aligned}$$

$$cpe_1 := \left(\frac{(cpe_{10'} - cpe_{10})}{15 - 5} \right) \cdot (\alpha - 5) + (cpe_{10}) = -1.12$$

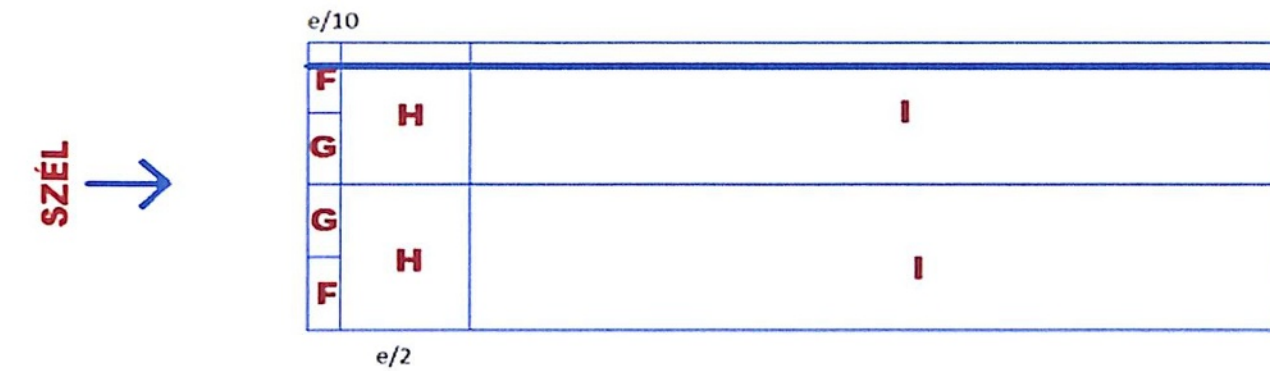
$$P_{weszGk'} := t \cdot qpz \cdot cpe_1 = -0.53 \quad \frac{kN}{m}$$

$$\begin{aligned} 5^\circ \quad cpe_{.10} &:= 0 \\ 15^\circ \quad cpe_{.10'} &:= 0.2 \end{aligned}$$

$$cpe_2 := \left(\frac{(cpe_{.10'} - cpe_{.10})}{15 - 5} \right) \cdot (\alpha - 5) + (cpe_{.10}) = 0.04$$

$$P_{wenyGk} := t \cdot qpz \cdot cpe_2 = 0.019 \quad \frac{kN}{m}$$

Hosszírányú - Külső



"F" zóna

$$\alpha := 7 \quad t := 1.6 \quad m$$

$$5^\circ \quad cpe10 := -1.6$$

$$15^\circ \quad cpe10' := -1.3$$

$$cpe1 := \left(\frac{(cpe10' - cpe10)}{15 - 5} \right) \cdot (\alpha - 5) + (cpe10) = -1.54$$

$$P_{weszFk'} := t \cdot qpz \cdot cpe1 = -0.728 \quad \frac{kN}{m}$$

"H" zóna

$$\alpha := 7 \quad t := 1.6 \quad m$$

$$5^\circ \quad cpe10 := -0.7$$

$$15^\circ \quad cpe10' := -0.6$$

$$cpe1 := \left(\frac{(cpe10' - cpe10)}{15 - 5} \right) \cdot (\alpha - 5) + (cpe10) = -0.68$$

$$P_{weszHk'} := t \cdot qpz \cdot cpe1 = -0.322 \quad \frac{kN}{m}$$

"I" zóna

$$\alpha := 7 \quad t := 1.6 \quad m$$

$$5^\circ \quad cpe10 := -0.6$$

$$15^\circ \quad cpe10' := -0.5$$

$$cpe1 := \left(\frac{(cpe10' - cpe10)}{15 - 5} \right) \cdot (\alpha - 5) + (cpe10) = -0.58$$

$$P_{weszIk'} := t \cdot qpz \cdot cpe1 = -0.274 \quad \frac{kN}{m}$$

Belső nyomás

$$c_{pei} := 0.2 \quad t := 1.6 \text{ m}$$

$$P_{winyk} := t \cdot q_{pz} \cdot c_{pei} = 0.095 \quad \frac{kN}{m}$$

Belső szívás

$$c_{pei} := -0.3 \quad t := 1.6 \text{ m}$$

$$P_{wisk} := t \cdot q_{pz} \cdot c_{pei} = -0.142 \quad \frac{kN}{m}$$

Teherkombinációk

$$\gamma_{gsup} := 1.35 \quad \gamma_{ginf} := 1.0 \quad \psi_{0w} := 0.6$$

$$\gamma_s := 1.5 \quad \gamma_w := 1.5$$

1.ULS önsúly+hó+szélnyomás

$$P_{u1} := (\gamma_{gsup} \cdot P_{gk}) + (\gamma_s \cdot P_{sk}) + (\psi_{0w} \cdot \gamma_w \cdot (P_{winyk} + P_{wisk})) = 2.498 \quad \frac{kN}{m}$$

1.SLS

$$P_{s2} := (P_{gk}) + (P_{sk}) + (\psi_{0w} \cdot (P_{winyk} + P_{wisk})) = 1.717 \quad \frac{kN}{m}$$

2.ULS önsúly+ szélszívás (KK + Belső)

$$P_{u3f} := (\gamma_{ginf} \cdot P_{gk}) + (\gamma_w \cdot (P_{weszFk'} + P_{winyk})) = -0.44 \quad \frac{kN}{m}$$

$$P_{u3g} := (\gamma_{ginf} \cdot P_{gk}) + (\gamma_w \cdot (P_{weszGk'} + P_{winyk})) = -0.142 \quad \frac{kN}{m}$$

2.SLS

$$P_{s4f} := (P_{gk}) + (P_{weszFk'} + P_{winyk}) = -0.123 \quad \frac{kN}{m}$$

$$P_{s4g} := (P_{gk}) + (P_{weszGk'} + P_{winyk}) = 0.075 \quad \frac{kN}{m}$$

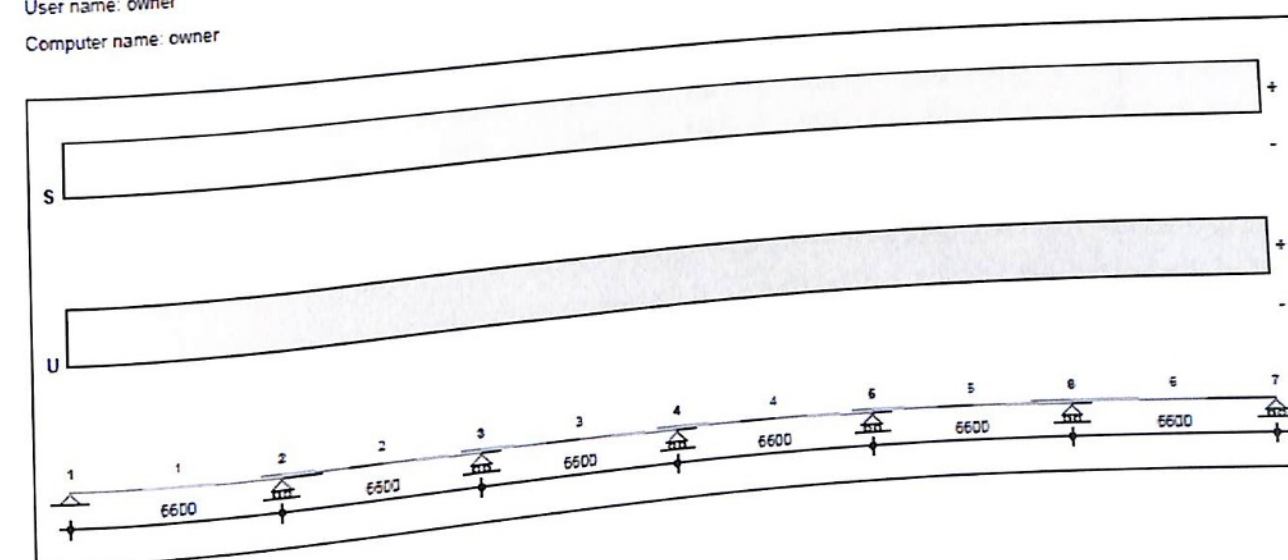
Dimroof

DIMroof 3.5 - D:\Diplomamunka\Erika\Z szelemen.dim

Date: 3/17/2015

User name: owner

Computer name: owner



Alapbeállítások

Funkció: Z-gerenda

Ország: Magyarország

Szerkezeti beállítások

Szelvény: Z 200

Statikai rendszer: Átfedésses Standard

Gyártó: Magyarország

Oldalsó támasz: Mindkét öv

Lemez Szelvény: LTP 45 Vastagság: 0.7

Csavar

Táv: Minden második völgyben

Típus Lemez: 4.8 Átfedés: 4.8 Támasz: 5.5

Lehajlás-számítás paraméterei

Lehajlási határ (nyílás): L / 200

Lehajlási határ (konzol): L / 150

Lehajlás típus: Maximális

Eredmények

ULS: 96%

SLS: 60%

Szabvány: Eurocode

Geometria						Nyílások			Toldóelem
No.	Pozíció [mm]	Típus	Támaszok			Méret [mm]	Vastagság 1 [mm]	Vastagság 2 [mm]	Vtg. [mm]
			Szélesség [mm]	L1 [mm]	L2 [mm]				
1	0	H	100	-	-	6600	2.0	-	-
2	6600	O	100	660	1320	6600	2.0	-	-
3	13200	O	100	660	660	6600	1.5	-	-
4	19800	O	100	660	660	6600	2.0	-	-
5	26400	O	100	660	660	6600	2.0	-	-
6	33000	O	100	1320	-	6600	2.0	-	-
7	39600	H	100	-	-	6600	2.0	-	-

Terhek							ULS/SLS
No.	Típus	Kezdőpont	Végpont	Kezdőint.	Végint.	Szélesség	ULS
							SLS
1	U			2.52			
2	U			1.74			

III. TRAPÉZLEMEZ MÉRETEZÉS

Trapézlemez terhek

Állandó teher

$$g_{k1} := 0.07$$

$$t := 1.0 \text{ m}$$

$$\alpha := 7$$

$$P_{gk} := t \cdot g_{k1} = 0.07 \frac{\text{kN}}{\text{m}}$$

Hóteher (vetületi teher)

$$s_k := 1.25 + \frac{A - 400}{400} = 1.25$$

$$c_e := 0.8 \text{ széles}$$

$$c_t := 1.0$$

$$s := c_e \cdot c_t \cdot \mu \cdot s_k = 0.8$$

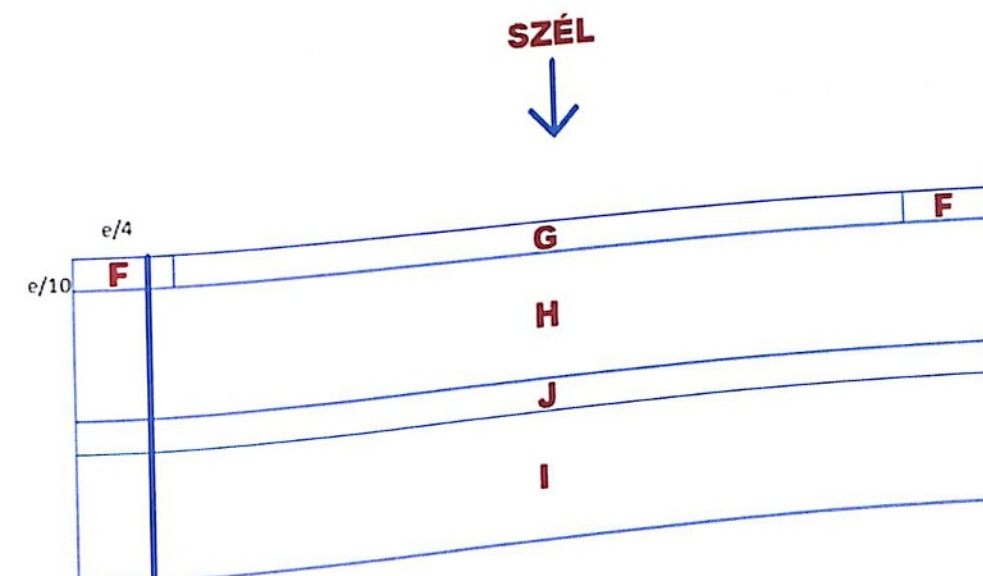
$$P_{sk} := t \cdot s = 0.8 \frac{\text{kN}}{\text{m}}$$

$$A := 400 \text{ m} \quad t := 1.0 \text{ m}$$

$$\mu := 0.8 \quad 0^\circ < \alpha < 30^\circ$$

$$\alpha := 7$$

Szélteher (merőleges a szeltámadta felületre)



$$e := 2 \cdot h = 2 \cdot 8995 = 17.99 \quad m$$

$$\frac{e}{10} = 1.799 \quad m$$

$$\frac{e}{4} = 4.495 \quad m$$

$$t = 1.00 \quad m$$

$$\alpha := 7 \quad t := 1.0 \quad m$$

"F" zóna

$$5^\circ \quad cpe_{10} := -2.5$$

$$15^\circ \quad cpe_{10'} := -2$$

$$cpe_1 := \left(\frac{cpe_{10'} - cpe_{10}}{15 - 5} \right) \cdot (\alpha - 5) + cpe_{10} = -2.4$$

$$P_{weszFk'} := t \cdot qp_z \cdot cpe_1 = -0.709 \quad \frac{kN}{m}$$

$$5^\circ \quad cpe_{.10} := 0$$

$$15^\circ \quad cpe_{.10'} := 0.2$$

$$cpe_2 := \left(\frac{cpe_{.10'} - cpe_{.10}}{15 - 5} \right) \cdot (\alpha - 5) + cpe_{.10} = 0.04$$

$$P_{wenyFk} := t \cdot qp_z \cdot cpe_2 = 0.012 \quad \frac{kN}{m}$$

"H" zóna

$$\alpha := 7 \quad t := 1.0 \text{ m}$$

$$5^\circ \quad cpe10 := -1.2$$

$$15^\circ \quad cpe10' := -0.3$$

$$cpe1 := \left(\frac{(cpe10' - cpe10)}{15 - 5} \right) \cdot (\alpha - 5) + (cpe10) = -1.02$$

$$P_{weszHk'} := t \cdot qpz \cdot cpe1 = -0.301 \quad \frac{kN}{m}$$

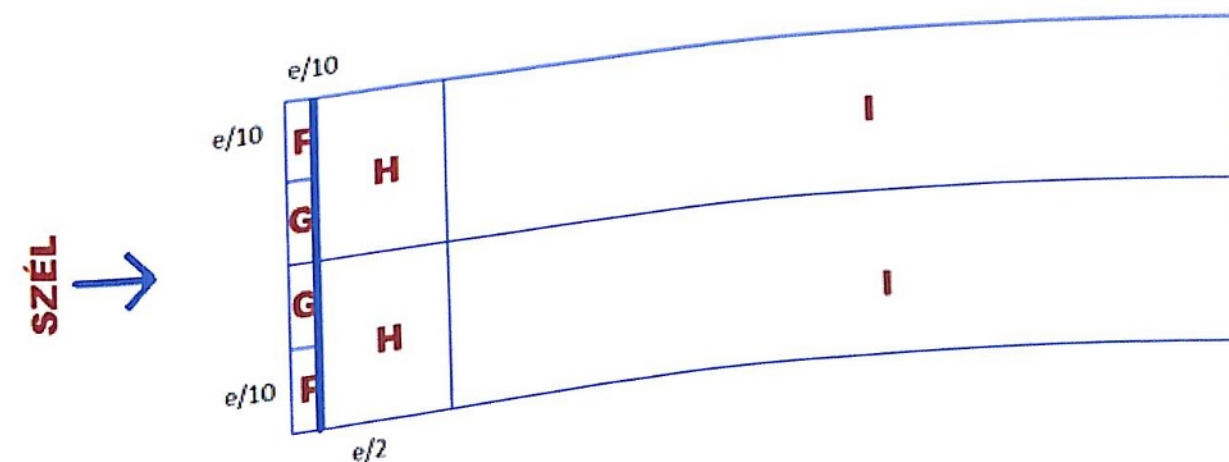
$$5^\circ \quad cpe.10 := 0$$

$$15^\circ \quad cpe.10' := 0.2$$

$$cpe2 := \left(\frac{(cpe.10' - cpe.10)}{15 - 5} \right) \cdot (\alpha - 5) + (cpe.10) = 0.04$$

$$P_{wenyHk} := t \cdot qpz \cdot cpe2 = 0.012 \quad \frac{kN}{m}$$

Hosszirányú - Külső



"F" zóna

$$\alpha := 7 \quad t := 1.0 \text{ m}$$

$$5^\circ \quad cpe10 := -2.2$$

$$15^\circ \quad cpe10' := -2.0$$

$$cpe1 := \left(\frac{(cpe10' - cpe10)}{15 - 5} \right) \cdot (\alpha - 5) + (cpe10) = -2.16$$

$$P_{weszFk'} := t \cdot qpz \cdot cpe1 = -0.638 \quad \frac{kN}{m}$$

"G" zóna

$$\alpha := 7 \quad t := 1.0 \quad m$$

$$5^\circ \quad cpe10 := -2.0$$

$$15^\circ \quad cpe10' := -2.0$$

$$cpe1 := \left(\frac{(cpe10' - cpe10)}{15 - 5} \right) \cdot (\alpha - 5) + (cpe10) = -2$$

$$PweszGk' := t \cdot qpz \cdot cpe1 = -0.591 \quad \frac{kN}{m}$$

Teherkombinációk

$$\gamma_{gsup} := 1.35 \quad \gamma_{ginf} := 1.0 \quad \psi_{0w} := 0.6$$

$$\gamma_s := 1.5$$

$$\gamma_w := 1.5$$

1. ULS önsúly+hó+szélnyomás

$$Pu1 := (\gamma_{gsup} \cdot Pgk) + (\gamma_s \cdot Psk) + (\psi_{0w} \cdot \gamma_w \cdot PwenzFk) = 1.305 \quad \frac{kN}{m}$$

1. SLS

$$Ps2 := (Pgk) + (Ps_k) + (\psi_{0w} \cdot PwenzFk) = 0.877 \quad \frac{kN}{m}$$

2. ULS önsúly+ szélszívás (KK)

$$Pu3f := (\gamma_{ginf} \cdot Pgk) + (\gamma_w \cdot PweszFk') = -0.994 \quad \frac{kN}{m}$$

$$Pu3h := (\gamma_{ginf} \cdot Pgk) + (\gamma_w \cdot PweszHk') = -0.382 \quad \frac{kN}{m}$$

2. SLS

$$Ps4f := (Pgk) + (PweszFk') = -0.639 \quad \frac{kN}{m}$$

$$Ps4h := (Pgk) + (PweszHk') = -0.231 \quad \frac{kN}{m}$$

3. ULS önsúly+ szélszívás

$$Pu5f := (\gamma_{ginf} \cdot Pgk) + (\gamma_w \cdot PweszFk') = -0.887 \quad \frac{kN}{m}$$

$$Pu5g := (\gamma_{ginf} \cdot Pgk) + (\gamma_w \cdot PweszGk') = -0.817 \quad \frac{kN}{m}$$

3. SLS

$$Ps6f := (Pgk) + (PweszFk') = -0.568 \quad \frac{kN}{m}$$

$$Ps6g := (Pgk) + (PweszGk') = -0.521 \quad \frac{kN}{m}$$

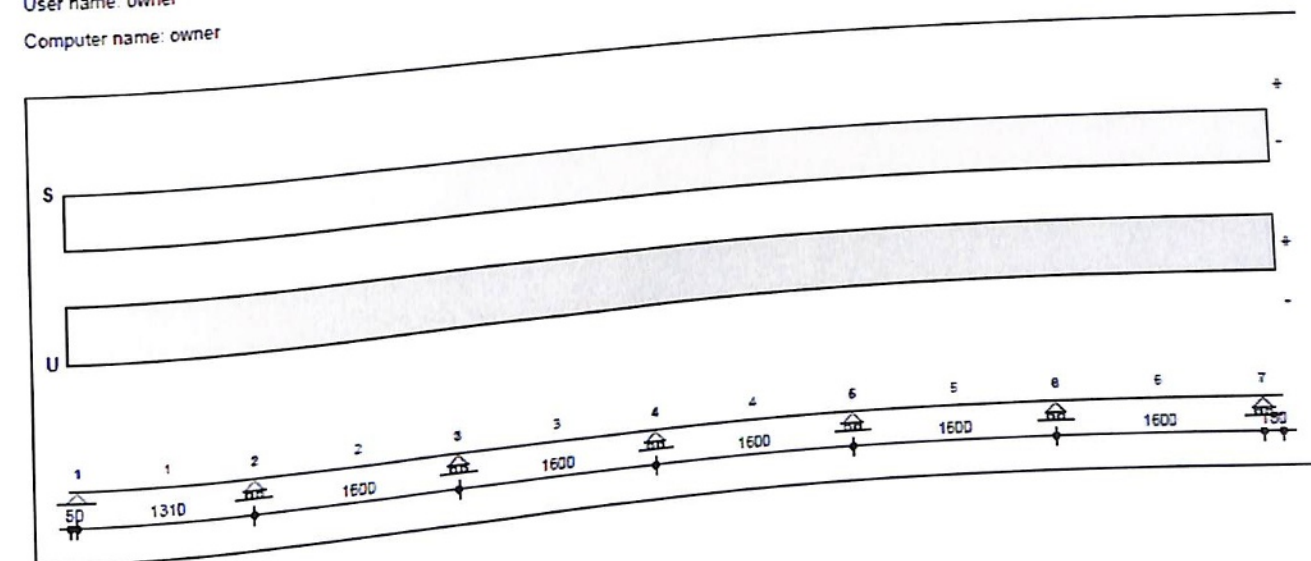
Dimroof

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Date: 3/24/2015

User name: owner

Computer name: owner



Alapbeállítások

Funkció: Tető

Ország: Magyarország

Szerkezeti beállítások

Szelvény: LTP 45

Statikai rendszer: Folytatólagos

Elhelyezés: Szokásos (keskeny)

Átfedés: 1 hullám

Csavar

Táv:

Lehajlás-számítás paraméterei

Lehajlási határ (nyílás): $L / 200$

Lehajlási határ (konzol): $L / 150$

Lehajlás típus: Maximális

Eredmények

ULS: 50%

SLS: 53%

Szabvány: Eurocode

Geometria						Nyílások			Csuklók
No.	Pozíció [mm]	Típus	Támaszok Szélesség [mm]	L1 [mm]	L2 [mm]	Méret [mm]	Vastagság 1 [mm]	Vastagság 2 [mm]	Pozíció [mm]
1	0	C	50	50	-	1310	0.4	-	
2	1310	C	1600	-	-	1600	0.4	-	
3	2910	C	1600	-	-	1600	0.4	-	
4	4510	C	1600	-	-	1600	0.4	-	
5	6110	C	1600	-	-	1600	0.4	-	
6	7710	C	1600	-	150				
7	9310	C	50	-					

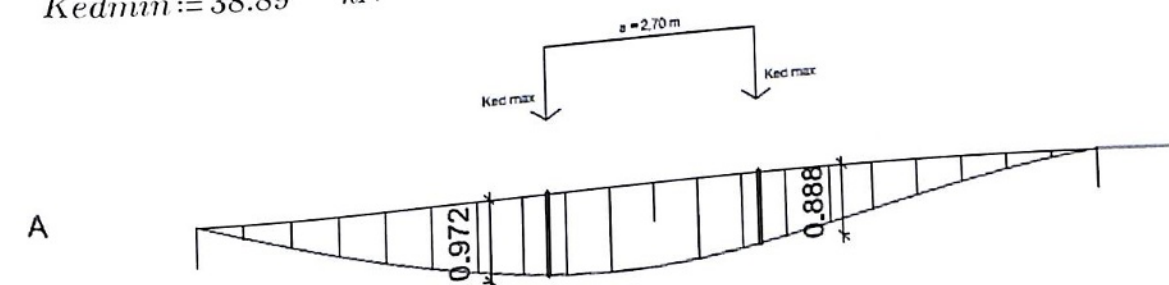
Terhek							ULS/SLS
No.	Típus	Kezdőpont	Végpont	Kezdőint.	Végint.	Szélesség	ULS
1	U			1.31			SLS
2	U			-0.88			

Reakció Erők

Függőleges

$$\begin{aligned} Ked_{max} &:= 85.61 \text{ kN} \\ Ked_{min} &:= 38.89 \text{ kN} \end{aligned}$$

daru számításból kell áthozni



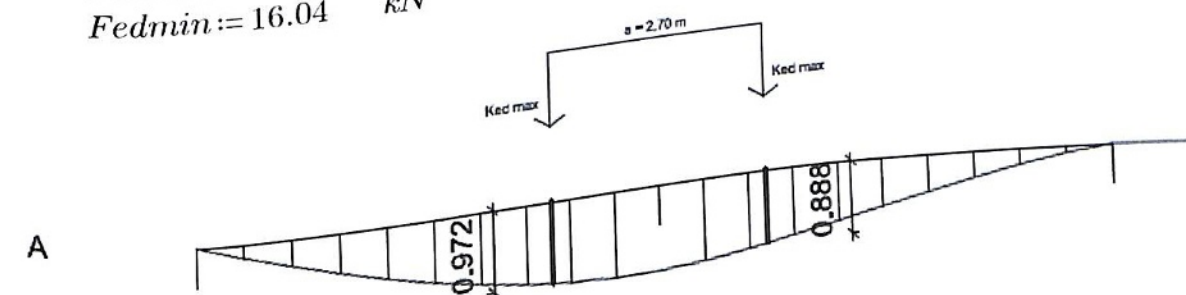
$$Ay1' := 0.972 \cdot Ked_{max} + 0.888 \cdot Ked_{max} = 159.235 \text{ kN}$$

$$Ay2' := 0.972 \cdot Ked_{min} + 0.888 \cdot Ked_{min} = 70.196 \text{ kN}$$

Vízszintes

$$\begin{aligned} Fed_{max} &:= 35.029 \text{ kN} \\ Fed_{min} &:= 16.04 \text{ kN} \end{aligned}$$

daru számításból kell áthozni



$$Ax1' := 0.972 \cdot Fed_{max} + 0.888 \cdot Fed_{max} = 65.154 \text{ kN}$$

$$Ax2' := 0.972 \cdot Fed_{min} + 0.888 \cdot Fed_{min} = 29.834 \text{ kN}$$

Első-Másodrendű számítás

$$\begin{aligned} H1 &:= -151.51 & V1 &:= -374.4 & e1 &:= -14.767 \\ H2 &:= 13.25 & V2 &:= -305.61 & e2 &:= -1.056 \end{aligned}$$

$$Hed := (H1 + H2) = -138.26$$

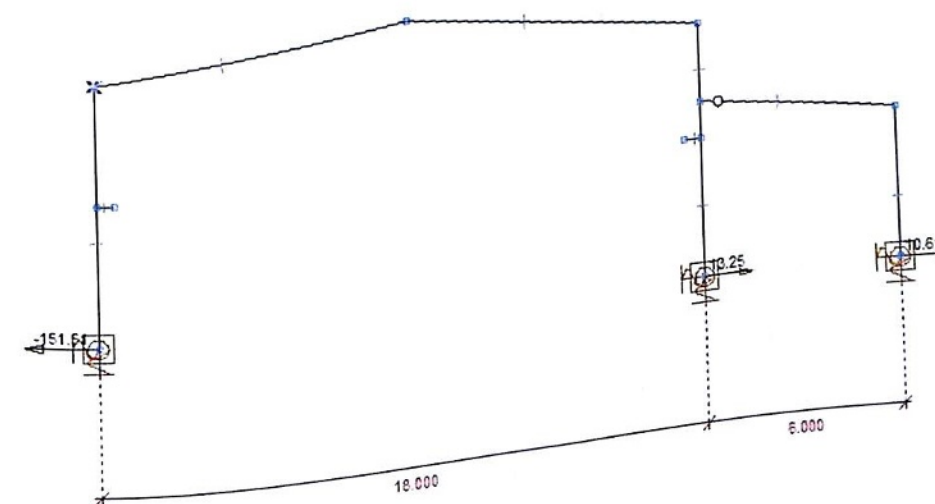
$$Ved := V1 + V2 = -680.01 \quad kN$$

$$\delta hed := \frac{e1 + e2}{2} = -7.912$$

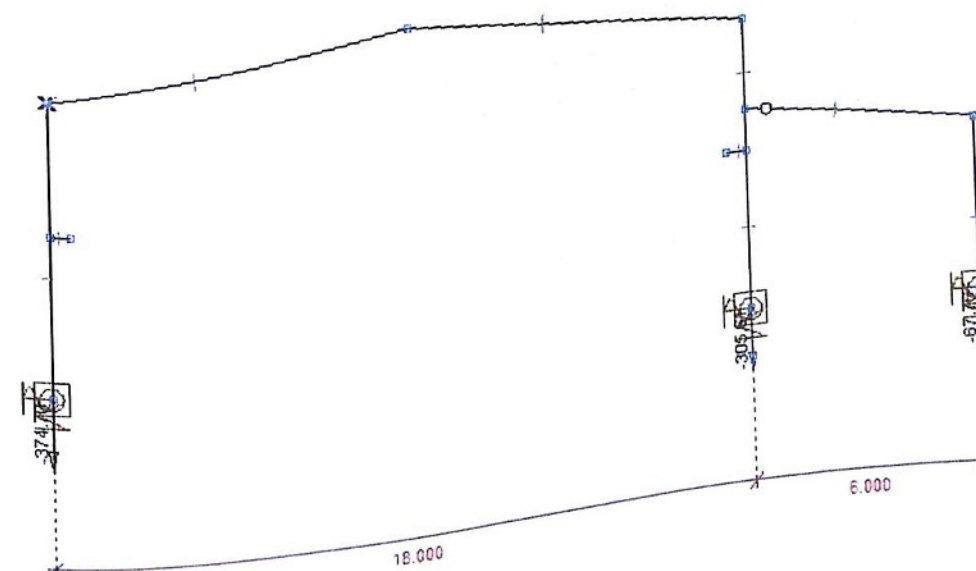
$$h := 7400 \quad m$$

$$\alpha cr := \left(\frac{Hed}{Ved} \right) \cdot \left(\frac{h}{\delta hed} \right) = -190.175 \quad \alpha cr > 10 \quad \text{rugalmas számítás esetén}$$

H1H2



V1V2



Geometriai Imperfekció (pontatlanság)

$$Hed \geq 0.15 \cdot Ved$$

$$Hed = -138.26 \quad 0.15 \cdot Ved = -102.002$$

Megfelel

Lehajlás ellenőrzése

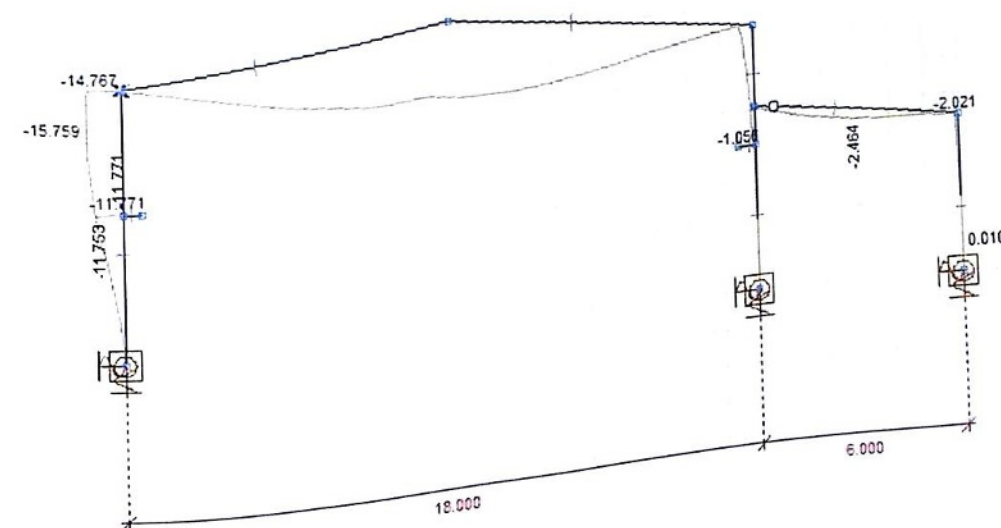
$$L := 18000 \text{ mm} \quad H := 7400 \text{ mm}$$

$$ez := \frac{L}{300} = 60 \text{ mm} \quad ezt := 33.396 \text{ mm}$$

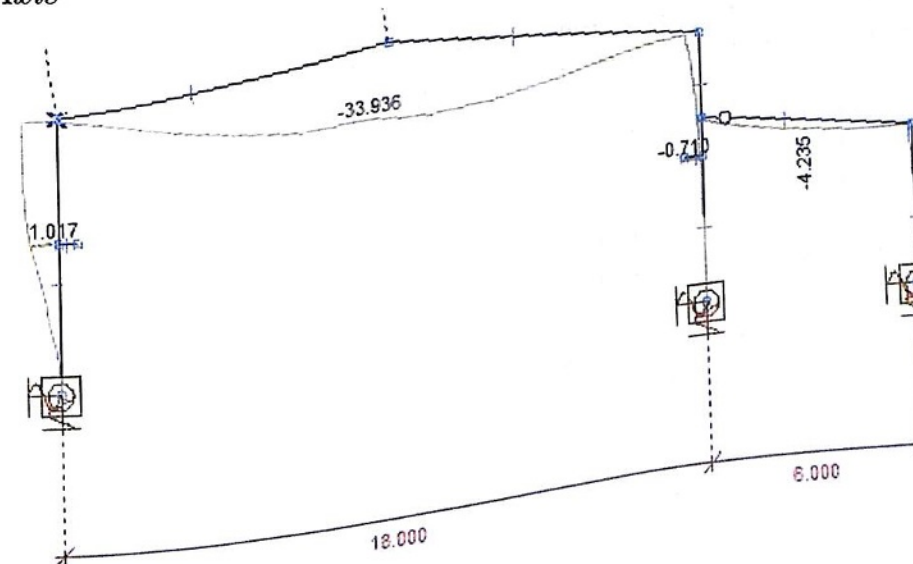
$$ex := \frac{H}{150} = 49.333 \text{ mm} \quad ext := 15.795 \text{ mm}$$

Megfelel

ex Axis



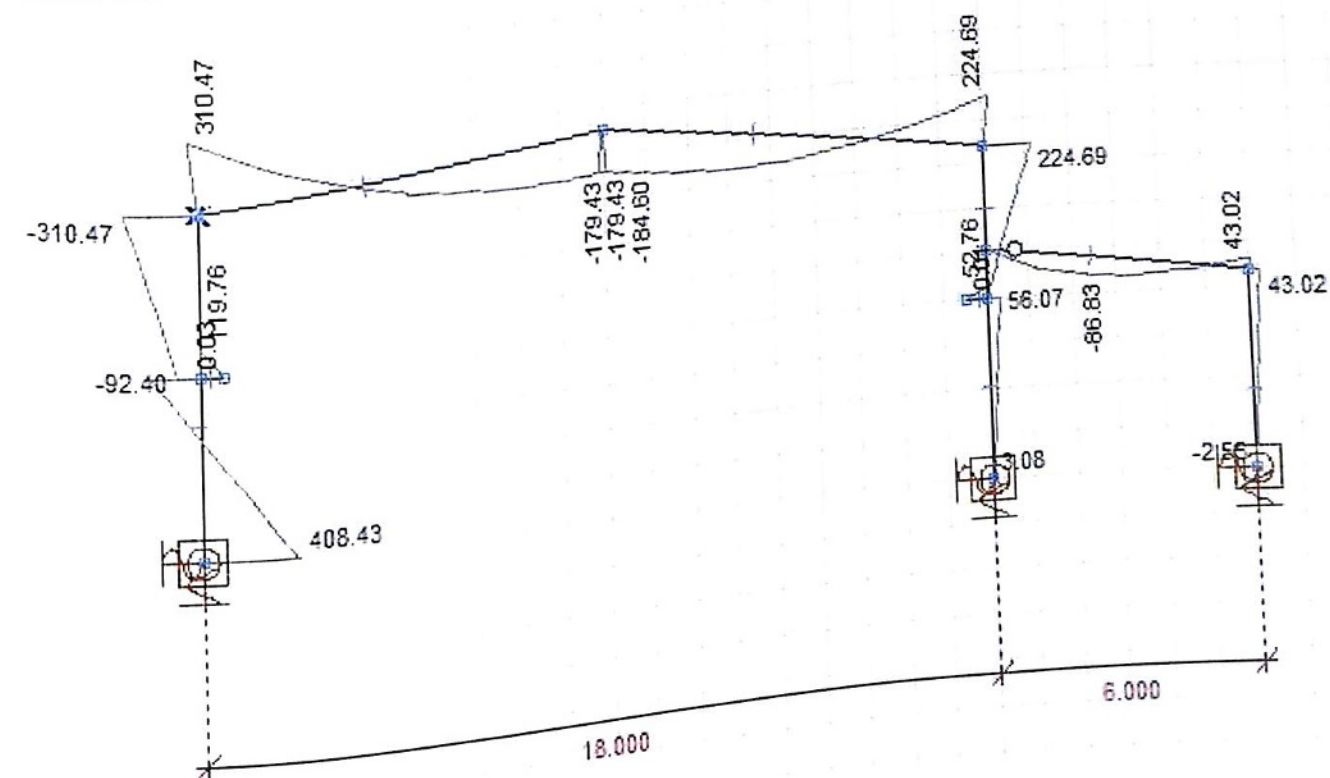
ez Axis



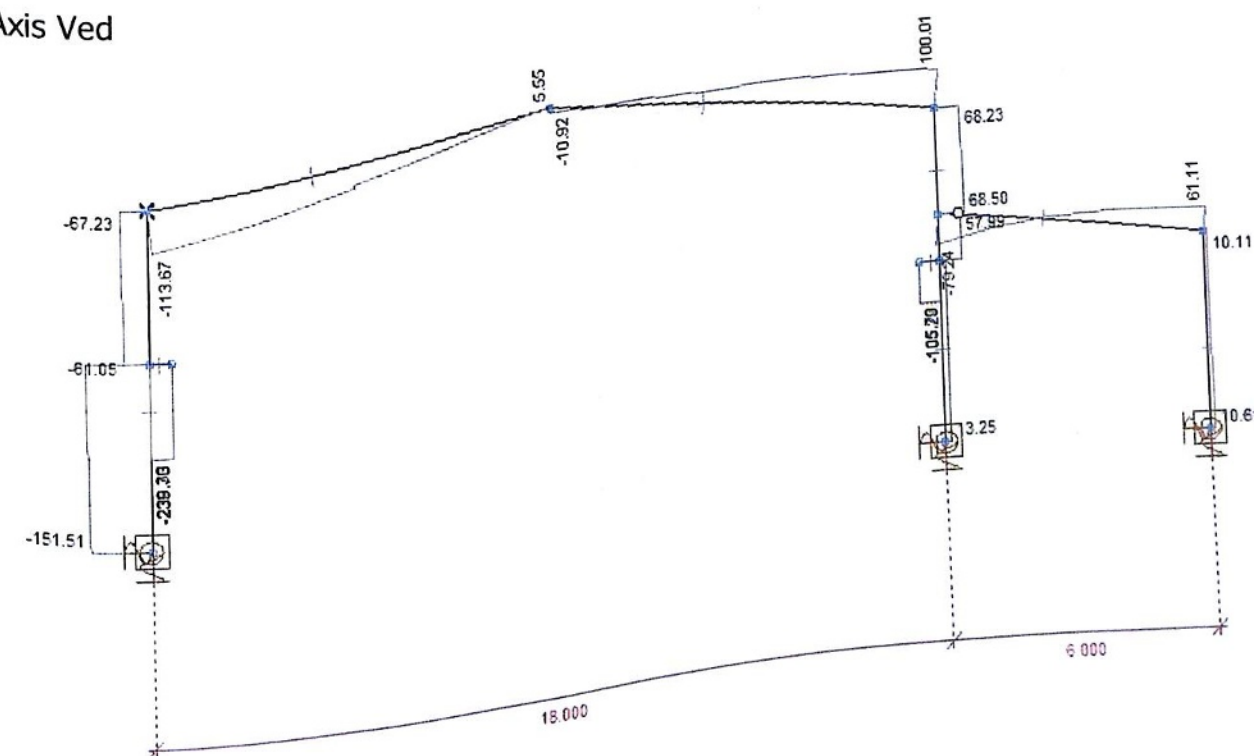
IV. FŐTARTÓ KERET VIZSGÁLATAI

Keresztmetszet Ellenőrzések

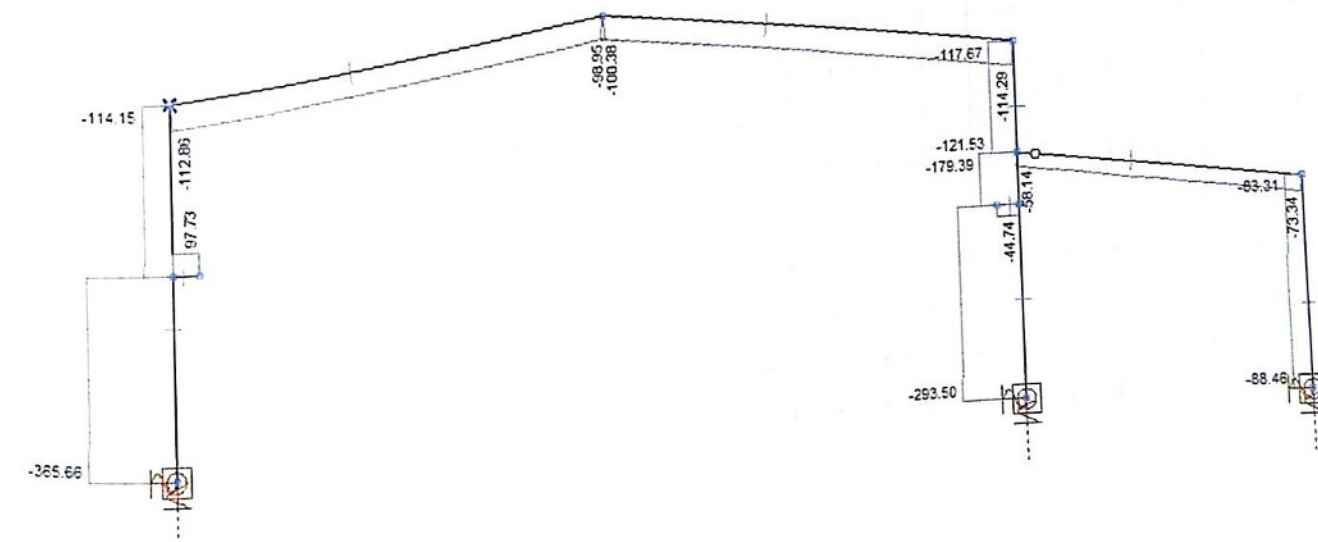
Axis Med



Axis Ved



Axis Ned



**Oszlop
HEA400**

$$\lambda_e := 93.9$$

$$\gamma_{m1} := 1.0$$

$$f_y := 235 \frac{N}{mm^2}$$

$$\gamma_{m0} := 1.0$$

$$A_o := 15.90$$

$$h_{wo} := 315 \text{ mm}$$

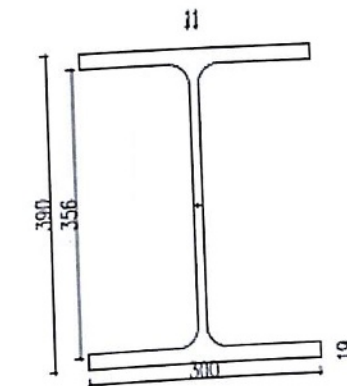
$$b_o := 300 \text{ mm}$$

$$W_{pl} := 2.562$$

$$A_{vo} := 5.733$$

$$t_{wo} := 11 \text{ mm}$$

$$t_{fo} := 19 \text{ mm}$$



Ellenőrzés nyomásra

$$N_{crdo} := \frac{A_o \cdot f_y}{\gamma_{m0}} = 3736.5 \text{ kN}$$

$$N_{ed1} := 374.7 \text{ kN}$$

$$N_{ed2} := 122.73 \text{ kN} \quad \text{MEGFELEL!}$$

Ellenőrzés tiszta hajlítás

$$M_{crdo} := \frac{W_{pl} \cdot f_y}{\gamma_{m0}} = 602.07 \text{ kNm}$$

$$M_{ed1} := 408.43 \text{ kNm} \quad \text{MEGFELEL!}$$

$$M_{ed2} := 310.47 \text{ kNm}$$

Ellenőrzés nyírásra

$$\frac{h_{wo}}{t_{wo}} = 28.636 \leq \frac{72 \cdot \varepsilon}{\eta} = 60$$

Nyírási horpadást nem kell vizsgálni

$$\varepsilon := 1 \quad \eta := 1.2$$

$$V_{crdo} := \frac{A_{vo} \cdot f_y}{\sqrt{3} \cdot \gamma_{m0}} = 777.838 \text{ kN}$$

$$V_{ed1} := 151.51 \text{ kN}$$

$$V_{ed2} := 67.23 \text{ kN} \quad \text{MEGFELEL!}$$

Hajítás + Nyírás kölcsönhatása

$$V_{ed} < \frac{V_{crd}}{2}$$

$$V_{ed} := 151.51 \text{ kN}$$

1 pontban

$$\frac{V_{crdo}}{2} = 388.919 \text{ kN}$$

MEGFELEL!

2 pontban

$$V_{ed} := 67.23 \text{ kN}$$

Normálerő hatását nem kell figyelembe venni.

ha:

$$N_{ed} < 0.25 \cdot N_{crd}$$

$$N_{ed} := 374.4 \text{ kN}$$

1 pontban

$$0.25 \cdot N_{crdo} = 934.125 \text{ kN}$$

MEGFELEL!

$$N_{ed} := 122.73 \text{ kN}$$

2 pontban

Nem kell figyelembe venni

**Gerenda
IPE500**

$$\lambda_e := 93.9$$

$$f_y := 235 \frac{N}{mm^2}$$

$$A_g := 11.15$$

$$h_{wg} := 468 \text{ mm}$$

$$b_g := 200 \text{ mm}$$

$$W_{pl} := 2.194$$

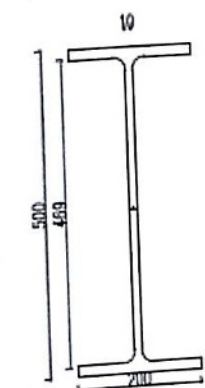
$$A_{vg} := 5.987$$

$$t_{wg} := 10.2 \text{ mm}$$

$$t_{fg} := 16 \text{ mm}$$

$$\gamma_{m1} := 1.0$$

$$\gamma_{m0} := 1.0$$



Ellenőrzés nyomásra

$$N_{crdg} := \frac{A_g \cdot f_y}{\gamma_{m0}} = 2620.25 \text{ kN}$$

$$N_{ed1} := 112.86 \text{ kN}$$

$$N_{ed2} := 100.38 \text{ kN} \quad \text{MEGFELEL!}$$

Ellenőrzés tiszta hajlítás

$$M_{crdg} := \frac{W_{pl} \cdot f_y}{\gamma_{m0}} = 515.59 \text{ kNm}$$

$$M_{ed1} := 310.47 \text{ kNm} \quad \text{MEGFELEL!}$$

$$M_{ed2} := 179.43 \text{ kNm}$$

Ellenőrzés nyírásra

$$\frac{h_{wg}}{t_{wg}} = 45.882 \leq \frac{72 \cdot \varepsilon}{\eta} = 60$$

Nyírási horpadást nem kell vizsgálni

$$V_{crdg} := \frac{A_{vg} \cdot f_y}{\sqrt{3} \cdot \gamma_{m0}} = 812.3 \text{ kN}$$

$$V_{ed1} := 113.78 \text{ kN}$$

$$V_{ed2} := 5.45 \text{ kN} \quad \text{MEGFELEL!}$$

Hajítás + Nyírás kölcsönhatása

$$V_{ed} < \frac{V_{crd}}{2}$$

$$V_{ed1} := 113.78 \text{ kN}$$

$$V_{ed2} := 5.45 \text{ kN}$$

$$\frac{V_{crdg}}{2} = 406.15 \text{ kN}$$

MEGFELEL!

Normálerő hatását nem kell figyelembe venni, ha:

$$N_{ed} < 0.25 \cdot N_{crd}$$

$$N_{ed1} := 112.86 \text{ kN}$$

$$N_{ed2} := 100.38 \text{ kN}$$

1 pontban

2 pontban

$$0.25 \cdot N_{crdg} = 655.063 \text{ kN}$$

MEGFELEL!

Nem kell figyelembe venni

Főtrató keret oszlop stabilitás vizsgálat

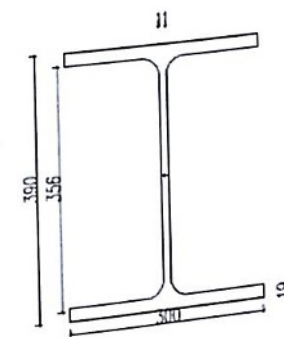
Anyagminőség és jellemzői S235

$$\lambda_e := 93.9 \quad \gamma_{m1} := 1.0$$

$$f_y := 235 \frac{N}{mm^2} \quad \gamma_{m0} := 1.0$$

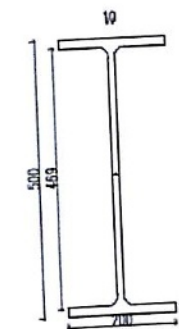
Oszlop
HEA400

$$\begin{aligned} I_{yo} &:= 45.070 \\ i_{yo} &:= 168.84 \text{ mm} \\ i_{zo} &:= 73.4 \text{ mm} \\ A_{yo} &:= 15.90 \\ A_{zo} &:= 15.90 \\ h_o &:= 352 \text{ mm} \\ b_o &:= 300 \text{ mm} \\ t_{wo} &:= 11 \text{ mm} \\ t_{fo} &:= 19 \text{ mm} \\ r_o &:= 27 \text{ mm} \end{aligned}$$



Gerenda
IPE500

$$\begin{aligned} I_{yg} &:= 48.200 \\ i_{yg} &:= 204.3 \text{ mm} \\ i_{zg} &:= 44 \text{ mm} \\ A_{yg} &:= 11.55 \\ A_{zg} &:= 11.55 \\ h_g &:= 468 \text{ mm} \\ b_g &:= 200 \text{ mm} \\ t_{wg} &:= 10.2 \text{ mm} \\ t_{fg} &:= 16 \text{ mm} \\ r_g &:= 21 \text{ mm} \end{aligned}$$



Oszlop vizsgálat

Kihajlás vizsgálat keret síkjában "y"

$$c := \frac{I_{yo} \cdot L}{I_{yg} \cdot H} = 2.274 \text{ mm} \quad c \leq 10 \text{ Megfelel}$$

$$\nu_y := \sqrt{1 + 0.35 \cdot c - 0.017 \cdot c^2} = 1.307$$

$$\lambda_y := \frac{\nu_y \cdot H}{i_{yo}} = 57.282 \quad \lambda := \frac{\lambda_y}{\lambda_e} = 0.61$$

$$L := 18000 \text{ mm} \quad H := 7400 \text{ mm}$$

"a" görbe alapján

$$\begin{aligned} \lambda_1 &:= 0.6 & a_1 &:= 0.8900 \\ \lambda_2 &:= 0.7 & a_2 &:= 0.8477 \end{aligned}$$

$$\chi_y := \left(\frac{(a_2 - a_1)}{(\lambda_2 - \lambda_1)} \cdot (\lambda - \lambda_1) \right) + a_1 = 0.8858$$

$$N_{brdy} := \frac{\chi_y \cdot A_{yo} \cdot f_y}{\gamma_{m1}} = 3309.634 \text{ kN}$$

$$\begin{aligned} N_{ed1} &:= 374.70 \text{ kN} \\ N_{ed2} &:= 122.73 \text{ kN} \end{aligned}$$

Megfelel

Kihajlás vizsgálat keret síkjára merőleges "z"

- oszlop felső szakasza

$$\nu_z := 1 \quad l := 1600 \quad mm$$

$$\lambda_z := \frac{\nu_z \cdot l}{i_{zo}} = 21.798$$

$$\lambda := \frac{\lambda_z}{\lambda_e} = 0.232$$

"b" görbe alapján

$$\begin{aligned} \lambda_1 &:= 0.2 & b_1 &:= 1 \\ \lambda_2 &:= 0.3 & b_2 &:= 0.9641 \end{aligned}$$

$$\chi_z := \left(\frac{(b_2 - b_1)}{(\lambda_2 - \lambda_1)} \cdot (\lambda - \lambda_1) \right) + b_1 = 0.9885$$

$$N_{brdz} := \frac{\chi_z \cdot A_{zo} \cdot f_y}{\gamma_{m1}} = 3693.381 \quad \frac{kN}{Megfelel} \quad Ned := 122.73 \quad kN$$

- oszlop alsó szakasza

$$\nu_z := 0.7 \quad l := 1600 \quad mm$$

$$\lambda_z := \frac{\nu_z \cdot l}{i_{zo}} = 15.259$$

$$\lambda := \frac{\lambda_z}{\lambda_e} = 0.163$$

"b" görbe alapján

$$\begin{aligned} \lambda_1 &:= 0.2 & b_1 &:= 1 \\ \lambda_2 &:= 0.3 & b_2 &:= 0.9641 \end{aligned}$$

$$\chi_z := \left(\frac{(b_2 - b_1)}{(\lambda_2 - \lambda_1)} \cdot (\lambda - \lambda_1) \right) + b_1 = 1.0135$$

$$N_{brdz} := \frac{\chi_z \cdot A_{zo} \cdot f_y}{\gamma_{m1}} = 3786.801 \quad \frac{kN}{Megfelel} \quad Ned := 374.40 \quad kN$$

Kifordulás vizsgálat (övmerevség)

$$k_{fl} := 1.1$$

$$M_1 := 168.33$$

$$M_2 := 310.47$$

$$\psi := \frac{M_1}{M_2} = 0.542$$

$$\lambda_{co} := 0.5$$

$$k_c := \frac{1}{1.33 - 0.33 \cdot \psi} = 0.869$$

$$L_c := 1600$$

$$I_{eff} := \frac{two^3 \cdot \left(\frac{ho}{6} \right)}{12 \cdot ho} + \frac{bo^3 \cdot tfo}{12} = 4.276 \cdot 10^7$$

Acélszerkezetű csarnok méretezése

$$A_{eff} := b_o \cdot t_{fo} + \frac{h_o}{6} \cdot t_{wo} = 6.345 \cdot 10^3$$

$$i_{fz} := \sqrt{\frac{I_{eff}}{A_{eff}}} = 82.087$$

$$M_{ed} := 310.47 \text{ kNm}$$

- kifordulás vizsgálat nélküli MEGFELELŐSSÉG feltétele

$$\lambda_f := \frac{k_c \cdot L_c}{i_{fz} \cdot \lambda_e} = 0.18 \quad \square < \square \quad \frac{M_{crdo}}{M_{ed}} \cdot \lambda_{co} = 0.97$$

$$\text{Tehát} \quad \chi := 1$$

- kifordulási ellenállás

$$M_{brd} := k_{fl} \cdot \chi \cdot M_{crdo} = 662.277 \text{ kNm} \quad \square > \square \quad M_{ed} = 310.47 \text{ kNm}$$

Megfelel

Kihajlás és kifordulás Interakciója

$$c_{my} := 0.9 \quad \text{kilendülő keret}$$

$$N_{ed} := 374.40$$

$$M_{ed} := 310.47$$

$$k_{yy} := c_{my} \cdot \left(1 + (\lambda_y - 0.2) \cdot \frac{N_{ed} \cdot \gamma_{m1}}{\chi_y \cdot A_o \cdot f_y} \right) = 6.712$$

$$k_{yy} := c_{my} \cdot \left(1 + \left(0.8 \cdot \frac{N_{ed} \cdot \gamma_{m1}}{\chi_y \cdot A_o \cdot f_y} \right) \right) = 0.981$$

$$k_{zy} := 0.6 \cdot k_{yy} = 0.589$$

$$\eta := \frac{N_{ed}}{N_{brdy}} + k_{yy} \cdot \frac{M_{ed}}{M_{brd}} = 0.573 \quad \eta \leq 1 \quad \text{Megfelel}$$

$$\eta := \frac{N_{ed}}{N_{brdz}} + k_{zy} \cdot \frac{M_{ed}}{M_{brd}} = 0.375 \quad \eta \leq 1 \quad \text{Megfelel}$$

Gerenda vizsgálat

Kihajlás vizsgálat keret síkjában "y"

$$\nu_y := 1 \quad l := 18000 \text{ mm}$$

$$\lambda_y := \frac{\nu_y \cdot l}{i_{yg}} = 88.106$$

$$\lambda := \frac{\lambda_y}{\lambda_e} = 0.938$$

"a" görbe alapján

$$\begin{aligned} \lambda_1 &:= 0.9 & a_1 &:= 0.7339 \\ \lambda_2 &:= 1.00 & a_2 &:= 0.6656 \end{aligned}$$

$$\chi_y := \left(\frac{(a_2 - a_1)}{(\lambda_2 - \lambda_1)} \cdot (\lambda - \lambda_1) \right) + a_1 = 0.7077$$

$$N_{brdy} := \frac{\chi_y \cdot A_{yg} \cdot f_y}{\gamma_{m1}} = 1920.999 \quad \frac{kN}{Megfelel}$$

$$\begin{aligned} Ned &:= 112.86 \text{ kN} \\ Ned2 &:= 100.38 \text{ kN} \end{aligned}$$

Kihajlás vizsgálat keret síkjában "z"

$$\nu_z := 1 \quad l := 1600 \text{ mm}$$

$$\lambda_z := \frac{\nu_z \cdot l}{i_{zg}} = 36.364$$

$$\lambda := \frac{\lambda_z}{\lambda_e} = 0.387$$

"b" görbe alapján

$$\begin{aligned} \lambda_1 &:= 0.3 & b_1 &:= 0.9641 \\ \lambda_2 &:= 0.4 & b_2 &:= 0.9261 \end{aligned}$$

$$\chi_z := \left(\frac{(b_2 - b_1)}{(\lambda_2 - \lambda_1)} \cdot (\lambda - \lambda_1) \right) + b_1 = 0.9309$$

$$N_{brdz} := \frac{\chi_z \cdot A_{zo} \cdot f_y}{\gamma_{m1}} = 3478.463 \quad \frac{kN}{Megfelel}$$

$$Ned := 112.86 \text{ kN}$$

Kifordulás vizsgálat (övmerevség)

$$k_{fl} := 1.1$$

$$k_c := 0.9$$

$$L_c := 1600$$

$$I_{eff} := \frac{twg^3 \cdot \left(\frac{hg}{6}\right)}{12} + \frac{bg^3 \cdot tfg}{12} = 1.067 \cdot 10^7$$

$$A_{eff} := bg \cdot tfg + \frac{hg}{6} \cdot twg = 3.996 \cdot 10^3$$

$$i_{fz} := \sqrt{\frac{I_{eff}}{A_{eff}}} = 51.685$$

$$\lambda_f := \frac{k_c \cdot L_c}{i_{fz} \cdot \lambda_e} = 0.297$$

"c" görbe alapján

$$\begin{aligned} \lambda_1 &:= 0.2 & c_1 &:= 1 \\ \lambda_2 &:= 0.3 & c_2 &:= 0.9491 \end{aligned}$$

$$\chi := \left(\frac{(c_2 - c_1)}{(\lambda_2 - \lambda_1)} \cdot (\lambda_f - \lambda_1) \right) + c_1 = 0.9508$$

- kifordulási ellenállás

$$M_{brd} := k_{fl} \cdot \chi \cdot M_{crdg} = 539.231 \text{ kNm} \quad \square > \square \quad \text{Megfelel} \quad Med := 310.47 \text{ kNm}$$

- Kifordulás vizsgálat nélküli MEGFELELŐSÉG feltétele

$$\lambda_f = 0.297 \quad \square < \square \quad \lambda_{co} \cdot \frac{M_{crdg}}{Med} = 0.83$$

Kihajlás és kifordulás Interakciója

$$c_{my} := 0.9 \quad \text{kilendülő keret}$$

$$k_{yy} := c_{my} \cdot \left(1 + (\lambda_y - 0.2) \cdot \frac{Ned \cdot \gamma_{m1}}{\chi_y \cdot Ag \cdot f_y} \right) = 2.881$$

$$k_{yy} := c_{my} \cdot \left(1 + \left(0.8 \cdot \frac{Ned \cdot \gamma_{m1}}{\chi_y \cdot Ag \cdot f_y} \right) \right) = 0.944$$

$$k_{zy} := 0.6 \cdot k_{yy} = 0.566$$

$$\eta := \frac{Ned}{N_{brdy}} + k_{yy} \cdot \frac{Med}{M_{brd}} = 0.602 \quad \eta \leq 1 \quad \text{Megfelel}$$

$$\eta := \frac{Ned}{N_{brdz}} + k_{zy} \cdot \frac{Med}{M_{brd}} = 0.358 \quad \eta \leq 1 \quad \text{Megfelel}$$

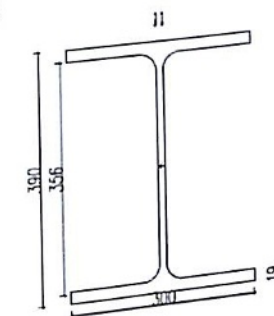
VII. Csomópontok méretezése

Gerenda-Gerenda kapcsolat

1. Csomópont

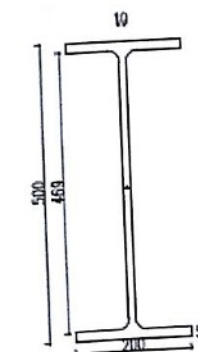
Oszlop
HEA400

$I_{yo} := 45.070$
 $i_{yo} := 168.84 \text{ mm}$
 $i_{zo} := 73.4 \text{ mm}$
 $A_{yo} := 15.90$
 $A_{zo} := 15.90$
 $h_o := 352 \text{ mm}$
 $b_o := 300 \text{ mm}$
 $t_{wo} := 11 \text{ mm}$
 $t_{fo} := 19 \text{ mm}$
 $r_o := 27 \text{ mm}$



Gerenda
IPE500

$I_{yg} := 48.200$
 $i_{yg} := 204.3 \text{ mm}$
 $i_{zg} := 44 \text{ mm}$
 $A_{yg} := 11.55$
 $A_{zg} := 11.55$
 $h_g := 468 \text{ mm}$
 $b_g := 200 \text{ mm}$
 $t_{wg} := 10.2 \text{ mm}$
 $t_{fg} := 16 \text{ mm}$
 $r_g := 21 \text{ mm}$



S235

$\gamma_{m0} := 1.0$
 $\gamma_{m2} := 1.25$

feltételezzük
M27 10.9 csavar

$f_y := 235 \frac{N}{mm^2}$
 $f_u := 360 \frac{N}{mm^2}$
 $f_{ub} := 1000 \frac{N}{mm^2}$
 $f_{yb} := 900 \frac{N}{mm^2}$

$$d := 2.9 \cdot t_{fo} \cdot \sqrt{\frac{f_y}{f_{ub}}} = 26.711$$

alkalmazott

$d_{alk} := 27 \text{ mm}$

$d_0 := 30 \text{ mm}$

$A := 573 \text{ mm}^2$

$A_s := 459 \text{ mm}^2$

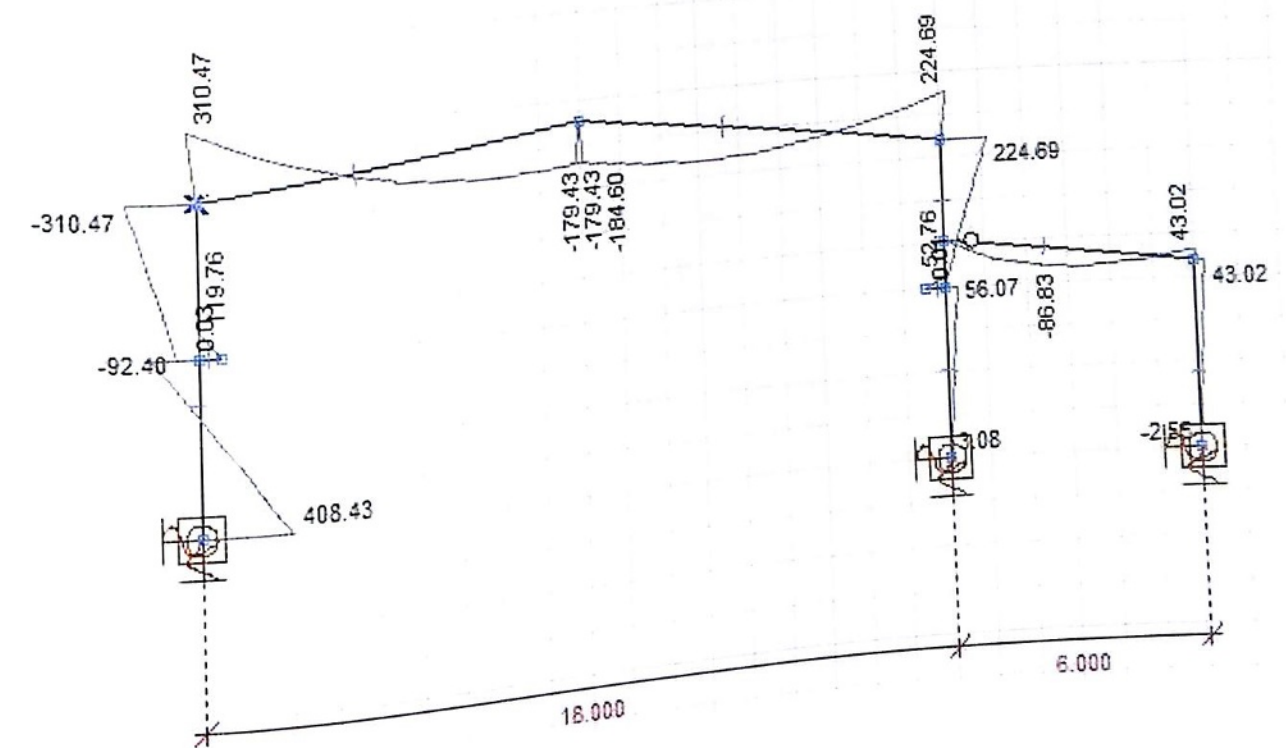
$d_m := 1.6 \cdot d_{alk} = 43.2 \text{ mm}$

$$t := 0.36 \cdot d_{alk} \cdot \sqrt{\frac{f_{ub}}{f_y}} = 20.051$$

alkalmazott

$t_{alk} := 25 \text{ mm}$

Med



Húzott csavarok ellenállása

Húzási ellenállás

$$F_{rdh} := \frac{\left(0.9 \cdot \frac{A_s \cdot f_{ub}}{\gamma_{m2}}\right)}{1000} = 330.48 \quad kN$$

Kigombolódási ellenállás

$$B_{rd} := \frac{\left(0.6 \cdot \frac{\pi \cdot d_m \cdot t_{fo} \cdot f_u}{\gamma_{m2}}\right)}{1000} = 445.585 \quad kN$$

Nyírási ellenállás $n := 1$

$$F_{rdny} := \frac{\left(n \cdot \frac{0.6 \cdot f_{ub} \cdot A}{\gamma_{m2}}\right)}{1000} = 275.04 \quad kN$$

Gerenda homloklemez ellenállás

- Gerenda öv alatti felső csavarsor ellenállás

$$e_a := 3.5 \cdot d_0 = 105 \quad w := 120 \quad mm$$

$$e_f := 2.5 \cdot d_0 = 75 \quad e := 55 \quad mm$$

$$a_c := \frac{tw_g}{2} = 5.1 \quad a_f := \frac{tf_g}{2} = 8$$

$$m := \frac{w}{2} - \frac{tw_g}{2} - 0.8 \cdot a_c \cdot \sqrt{2} = 49.13$$

$$m_2 := e_f - 0.8 \cdot a_f \cdot \sqrt{2} = 65.949$$

$$b_{alk} := 200 \quad mm$$

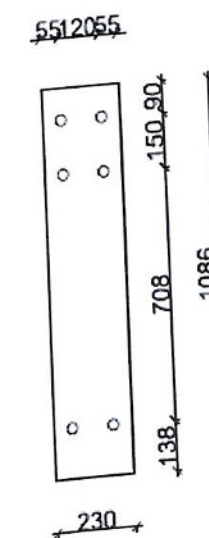
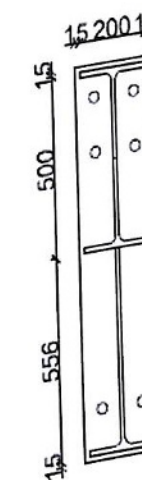
$$p := 3 \cdot m = 147.39$$

$$\lambda_1 := \frac{m}{m + e} = 0.472$$

$$\lambda_2 := \frac{m_2}{m + e} = 0.633$$

$$\alpha := 5.5$$

$$p_{alk} := 150 \quad mm$$



Nem kör alakú töréskép esetén

$$l_{eff1} := 0.5 \cdot palk + \alpha \cdot m - (2 \cdot m + 0.625 \cdot e) = 212.58 \text{ mm}$$

Kör alakú töréskép esetén

$$l_{eff2} := \pi \cdot m + palk = 304.346 \text{ mm}$$

Homloklemez nyomatéki teherbírása

$$M_{plrd} := \frac{\left(0.25 \cdot l_{eff1} \cdot t_{alk}^2 \cdot \frac{f_y}{\gamma_{m0}}\right)}{1000} = 7805.673 \text{ kNmm}$$

T elem húzó ellenállása

- T elem 1. tönkremeneteli módja

$$F_{1rd} := 4 \cdot \frac{M_{plrd}}{m} = 635.512 \text{ kN}$$

- T elem 2. tönkremeneteli módja $n := e$

$$F_{2rd} := \frac{2 \cdot M_{plrd} + e \cdot 2 \cdot F_{rdh}}{m + e} = 499.031 \text{ kN}$$

- T elem 3. tönkremeneteli módja

$$F_{3rd} := 2 \cdot F_{rdh} = 660.96 \text{ kN}$$

Legkisebb a mértékadó $F_{2rd} = 499.031 \text{ kN}$

- Gerenda öv alatti második (utolsó) csavarsor ellenállása

Nem kör alakú töréskép esetén

$$l_{eff1} := 2 \cdot m + 0.625 \cdot e + 0.5 \cdot palk = 207.635 \text{ mm}$$

Kör alakú töréskép esetén

$$l_{eff2} := \pi \cdot m + palk = 304.346 \text{ mm}$$

Homloklemez nyomatéki teherbírása

$$M_{plrd} := \frac{\left(0.25 \cdot l_{eff1} \cdot t_{alk}^2 \cdot \frac{f_y}{\gamma_{m0}}\right)}{1000} = 7624.098 \quad \text{kNmm}$$

- T elem 1. tönkremeneteli módja

$$F_{1rd'} := 4 \cdot \left(\frac{M_{plrd}}{m}\right) = 620.728 \quad \text{kN}$$

- T elem 2. tönkremeneteli módja

$$F_{2rd'} := \frac{2 \cdot M_{plrd} + e \cdot 2 \cdot F_{rdh}}{m + e} = 495.544 \quad \text{kN}$$

- T elem 3. tönkremeneteli módja

$$F_{3rd'} := 2 \cdot F_{rdh} = 660.96 \quad \text{kN}$$

Legkisebb a mértékadó

$$F_{2rd'} = 495.544 \quad \text{kN}$$

Ellenőrzés :

Teljes homloklemez hajlítási ellenállás

csavarok távolsága

$$z_1 := 963 \quad \text{mm}$$

$$z_2 := 813 \quad \text{mm}$$

$$M_{rd} := \frac{(F_{2rd} \cdot z_1 + F_{2rd'} \cdot z_2)}{1000} = 883.444 \quad \text{kNm}$$

$$M_{ed} = 310.47 \quad \text{kNm}$$

$M_{rd} \geq M_{ed}$
Megfelel

Oszlop övének ellenállása

- Felső (utolsó) csavarsor ellenőrzése $r_o = 27 \text{ mm}$ $w := 120$

$$m := \frac{w}{2} - \frac{t_{wo}}{2} - 0.8 \cdot r_o = 32.9 \text{ mm}$$

$$p_{alk} = 150 \text{ mm}$$

$$e = 55 \text{ mm} \quad e_a = 105 \text{ mm}$$

$$n := 1.25 \cdot m = 41.125 \text{ mm}$$

Nem kör alakú töréskép

$$l_{eff1'} := 4 \cdot m + 1.25 \cdot e = 200.35 \text{ mm}$$

$$l_{eff1''} := e_a + 2 \cdot m + 0.625 \cdot e = 205.175 \text{ mm}$$

$$l_{eff2'} := 0.5 \cdot p + 2 \cdot m + 0.625 \cdot e = 173.87 \text{ mm}$$

$$l_{eff2''} := e_a + 0.5 \cdot p = 178.695 \text{ mm}$$

Kör alakú töréskép nem mértékadó

Övlemez nyomatéki teherbírása

$$M_{plrd} := \frac{\left(0.25 \cdot l_{eff2'} \cdot t_{fo}^2 \cdot \frac{f_y}{\gamma_{m0}} \right)}{1000} = 3687.566 \text{ kNmm}$$

- T elem 1. tönkremeneteli módja

$$F_{1rd} := 4 \cdot \frac{M_{plrd}}{m} = 448.336 \text{ kN}$$

- T elem 2. tönkremeneteli módja

$$F_{2rd} := \frac{2 \cdot M_{plrd} + e \cdot 2 \cdot F_{rdh}}{m + e} = 497.474 \text{ kN}$$

- T elem 3. tönkremeneteli módja

$$F_{3rd} := 2 \cdot F_{rdh} = 660.96 \text{ kN}$$

Legkisebb a mértékadó

$$F_{2rd} = 497.474 \text{ kN}$$

- Gerenda öv alatti második (utolsó) csavarsor ellenállása

Nem kör alakú töréskép esetén

$$l_{eff1} := 0.5 \cdot p + 2 \cdot m + 0.625 \cdot e = 173.87 \text{ mm}$$

Homloklemez nyomatéki teherbírása

$$M_{plrd} := \frac{\left(0.25 \cdot l_{eff1} \cdot t_{fo}^2 \cdot \frac{f_y}{\gamma_{m0}}\right)}{1000} = 3687.566 \text{ kNmm}$$

- T elem 1. tönkremeneteli módja

$$F_{1rd'} := 4 \cdot \left(\frac{M_{plrd}}{m}\right) = 448.336 \text{ kN}$$

- T elem 2. tönkremeneteli módja

$$F_{2rd'} := \frac{2 \cdot M_{plrd} + e \cdot 2 \cdot F_{rdh}}{m + e} = 497.474 \text{ kN}$$

- T elem 3. tönkremeneteli módja

$$F_{3rd'} := 2 \cdot F_{rdh} = 660.96 \text{ kN}$$

Legkisebb a mértékadó

$$F_{1rd'} = 448.336 \text{ kN}$$

Teljes homloklemez hajlítási ellenállás

csavarok távolsága

$$z_1 = 963 \text{ mm}$$

$$z_2 = 813 \text{ mm}$$

$$M_{rd} := \frac{(F_{2rd'} \cdot z_1 + F_{1rd'} \cdot z_2)}{1000} = 843.564 \text{ kNm}$$

$$M_{ed} = 310.47 \text{ kNm}$$

$M_{rd} \geq M_{ed}$
Megfelel

Oszlop gerincének ellenállása

Nyomó ellenállás

Mértékadó nyomatékból
 $Med := 310.47$

Nyomaték karja
 $z := 778 \text{ mm}$

$$Fed := \frac{Med}{z} \cdot 1000 = 399.062 \text{ kN}$$

$$tfg = 16$$

$$tfo = 19$$

$$ro := 27$$

$$talk = 25$$

$$ap := 0.5$$

$$beff := tfg + 2 \cdot \sqrt{2} \cdot ap + 5 (tfo + ro) + talk = 272.414 \text{ mm}$$

Egyoldali oszlop -gerenda kapcsolat tehát

$$\beta := 1.0$$

$$Avo := Avo \cdot 1000 = 5733 \text{ mm}^2$$

Csökkentő tényező

$$\omega := \frac{1}{\sqrt{1 + 1.3 \left(\frac{(beff \cdot two)^2}{Avo} \right)}} = 0.859$$

Tengely irányú nyomófeszültségtől függő csökkentő tényező

$$Ned := 122.73$$

$$Ncrdo = 3736.5 \text{ kN}$$

$$do := 298 \text{ mm}$$

$$ho := 390 \text{ mm}$$

$$Med := 310.47$$

$$Mcrdo = 602.07 \text{ kNm}$$

$$kwc := 1.0$$

$$\sigma_{ed} := \left(\frac{Ned}{Ncrdo} + \frac{Med}{Mcrdo} \right) \cdot \frac{do}{ho} = 0.419$$

$$E := 210000 \frac{\text{N}}{\text{mm}^2}$$

Kerinclemez karcsúsága

$$\lambda_p := 0.932 \cdot \sqrt{\frac{beff \cdot do \cdot fy}{E \cdot two^2}} = 0.808$$

Tehát

$$\rho := \frac{(\lambda_p - 0.2)}{\lambda_p^2} = 0.932$$

Merevítés nélküli oszlop gereinc ellenállása

$$F_{rd} := \frac{\left(\frac{\rho \cdot \omega \cdot k_{wc} \cdot beff \cdot two \cdot fy}{\gamma_{mo}} \right)}{1000} = 563.556 \quad kN$$

Megfelel $F_{ed} = 399.062 \quad kN$

Húzó ellenállás

$$beff := leff2' + leff2'' = 352.565 \quad mm$$

$$A_{vo} := A_{vo} \cdot 1000 = 5733 \quad mm^2$$

Csökkentő tényező

$$\omega := \frac{1}{\sqrt{1 + 1.3 \left(\frac{(beff \cdot two)^2}{A_{vo}} \right)}} = 0.792$$

$$F_{rd} := \frac{\left(\frac{\omega \cdot beff \cdot two \cdot fy}{\gamma_{mo}} \right)}{1000} = 721.661 \quad kN$$

Megfelel $F_{ed} = 399.062 \quad kN$

Oszlop gerincének nyírási teherbírás

$$\varepsilon := 1$$

Oszlop gerince nyírási horpadásra megfelel ha:

$$\frac{d_o}{two} = 27.091 \quad \square < 69 \cdot \varepsilon$$

Megfelel

Gereinc nyírási ellenállás

$$V_{rd} := \frac{0.9 \cdot fy \cdot A_{vo}}{\sqrt{3} \cdot \gamma_{mo}} = 700.054 \quad kN$$

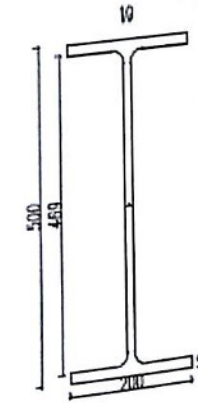
Megfelel $V_{ed} := 67.3 \quad kN$

Gerenda-Gerenda kapcsolat

2.Csomópont

Gerenda IPE500

$I_{yg} := 48.200$
 $i_{yg} := 204.3 \text{ mm}$
 $i_{zg} := 44 \text{ mm}$
 $A_{yg} := 11.55$
 $A_{zg} := 11.55$
 $h_g := 468 \text{ mm}$
 $b_g := 200 \text{ mm}$
 $t_{wg} := 10.2 \text{ mm}$
 $t_{fg} := 16 \text{ mm}$
 $r_g := 21 \text{ mm}$



S235

$\gamma_{m0} := 1.0$
 $\gamma_{m2} := 1.25$

feltételezzük
M24 10.9 csavar

$f_y := 235 \frac{N}{mm^2}$
 $f_u := 360 \frac{N}{mm^2}$
 $f_{ub} := 1000 \frac{N}{mm^2}$
 $f_{yb} := 900 \frac{N}{mm^2}$

$$d := 2.9 \cdot t_{fg} \cdot \sqrt{\frac{f_y}{f_{ub}}} = 22.493$$

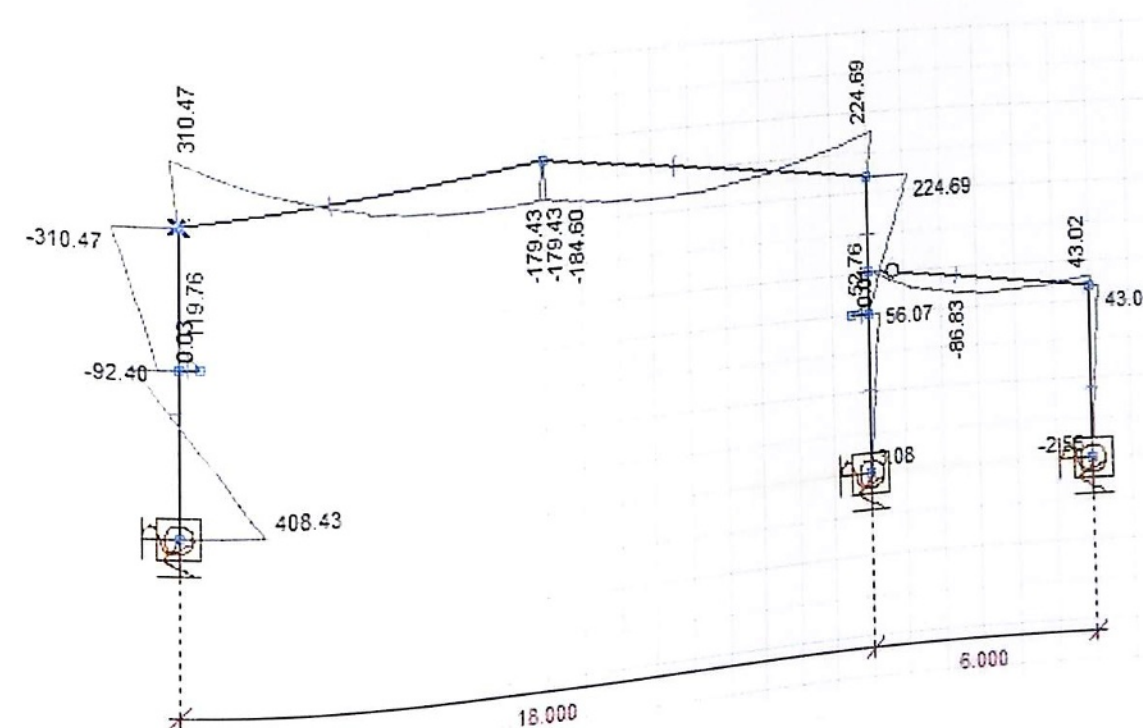
alkalmazott
 $d_{alk} := 24 \text{ mm}$

$d_0 := 26 \text{ mm}$
 $A := 452 \text{ mm}^2$
 $A_s := 353 \text{ mm}^2$
 $d_m := 1.6 \cdot d = 35.989 \text{ mm}$

$$t := 1.2 \cdot t_{fg} = 19.2$$

alkalmazott
 $t_{alk} := 20 \text{ mm}$

Med



Húzott csavarok ellenállása

Húzási ellenállás

$$F_{rdh} := \frac{\left(0.9 \cdot \frac{A_s \cdot f_{ub}}{\gamma_{m2}}\right)}{1000} = 254.16 \quad kN$$

Kigombolódási ellenállás

$$B_{rd} := \frac{\left(0.6 \cdot \frac{\pi \cdot d_m \cdot t_{fg} \cdot f_u}{\gamma_{m2}}\right)}{1000} = 312.597 \quad kN$$

Nyírási ellenállás $n := 2$

$$F_{rdny} := \frac{\left(n \cdot \frac{0.6 \cdot f_{ub} \cdot A}{\gamma_{m2}}\right)}{1000} = 433.92 \quad kN$$

Gerenda homloklemez ellenállás

- Gerenda öv feletti alsó csavarsor

$$e_a := 2 \cdot d_0 = 52 \quad w := 110 \quad mm$$

$$e_f := 2.5 \cdot d_0 = 65 \quad e := 60 \quad mm$$

$$a_c := \frac{t_{wg}}{2} = 5.1 \quad a_f := \frac{t_{fg}}{2} = 8$$

$$m := \frac{w}{2} - \frac{t_{wg}}{2} - 0.8 \cdot a_c \cdot \sqrt{2} = 44.13$$

$$m_2 := e_f - 0.8 \cdot a_f \cdot \sqrt{2} = 55.949$$

$$b_{alk} := 230 \quad mm$$

$$p := 3 \cdot m = 132.39$$

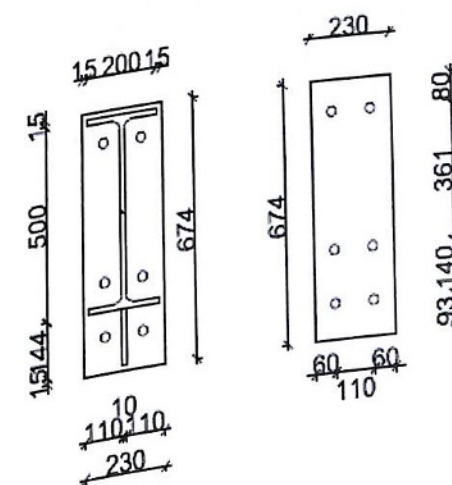
$$\lambda_1 := \frac{m}{m + e} = 0.424$$

$$\lambda_2 := \frac{m_2}{m + e} = 0.537$$

$$\alpha := 6.1$$

Nem kör alakú töréskép esetén

$$l_{eff} := \alpha \cdot m = 269.193 \quad mm$$



$$p_{alk} := 140 \quad mm$$

Homloklemez nyomatéki teherbírása

$$M_{plrd} := \frac{\left(0.25 \cdot l_{eff} \cdot t_{alk}^2 \cdot \frac{f_y}{\gamma_{m0}}\right)}{1000} = 6326.037 \quad \text{kNmm}$$

T elem 1. tönkremeneteli módja

$$F_{1rd} := 4 \cdot \left(\frac{M_{plrd}}{m}\right) = 573.4 \quad \text{kN}$$

T elem 2. tönkremeneteli módja

$$F_{2rd} := \frac{2 \cdot M_{plrd} + e \cdot 2 \cdot F_{rdh}}{m + e} = 414.398 \quad \text{kN}$$

T elem 3. tönkremeneteli módja

$$F_{3rd} := 2 \cdot F_{rdh} = 508.32 \quad \text{kN}$$

Legkisebb a mértékadó

$$F_{2rd} = 414.398 \quad \text{kN}$$

- Gerenda öv alatti (külső) csavarsor

$$l_{eff} := e_a + \alpha \cdot m - (2 \cdot m + 0.625 \cdot e) = 195.433 \quad \text{mm}$$

Homloklemez nyomatéki teherbírása

$$M_{plrd} := \frac{\left(0.25 \cdot l_{eff} \cdot t_{alk}^2 \cdot \frac{f_y}{\gamma_{m0}}\right)}{1000} = 4592.676 \quad \text{kNmm}$$

- T elem 1. tönkremeneteli módja

$$F_{1rd'} := 4 \cdot \left(\frac{M_{plrd}}{m}\right) = 416.286 \quad \text{kN}$$

- T elem 2. tönkremeneteli módja

$$F_{2rd'} := \frac{2 \cdot M_{plrd} + e \cdot 2 \cdot F_{rdh}}{m + e} = 381.106 \quad \text{kN}$$

- T elem 3. tönkremeneteli módja

$$F_{3rd'} := 2 \cdot F_{rdh} = 508.32 \quad \text{kN}$$

$$F_{2rd'} = 381.106 \quad \text{kN}$$

Legkisebb a mértékadó

Teljes homloklemez hajlítási ellenállás

csavarok távolsága

$$z1 := 550 \text{ mm} \quad z2 := 410 \text{ mm}$$

$$M_{rd} := \frac{(F2_{rd} \cdot z1 + F1_{rd} \cdot z2)}{1000} = 398.596 \text{ kNm}$$

$$M_{ed} := 179.43 \text{ kNm}$$

$M_{rd} \geq M_{ed}$
Megfelel

Gerenda-Oszlop kapcsolat mellékrajzánál

3.Csomópont

Oszlop

HEA400

$$\begin{aligned} t_w &:= 11 \text{ mm} \\ t_f &:= 19 \text{ mm} \\ r &:= 27 \text{ mm} \end{aligned}$$

S235

$$\gamma_{m0} := 1.0$$

$$\gamma_{m2} := 1.25$$

feltételezzük
M27 10.9 csavar

$$\begin{aligned} f_y &:= 235 \frac{N}{mm^2} & f_{ub} &:= 1000 \frac{N}{mm^2} \\ f_u &:= 360 \frac{N}{mm^2} & f_{yb} &:= 900 \frac{N}{mm^2} \end{aligned}$$

$$d := 2.9 \cdot t_f \cdot \sqrt{\frac{f_y}{f_{ub}}} = 26.711$$

alkalmazott

$$d_{alk} := 27 \text{ mm}$$

$$d_o := 30 \text{ mm}$$

$$A := 573 \text{ mm}^2$$

$$t := 0.36 \cdot d_{alk} \cdot \sqrt{\frac{f_{ub}}{f_y}} = 20.051$$

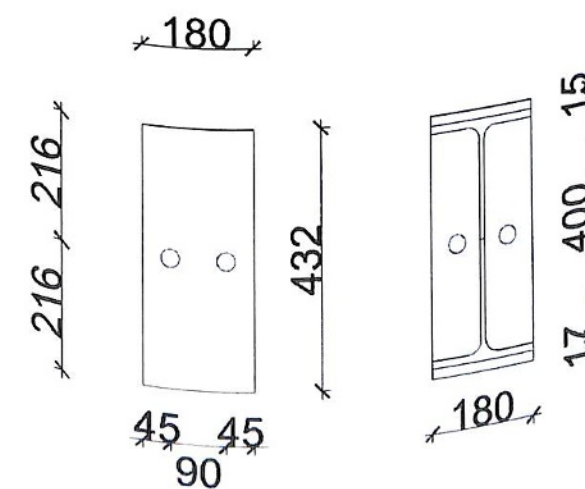
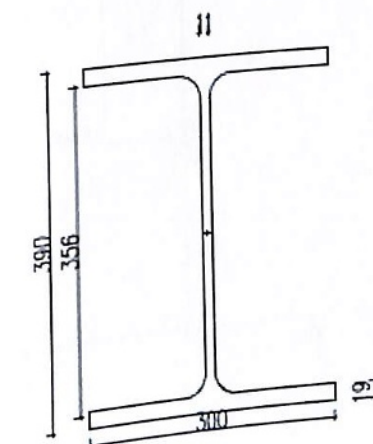
alkalmazott

$$t_{alk} := 25 \text{ mm}$$

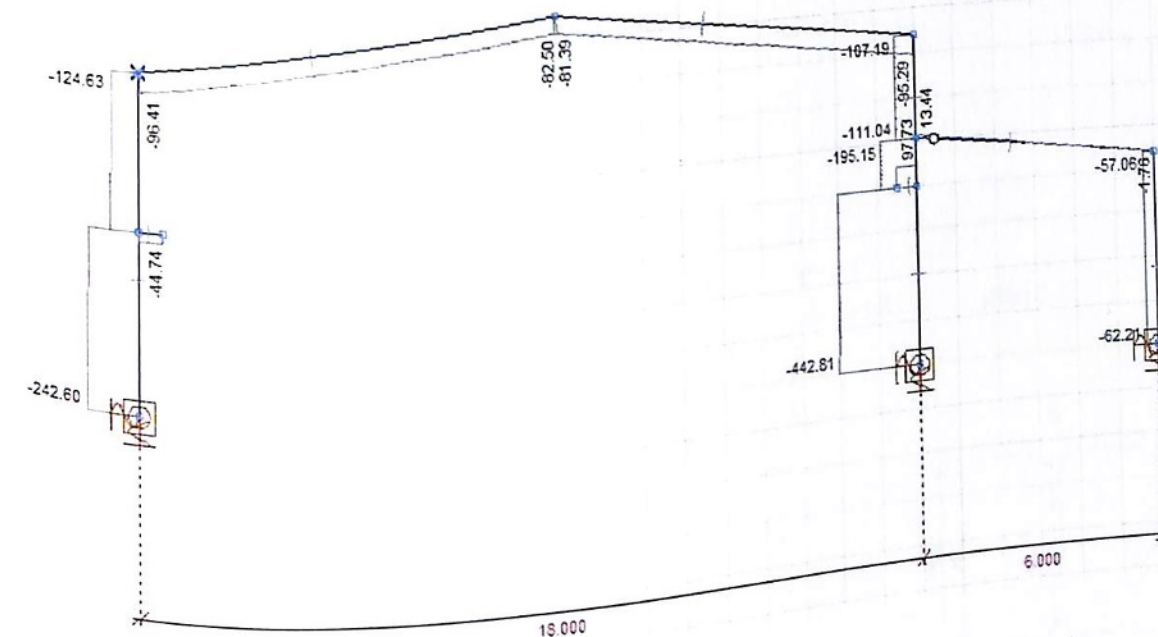
$$\begin{aligned} e1 &:= 2 \cdot d_o = 60 \\ e2 &:= 1.5 \cdot d_o = 45 \\ p2 &:= 3 \cdot d_o = 90 \\ p1 &:= 0 \end{aligned}$$

$$\frac{e1}{3 \cdot d_o} = 0.667 \quad \frac{f_{ub}}{f_u} = 2.778 \quad \frac{p1}{3 \cdot d_o} = 0 \quad 1.0$$

Tehát az $\alpha_b := \frac{e1}{3 \cdot d_o} = 0.667$



Ned



Húzott csavar ellenállása

$$Ned := 195.15 \text{ kN}$$

Nyírási ellenállás $n := 1$

$$Frd := \frac{\left(n \cdot \frac{0.6 \cdot f_{ub} \cdot A}{\gamma_{m2}} \right)}{1000} = 275.04 \text{ kN}$$

$Frd > Ned$
Megfelel

Palást nyomás ellenállás

$$Frd := \frac{\left(2.5 \cdot \frac{\alpha_b \cdot f_u \cdot d_{alk} \cdot t_{alk}}{\gamma_{m2}} \right)}{1000} = 324 \text{ kN}$$

$Frd > Ned$
Megfelel

Daru tartó toldása kapcsolat

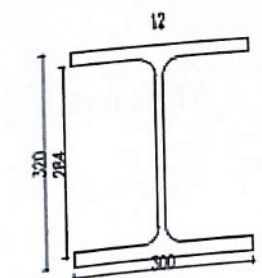
4.Csomópont

HEB 320

$$\begin{aligned} tw &:= 11.5 \text{ mm} & M20 & 8.8 \\ tf &:= 20.5 \text{ mm} & d &:= 20 \\ r &:= 27 \text{ mm} & do &:= 22 \end{aligned}$$

$$\begin{aligned} fy &:= 235 & f_{yb} &:= 640 & \gamma_{mo} &:= 1.0 & \varepsilon &:= 1 \\ fu &:= 360 & f_{ub} &:= 800 & \gamma_{m2} &:= 1.25 & A_{cs} &:= 314 \text{ mm}^2 \end{aligned}$$

$$\begin{aligned} A &:= 16130 & A_{vz} &:= 5177 & h &:= 320 \\ I_y &:= 30820 & W_{pl} &:= 2149 & h_w &:= 279 \\ I_z &:= 9239 & & & b &:= 300 \end{aligned}$$



Keresztmetszet hajlítási ellenállás

$$I_{yel} := \frac{tw \cdot h_w^3}{12} + 2 \cdot b \cdot tf \cdot 149.75^2 = 2.966 \cdot 10^8 \text{ mm}^4$$

$$W_{yel} := \frac{I_{yel}}{160} = 1.854 \cdot 10^6 \text{ mm}^3$$

$$M_{brd} := \frac{W_{yel} \cdot fy \cdot 10^{-6}}{\gamma_{mo}} = 435.691 \text{ KNm}$$

Öblemezek illesztése

Szelvény húzási ellenállása

$$N_{plrd} := \frac{b \cdot 140 \cdot fy \cdot 10^{-4}}{\gamma_{mo}} = 987 \text{ KN}$$

Csavarlyukkal gyengített szelvény törési tervezési ellenállás

$$N_{urd} := 0.9 \cdot \frac{A_{net} \cdot fu \cdot 10^{-3}}{\gamma_{m2}} = 769.306 \text{ KN}$$

$$N_{trd} := N_{urd} = 769.306 \text{ KN}$$

$$\begin{aligned} heveder &:= 140 \\ A_{net} &:= (300 - 4 \cdot 22) \cdot 14 = 2.968 \cdot 10^3 \end{aligned}$$

Csavarok ellenállása

$$n := 2$$

$$\alpha := 0.6$$

Egy csavar nyírási ellenállása

$$F_{vrd} := n \cdot \alpha \cdot \frac{A_{cs} \cdot f_{ub} \cdot 10^{-3}}{\gamma_{m2}} = 241.152 \quad KN$$

Egy csavar palástnyomási ellenállása

K1 számítás

$$\begin{aligned} & \text{minimum} \\ & 2.8 \cdot \frac{e_1}{d_o} - 1.7 = 2.8 \cdot \frac{35}{22} - 17 = 2.75 \quad -k_{1c} = 2.5 \\ & 2.5 \end{aligned}$$

$$-k_{min} = 2.5$$

$$\begin{aligned} & 1.4 \cdot \frac{P_2}{d_o} - 1.7 = 1.4 \cdot \frac{70}{22} - 17 = 2.75 \quad -k_{1p} = 2.5 \\ & 2.5 \end{aligned}$$

-ab számítása

- erő irányában szélső távolságra

$$\begin{aligned} & \text{minimum} \\ & \frac{e_1}{3 \cdot d_o} = \frac{45}{3 \cdot 22} = 0.68 \\ & \frac{f_{ub}}{f_u} = \frac{80}{51} = 1.57 \end{aligned}$$

$$-ab.e = 0.68$$

$$-abmin = 0.81$$

- erő irányában közbenső távolságra

$$\begin{aligned} & \text{minimum} \\ & \frac{p_1}{3 \cdot d_o} - \frac{1}{4} = 0.81 \\ & \square \end{aligned}$$

$$-ab.e = 0.81$$

$$\frac{f_{ub}}{f_u} = \frac{80}{51} = 1.57$$

$$\begin{aligned} & \text{Palástnyomási ellenállás} \\ & k_1 \cdot \alpha \cdot f_u \cdot d \cdot t \quad 2.5 \cdot 0.68 \cdot 360 \cdot 20 \cdot 14 \end{aligned}$$

$$F_{brd} = \frac{k_1 \cdot \alpha \cdot f_u \cdot d \cdot t}{\gamma_2} = \frac{2.5 \cdot 0.68 \cdot 360 \cdot 20 \cdot 14}{1.25} = 137.9 \text{ kN}$$

Egy csavarra jutó nyíróerő

$$F_{ed} = \frac{N_{trd}}{m} = \frac{769.305}{8} = 96.163 \leq F_{brd} = 137.163 \text{ kN} \quad \text{Megfelel!}$$

Övhevederek méretei:

$$\text{külső hevederek:} \quad 300 - 8 \quad A_{n.k} = (30 - 4 \cdot 2.2) \cdot 0.8 = 16.96 \text{ cm}^2$$

$$\text{belső hevederek:} \quad 2 \cdot 140 - 8 \quad A_{n.b} = 2 \cdot (14 - 2 \cdot 2.2) \cdot 0.8 = 15.36 \text{ cm}^2$$

$$A_{nhev} = 16.96 + 15.36 = 32.32 \text{ cm}^2 \quad \square \geq \square \quad A_{nlem} = (30 - 4 \cdot 2.2) \cdot 1.4 = 29.68 \text{ cm}^2$$

Megfelel!

Gerinclemez illesztése:

$$V_{ed} := 195.75 \text{ kN} \quad \text{daru} \quad m := 8$$

$$F_{ved} := \frac{V_{ed} \cdot 10^{-3}}{m} = 24.469 \text{ N}$$

$$M_{g.brd} := 435.68 \cdot \frac{20.812}{296.64} = 30.567 \text{ kN} \quad Z_{max} := 40$$

$$F_{vM.ed} := M_{g.brd} \cdot \frac{Z_{max}}{8 \cdot 40^2} = 30.567 \cdot \frac{40}{8 \cdot 40^2} = 95.52 \text{ kN}$$

$$F_{ved} := \sqrt{95.5^2} + \sqrt{24.469^2} = 119.969 \text{ kN}$$

A legjobban igénybevett csavar ellenállása:

Palástnyomási ellenállás

- k1 számítás

- erő irányára merőlegesen szélső csavar

$$-\square \quad 2.8 \cdot \frac{e_2}{d_o} - 1.7 = 2.8 \cdot \frac{40}{22} - 1.7 = 4.5$$

$$-\square \quad k1e := 2.5$$

2.5

- erő irányára merőlegesen közbenső távolságra

$$1.4 \cdot \frac{p_2}{d_o} - 1.7 = 1.4 \cdot \frac{80}{22} = 3.74$$

$$-\square \quad k1p := 2.5$$

2.5

- α számítása

-\square \quad minimum

$$\frac{e_1}{3 \cdot d_o} = \frac{50}{3 \cdot 22} = 0.925$$

$$\frac{f_{ub}}{f_u} = \frac{80}{51} = 1.57$$

$$-\square \quad \alpha := 0.925$$

- erő irányára közbenső távolságra

-\square \quad minimum

$$\frac{p_1}{3 \cdot d_o} - \frac{1}{4} = \frac{80}{3 \cdot 18} - \frac{1}{4} = 1.296$$

$$\frac{f_{ub}}{f_u} = \frac{80}{51} = 1.57$$

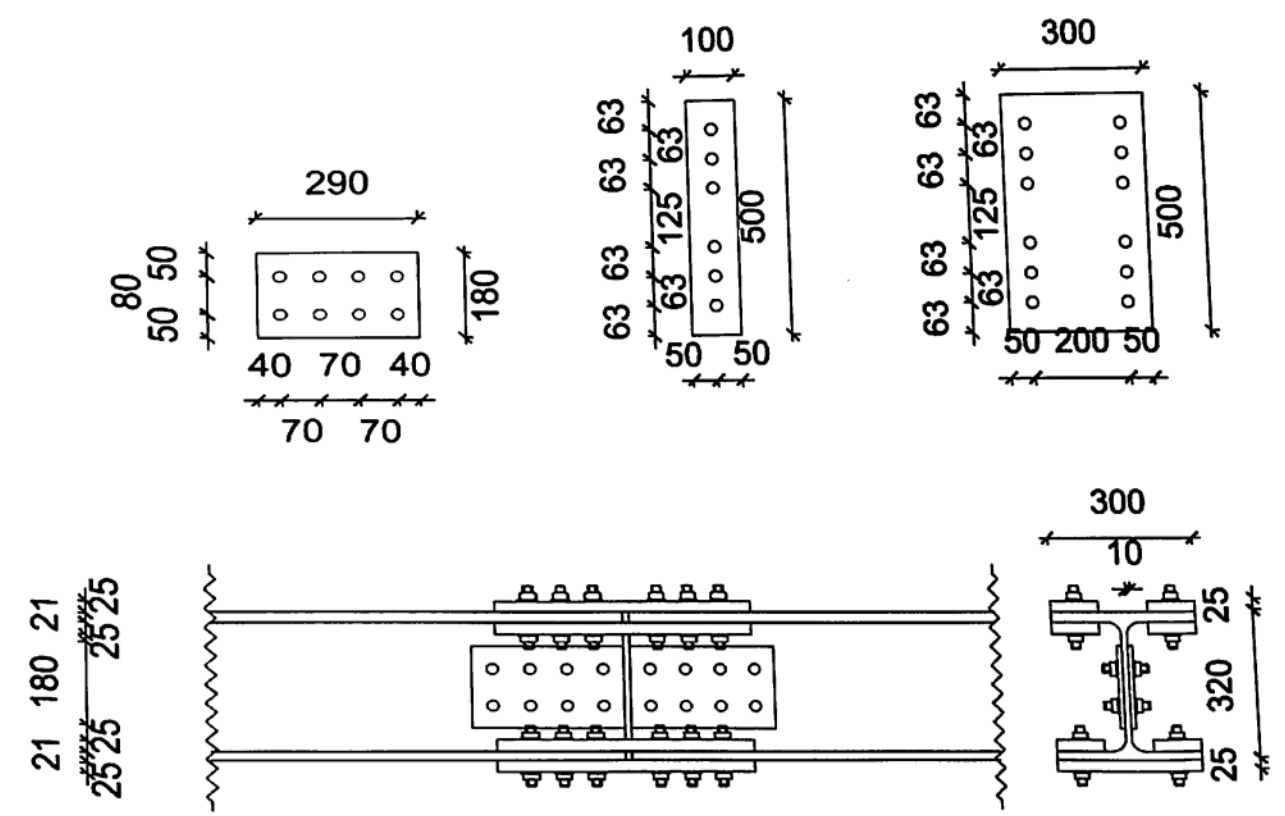
$$-\square \quad \alpha b.p := 1$$

$$Frd := \frac{k1e \cdot \alpha \cdot 360 \cdot 18 \cdot 11.5 \cdot 10^{-3}}{1.25} = 137.862 \quad kN$$

Ellenőrzés:

$$F_{ved} = 119.969 \quad kN \quad \square < \square \quad Frd = 137.862 \quad kN$$

Megfelel



VII. Falvázoszlop méretezése**Rétegrend**

külső trapézlemez LTP45/0.7	$g_{k1} := 0.7$	$a := 2.300$	m
HEA 200 falvázoszlop	$g_o := 0.423$	$b := 1.600$	m
hőszigetelés 15 cm	$g_{hsz} := 0.12$	$h := 8.243$	m
párazáró fólia 0.1 cm	$g_{sz} := 0.014$	$n := 6$	db
belső trapézlemez LTP45/0.7	$g_{b1} := 0.7$		

Burkolat súlya

$$t := a \cdot b = 3.68 \quad m^2$$

$$p_{burkolat} := t \cdot (g_{k1} + g_{hsz} + g_{sz} + g_{b1}) + a \cdot g_o = 6.618 \quad \frac{kN}{m}$$

$$G := g_o + (n \cdot p_{burkolat}) = 40.131$$

Szélteher

Terepkategória IV

$$z := 7400 + 200 + 200 + 45 + 1000 = 8845 \quad mm$$

$$\text{szél torlónyomás} \quad q_{pz} := 0.2955$$

"D" zóna

$$c_{pe10} := 0.8$$

$$p_{wenyD} := a \cdot q_{pz} \cdot c_{pe10} = 0.544$$

$$p_{wd} := p_{wenyD} \cdot 1.5 = 0.816 \quad kN$$

Teherbírás

$$M_{ed} := \frac{p_{wd} \cdot h^2}{8} = 6.927 \quad kNm$$

$$V_{ed} := \frac{p_{wd} \cdot h}{2} = 3.361 \quad kN$$

Geometriai Interakció

 $l := 18 \text{ m}$
 $Med := 179.43 \text{ kNm}$
 $h := 0.484$

merevítéshez kapcsolt főtartók száma

$$Ned := \frac{Med}{h} = 370.723 \text{ kN}$$

$$m := \frac{6}{2} = 3$$

$$\alpha m := \sqrt{0.5 \cdot \left(1 + \frac{1}{m}\right)} = 0.816$$

$$eo := \frac{\alpha m \cdot l}{500} = 0.029 \text{ m}$$

$$\delta o := \frac{l}{2500} = 0.007 \text{ m}$$

$$q := 8 \cdot m \cdot Ned \cdot \frac{eo + \delta o}{l^2} = 1.005$$

$$Ved2 := \frac{q \cdot l}{2} = 9.044 \text{ kN}$$

$$V := Ved + Ved2 = 12.406 \text{ kN}$$

$$\frac{V}{2} = 6.203 \text{ kN}$$

$$s := 7.59 \text{ m}$$

$$t := 3.3 \text{ m}$$

$$SVed := V \cdot \frac{s}{t} = 28.533 \text{ KN}$$

$$\gamma mo := 1$$

$$fy := 235 \frac{\text{N}}{\text{mm}^2}$$

$$\phi := 20$$

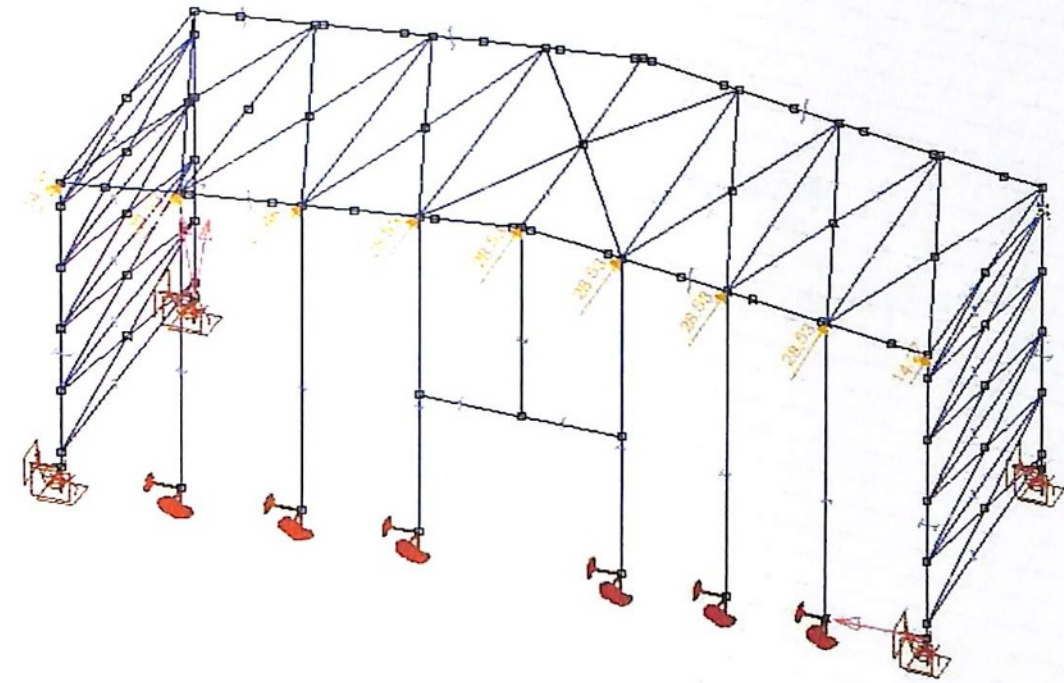
$$A := \frac{\phi^2 \pi}{4} = 314.159$$

$$Lcr := L = 3.497$$

$$Nrd := \frac{A \cdot fy}{\gamma mo} = 73.827 \text{ kN}$$

$$SVed = 28.533 \text{ KN}$$

megfelel



VIII. Alapozás Tervezés**HEA400**

Alaptest súlya: $\gamma b := 25 \frac{kN}{m^3}$ beton sűrűsége

$B := 2.30$ $b := 1.00$ $\gamma g := 1.35$
 $L := 2.30$ $l := 1.00$ $\gamma q := 1.5$
 $H := 1.40$
 $h := 0.80$

$$Ga := B \cdot L \cdot h \cdot \gamma b = 105.8 \quad kN$$

Alaptest feletti föld súlya:

$$Gf := (B \cdot L - b \cdot l) \cdot (0.6 \cdot 18) = 46.332 \quad KN$$

Alaptestre ható felhajtóerő:

$$Ff := 0 \quad kN \quad \text{Talajvízszint 8 m mélyen van}$$

$$\text{Szerkezeti terhelés:} \quad Gvk := 254 \quad kN$$

Alapra jutó tervezési érték:

$$Vd := \gamma g \cdot (Gvk + Ga + (Gf - Ff)) = 548.278 \quad \frac{kN}{m} \quad V_k := \frac{Vd}{\gamma g} = 406.132$$

$$Med := 277 + 102.84 \cdot H = 420.976 \quad Ned := V_k = 406.132$$

$$eo := \frac{Med}{Ned} = 1.037$$

$$B' := B - 2 \cdot eo = 0.227$$

$$L' := L = 2.3$$

Talaj Ellenőrzése

Törő képlet: $\varphi := 17$ $\varphi := \frac{\varphi}{360} \cdot (2 \cdot \pi) = 0.297 \text{ rad}$

$$Nq := e^{(\pi \cdot (\tan(\varphi)))} \cdot \left(\tan \left(45 + \frac{\varphi}{2} \right) \right)^2 = 12.917$$

$$\begin{aligned} ctg17 &:= 3.27 \\ Nq &:= 12.917 \\ \tan\varphi &:= 0.305 \\ \sin\varphi &:= 0.192 \end{aligned}$$

$$Nc := (Nq - 1) \cdot ctg17 = 38.969$$

$$N\gamma := 2 (Nq - 1) \cdot \tan\varphi = 7.269$$

$$sq := 1 + \frac{B'}{L'} \cdot \sin\varphi = 1.019$$

$$sc := \frac{(sq \cdot Nq - 1)}{Nq - 1} = 1.021$$

$$s\gamma := 1 - \left(0.3 \cdot \frac{B'}{L'} \right) = 0.97$$

$$ck := 115$$

$$q := h \cdot \varphi = 13.6$$

$$\begin{aligned} bq &:= 1 & bc &:= 1 & b\gamma &:= 1 \\ iq &:= 1 & ic &:= 1 & i\gamma &:= 1 \end{aligned}$$

Alap alatti térfogatsúly

$$\gamma' := 18$$

Alap dolgozó területe

$$A' := B' \cdot L' = 0.522 \text{ m}^2$$

$$Rk := A' \cdot (q \cdot Nq \cdot bq \cdot sq \cdot iq + ck \cdot Nc \cdot bc \cdot sc \cdot ic + 0.5 \cdot B \cdot \gamma' \cdot N\gamma \cdot b\gamma \cdot s\gamma \cdot i\gamma) = 2.556 \cdot 10^3 \text{ KN}$$

$$Rk \geq Vd$$

$$\frac{Rk}{1.4} = 1.826 \cdot 10^3 \text{ KN} \quad \square > \square \quad Vd = 548.278 \text{ KN}$$

Megfelel $\square$

HEA300

Alaptest súlya: $\gamma b := 25 \frac{kN}{m^3}$ beton sűrűsége

$B := 1.50$ $b := 1.00$ $\gamma g := 1.35$
 $L := 1.50$ $l := 1.00$ $\gamma q := 1.5$
 $H := 1.40$
 $h := 0.50$

$$Ga := B \cdot L \cdot h \cdot \gamma b = 28.125 \quad kN$$

Alaptest feletti föld súlya:

$$Gf := (B \cdot L - b \cdot l) \cdot (0.9 \cdot 18) = 20.25 \quad KN$$

Alaptestre ható felhajtóerő:

$$Ff := 0 \quad kN \quad \text{Talajvízszint 8 m mélyen van}$$

Szerkezeti terhelés: $Gvk := 46.89 \quad kN$

Alapra jutó tervezési érték:

$$Vd := \gamma g \cdot (Gvk + Ga + (Gf - Ff)) = 128.608 \quad \frac{kN}{m} \quad V_k := \frac{Vd}{\gamma g} = 95.265$$

$$Med := 30.029 + 7.74 \cdot H = 40.865 \quad Ned := V_k = 95.265$$

$$eo := \frac{Med}{Ned} = 0.429$$

$$B' := B - 2 \cdot eo = 0.642$$

$$L' := L = 1.5$$

Talaj Ellenőrzése

Törő képlet: $\varphi := 17$ $\varphi := \frac{\varphi}{360} \cdot (2 \cdot \pi) = 0.297 \text{ rad}$

$$Nq := e^{(\pi \cdot (\tan(\varphi)))} \cdot \left(\tan \left(45 + \frac{\varphi}{2} \right) \right)^2 = 12.917$$

$$\begin{aligned} ctg17 &:= 3.27 \\ Nq &:= 12.917 \\ \tan\varphi &:= 0.305 \\ \sin\varphi &:= 0.192 \end{aligned}$$

$$Nc := (Nq - 1) \cdot ctg17 = 38.969$$

$$N\gamma := 2 (Nq - 1) \cdot \tan\varphi = 7.269$$

$$sq := 1 + \frac{B'}{L'} \cdot \sin\varphi = 1.082$$

$$sc := \frac{(sq \cdot Nq - 1)}{Nq - 1} = 1.089$$

$$s\gamma := 1 - \left(0.3 \cdot \frac{B'}{L'} \right) = 0.872$$

$$ck := 115$$

$$q := h \cdot \varphi = 8.5$$

$$\begin{aligned} bq &:= 1 & bc &:= 1 & b\gamma &:= 1 \\ iq &:= 1 & ic &:= 1 & i\gamma &:= 1 \end{aligned}$$

Alap alatti térfogatsúly

$$\gamma' := 18$$

Alap dolgozó területe

$$A' := B' \cdot L' = 0.963 \text{ m}^2$$

$$Rk := A' \cdot (q \cdot Nq \cdot bq \cdot sq \cdot iq + ck \cdot Nc \cdot bc \cdot sc \cdot ic + 0.5 \cdot B \cdot \gamma' \cdot N\gamma \cdot b\gamma \cdot s\gamma \cdot i\gamma) = 4.897 \cdot 10^3 \text{ KN}$$

$$Rk \geq Vd$$

$$\frac{Rk}{1.4} = 3.498 \cdot 10^3 \text{ KN} \quad \square > \square \quad Vd = 128.608 \text{ KN}$$

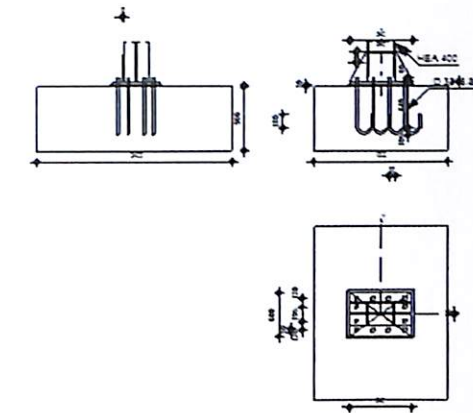
Megfelel $\square$



Fixed column base design

Eurocode 3: EN 1993-1-8:2005/AC:2009

OK

Ratio
0.98

GENERAL

Connection no.: 1
Connection name: OT1
Structure node: 1
Structure bars: 2

GEOMETRY

COLUMN

Section: HEA 400
Bar no.: 2

L_c	=	7.40	[m]	Column length
a	=	0.0	[Deg]	Inclination angle
h_c	=	390	[mm]	Height of column section
b_{fc}	=	300	[mm]	Width of column section
t_{wc}	=	11	[mm]	Thickness of the web of column section
t_{fc}	=	19	[mm]	Thickness of the flange of column section
r_c	=	27	[mm]	Radius of column section fillet
A_c	=	158.98	[cm ²]	Cross-sectional area of a column
I_{yc}	=	45069.40	[cm ⁴]	Moment of inertia of the column section
Material: S235				
f_{yc}	=	235.00	[MPa]	Resistance
f_{uc}	=	360.00	[MPa]	Yield strength of a material

COLUMN BASE

l_{pd}	=	840	[mm]	Length
b_{pd}	=	600	[mm]	Width
t_{pd}	=	30	[mm]	Thickness
Material: S275				
f_{ypd}	=	275.00	[MPa]	Resistance
f_{upd}	=	430.00	[MPa]	Yield strength of a material

ANCHORAGE

The shear plane passes through the UNTHREADED portion of the bolt.

Class	=	8.8		Anchor class
f_{yb}	=	640.00	[MPa]	Yield strength of the anchor material
f_{ub}	=	800.00	[MPa]	Tensile strength of the anchor material
d	=	33	[mm]	Bolt diameter
A_s	=	6.94	[cm ²]	Effective section area of a bolt
A_v	=	8.55	[cm ²]	Area of bolt section
n_{H1}	=	4		Number of bolt columns
n_V	=	4		Number of bolt rows
Horizontal spacing e_{H1}	=	215; 215	[mm]	
Vertical spacing e_{V1}	=	195; 120	[mm]	

Anchor dimensions

L ₁ =	60	[mm]
L ₂ =	640	[mm]
L ₃ =	210	[mm]
L ₄ =	180	[mm]

Washer			
l _{wd} =	50	[mm]	Length
b _{wd} =	50	[mm]	Width
t _{wd} =	5	[mm]	Thickness

STIFFENER			
l _s =	840	[mm]	Length
h _s =	390	[mm]	Height
t _s =	16	[mm]	Thickness
d ₁ =	20	[mm]	Cut
d ₂ =	20	[mm]	Cut

MATERIAL FACTORS		
EM0 =	1.00	Partial safety factor
EM2 =	1.25	Partial safety factor
EC =	1.50	Partial safety factor

SPREAD FOOTING			
L =	1800	[mm]	Spread footing length
B =	2400	[mm]	Spread footing width
H =	900	[mm]	Spread footing height

Concrete			
Class	C20/25		
f _{ck} =	20.00	[MPa]	Characteristic resistance for compression

Grout layer			
l _g =	30	[mm]	Thickness of leveling layer (grout)
f _{ck,g} =	12.00	[MPa]	Characteristic resistance for compression
C _{f,d} =	0.30		Coeff. of friction between the base plate and concrete

WELDS			
a _p =	7	[mm]	Footing plate of the column base
a _s =	4	[mm]	Stiffeners

LOADS			
Case: 104: ULS/90=1*1.35 + 6*1.35 + 3*1.50 + 9*0.90 + 13*0.90 + 10*0.75 (1+6)*1.35+3*1.50+(9+13)*0.90+10*0.75			
N _{j,Ed} =	-198.91	[kN]	Axial force
V _{j,Ed,z} =	139.82	[kN]	Shear force
M _{j,Ed,y} =	-392.69	[kN*m]	Bending moment

RESULTS

COMPRESSION ZONE

COMPRESSION OF CONCRETE

$f_{cd} =$	13.33	[MPa]	Design compressive resistance	EN 1992-1:[3.1.6.(1)]
$f_j =$	20.23	[MPa]	Design bearing resistance under the base plate	[6.2.5.(7)]
$c = t_p \diamond$				[6.2.5.(4)]
$c =$	64	[mm]	Additional width of the bearing pressure zone	[6.2.5.(3)]
$b_{eff} =$	147	[mm]	Effective width of the bearing pressure zone under the flange	[6.2.5.(3)]
$l_{eff} =$	428	[mm]	Effective length of the bearing pressure zone under the flange	[6.2.5.(3)]
$A_{c0} =$	627.59	[cm ²]	Area of the joint between the base plate and the foundation	EN 1992-1:[6.7.(3)]
$A_{c1} =$	5648.29	[cm ²]	Maximum design area of load distribution	EN 1992-1:[6.7.(3)]
$F_{rd,u} = A_{c0} \cdot f_{cd} \diamond$				
$F_{rd,u} =$	2510.35	[kN]	Bearing resistance of concrete	EN 1992-1:[6.7.(3)]
$h_j =$	0.67		Reduction factor for compression	[6.2.5.(7)]
$f_{j1} = h_j \cdot F_{rd,u} / (b_{eff} \cdot l_{eff})$				
$f_{j1} =$	26.67	[MPa]	Design bearing resistance	[6.2.5.(7)]

$A_{c,n}$	3694.46	[cm ²]	Bearing area for compression	[6.2.8.2.(1)]
$A_{c,y}$	1369.33	[cm ²]	Bearing area for bending M_y	[6.2.8.3.(1)]
$F_{c,Rd,i} = A_{c,i} \cdot f_{jd}$				
$F_{c,Rd,n}$	9851.91	[kN]	Bearing resistance of concrete for compression	[6.2.8.2.(1)]
$F_{c,Rd,y}$	3651.54	[kN]	Bearing resistance of concrete for bending M_y	[6.2.8.3.(1)]

COLUMN FLANGE AND WEB IN COMPRESSION

CL	1.00		Section class	EN 1993-1-1:[5.5.2]
$W_{pl,y}$	10420.77	[cm ³]	Plastic section modulus	EN1993-1-1:[6.2.5.(2)]
$M_{c,Rd,y}$	2448.88	[kN*m]	Design resistance of the section for bending	EN1993-1-1:[6.2.5]
$h_{f,y}$	527	[mm]	Distance between the centroids of flanges	[6.2.6.7.(1)]
$F_{c,fc,Rd,y} = M_{c,Rd,y} / h_{f,y}$				
$F_{c,fc,Rd,y}$	4645.00	[kN]	Resistance of the compressed flange and web	[6.2.6.7.(1)]

RESISTANCES OF SPREAD FOOTING IN THE COMPRESSION ZONE

$N_{j,Rd} = F_{c,Rd,n}$				
$N_{j,Rd}$	9851.91	[kN]	Resistance of a spread footing for axial compression	[6.2.8.2.(1)]
$F_{c,Rd,y} = \min(F_{c,Rd,y}, F_{c,fc,Rd,y})$				
$F_{c,Rd,y}$	3651.54	[kN]	Resistance of spread footing in the compression zone	[6.2.8.3]

TENSION ZONE

STEEL FAILURE

A_b	6.94	[cm ²]	Effective anchor area	[Table 3.4]
f_{ub}	800.00	[MPa]	Tensile strength of the anchor material	[Table 3.4]
$Beta$	0.85		Reduction factor of anchor resistance	[3.6.1.(3)]
$F_{t,Rd,s1} = beta \cdot 0.9 \cdot f_{ub} \cdot A_b / \gamma_{M2}$				
$F_{t,Rd,s1}$	339.78	[kN]	Anchor resistance to steel failure	[Table 3.4]
$F_{t,Rd,s} = F_{t,Rd,s1}$				
$F_{t,Rd,s}$	339.78	[kN]	Anchor resistance to steel failure	

PULL-OUT FAILURE

f_{ck}	20.00	[MPa]	Characteristic compressive strength of concrete	EN 1992-1:[3.1.2]
$f_{cd} = 0.7 \cdot 0.3 \cdot f_{ck}^{2/3} / \gamma_c$				
f_{cd}	1.03	[MPa]	Design tensile resistance	EN 1992-1:[8.4.2.(2)]
h_1	1.00		Coeff. related to the quality of the bond conditions and concreting conditions	EN 1992-1:[8.4.2.(2)]
h_2	0.99		Coeff. related to the bar diameter	EN 1992-1:[8.4.2.(2)]
$f_{bd} = 2.25 \cdot h_1 \cdot h_2 \cdot f_{cd}$				
f_{bd}	2.30	[MPa]	Design value of the ultimate bond stress	EN 1992-1:[8.4.2.(2)]
h_{ef}	640	[mm]	Effective anchorage depth	EN 1992-1:[8.4.2.(2)]
$F_{t,Rd,p} = p \cdot d \cdot h_{ef} \cdot f_{bd}$				
$F_{t,Rd,p}$	152.46	[kN]	Design uplift capacity	EN 1992-1:[8.4.2.(2)]

TENSILE RESISTANCE OF AN ANCHOR

$F_{t,Rd} = \min(F_{t,Rd,s}, F_{t,Rd,p})$				
$F_{t,Rd}$	152.46	[kN]	Tensile resistance of an anchor	

BENDING OF THE BASE PLATE

Bending moment $M_{j,Ed,y}$				
$l_{eff,1}$	199	[mm]	Effective length for a single bolt for mode 1	[6.2.6.5]
$l_{eff,2}$	199	[mm]	Effective length for a single bolt for mode 2	[6.2.6.5]
m	44	[mm]	Distance of a bolt from the stiffening edge	[6.2.6.5]
$M_{pl,1,Rd}$	12.30	[kN*m]	Plastic resistance of a plate for mode 1	[6.2.4]
$M_{pl,2,Rd}$	12.30	[kN*m]	Plastic resistance of a plate for mode 2	[6.2.4]
$F_{T,1,Rd}$	1128.58	[kN]	Resistance of a plate for mode 1	[6.2.4]
$F_{T,2,Rd}$	589.59	[kN]	Resistance of a plate for mode 2	[6.2.4]
$F_{T,3,Rd}$	609.82	[kN]	Resistance of a plate for mode 3	[6.2.4]
$F_{t,pl,Rd,y} = \min(F_{T,1,Rd}, F_{T,2,Rd}, F_{T,3,Rd})$				
$F_{t,pl,Rd,y}$	589.59	[kN]	Tension resistance of a plate	[6.2.4]

RESISTANCES OF SPREAD FOOTING IN THE TENSION ZONE

$F_{T,Rd,y} = F_{t,pl,Rd,y}$				
$F_{T,Rd,y}$	589.59	[kN]	Resistance of a column base in the tension zone	[6.2.8.3]

CONNECTION CAPACITY CHECK

$N_{j,Ed} / N_{j,Rd} \leq 1.0$ (6.24)		0.02 < 1.00	verified	(0.02)
$e_y =$	1974	[mm]	Axial force eccentricity	[6.2.8.3]
$z_{c,y} =$	264	[mm]	Lever arm $F_{C,Rd,y}$	[6.2.8.1.(2)]
$z_{t,y} =$	323	[mm]	Lever arm $F_{T,Rd,y}$	[6.2.8.1.(3)]
$M_{j,Rd,y} =$	398.81	[kN*m]	Connection resistance for bending	[6.2.8.3]
$M_{j,Ed,y} / M_{j,Rd,y} \leq 1.0$ (6.23)		0.98 < 1.00	verified	(0.98)

SHEAR

BEARING PRESSURE OF AN ANCHOR BOLT ONTO THE BASE PLATE

Shear force $V_{j,Ed,z}$				
$a_{d,z} =$	0.93		Coeff. taking account of the bolt position - in the direction of shear	[Table 3.4]
$a_{b,z} =$	0.93		Coeff. for resistance calculation $F_{1,vb,Rd}$	[Table 3.4]
$k_{1,z} =$	2.50		Coeff. taking account of the bolt position - perpendicularly to the direction of shear	[Table 3.4]
$F_{1,vb,Rd,z} = k_{1,z} \cdot a_{b,z} \cdot f_{up} \cdot d \cdot t_p / \gamma_{M2}$				[6.2.2.(7)]
$F_{1,vb,Rd,z} =$	790.59	[kN]	Resistance of an anchor bolt for bearing pressure onto the base plate	

SHEAR OF AN ANCHOR BOLT

$a_b =$	0.25		Coeff. for resistance calculation $F_{2,vb,Rd}$	[6.2.2.(7)]
$A_{vb} =$	8.55	[cm ²]	Area of bolt section	[6.2.2.(7)]
$f_{ub} =$	800.00	[MPa]	Tensile strength of the anchor material	[6.2.2.(7)]
$\gamma_{M2} =$	1.25		Partial safety factor	[6.2.2.(7)]
$F_{2,vb,Rd} = a_b \cdot f_{ub} \cdot A_{vb} / \gamma_{M2}$				[6.2.2.(7)]
$F_{2,vb,Rd} =$	135.75	[kN]	Shear resistance of a bolt - without lever arm	

SPLITTING RESISTANCE

$C_{f,d} =$	0.30		Coeff. of friction between the base plate and concrete	[6.2.2.(6)]
$N_{c,Ed} =$	198.91	[kN]	Compressive force	[6.2.2.(6)]
$F_{f,Rd} = C_{f,d} \cdot N_{c,Ed}$				[6.2.2.(6)]
$F_{f,Rd} =$	59.67	[kN]	Slip resistance	

SHEAR CHECK

$V_{j,Rd,z} = n_b \cdot \min(F_{1,vb,Rd,z}, F_{2,vb,Rd}) + F_{f,Rd}$				
$V_{j,Rd,z} =$	1688.71	[kN]	Connection resistance for shear	(0.08)
$V_{j,Ed,z} / V_{j,Rd,z} \leq 1.0$		0.08 < 1.00	verified	

STIFFENER CHECK

Stiffener parallel to the web (along the extension of the column web)

$M_1 =$	24.67	[kN*m]	Bending moment acting on a stiffener	
$Q_1 =$	193.51	[kN]	Shear force acting on a stiffener	
$z_s =$	134	[mm]	Location of the neutral axis (from the plate base)	
$I_s =$	19824.28	[cm ⁴]	Moment of inertia of a stiffener	
$\sigma_d =$	12.99	[MPa]	Normal stress on the contact surface between stiffener and plate	EN 1993-1-1:[6.2.1.(5)]
$\sigma_g =$	35.55	[MPa]	Normal stress in upper fibers	EN 1993-1-1:[6.2.1.(5)]
$\tau =$	31.01	[MPa]	Tangent stress in a stiffener	EN 1993-1-1:[6.2.1.(5)]
$\sigma_z =$	55.26	[MPa]	Equivalent stress on the contact surface between stiffener and plate	EN 1993-1-1:[6.2.1.(5)]
$\max(\sigma_g, \tau / (0.58), \sigma_z) / (f_{yp} / \gamma_{M0}) \leq 1.0$ (6.1)		0.20 < 1.00	verified	(0.20)

Trapezoid plate parallel to the column web

$M_1 =$	24.67	[kN*m]	Bending moment acting on a stiffener	
$Q_1 =$	193.51	[kN]	Shear force acting on a stiffener	
$z_s =$	112	[mm]	Location of the neutral axis (from the plate base)	
$I_s =$	22733.88	[cm ⁴]	Moment of inertia of a stiffener	
$\sigma_d =$	8.93	[MPa]	Normal stress on the contact surface between stiffener and plate	EN 1993-1-1:[6.2.1.(5)]
$\sigma_g =$	33.40	[MPa]	Normal stress in upper fibers	EN 1993-1-1:[6.2.1.(5)]
$\tau =$	31.01	[MPa]	Tangent stress in a stiffener	EN 1993-1-1:[6.2.1.(5)]
$\sigma_z =$	54.45	[MPa]	Equivalent stress on the contact surface between stiffener and plate	EN 1993-1-1:[6.2.1.(5)]
$\max(\sigma_g, \tau / (0.58), \sigma_z) / (f_{yp} / \gamma_{M0}) \leq 1.0$ (6.1)		0.20 < 1.00	verified	(0.20)

WELDS BETWEEN THE COLUMN AND THE BASE PLATE

$s_{\perp}$	=	41.32	[MPa]	Normal stress in a weld	[4.5.3.(7)]
$t_{\perp}$	=	41.32	[MPa]	Perpendicular tangent stress	[4.5.3.(7)]
$t_{ }$	=	0.00	[MPa]	Tangent stress parallel to $V_{j,Ed,y}$	[4.5.3.(7)]
$t_{2 }$	=	6.36	[MPa]	Tangent stress parallel to $V_{j,Ed,z}$	[4.5.3.(7)]
b_W	=	0.80		Resistance-dependent coefficient	[4.5.3.(7)]
$s_{\perp} / (0.9 \cdot f_{u,GM2}) \leq 1.0$ (4.1)				0.16 < 1.00	verified (0.16)
◆				0.23 < 1.00	verified (0.23)
◆				0.20 < 1.00	verified (0.20)

VERTICAL WELDS OF STIFFENERS

Stiffener parallel to the web (along the extension of the column web)

$s_{\perp}$	=	86.03	[MPa]	Normal stress in a weld	[4.5.3.(7)]
$t_{\perp}$	=	86.03	[MPa]	Perpendicular tangent stress	[4.5.3.(7)]
$t_{ }$	=	62.02	[MPa]	Parallel tangent stress	[4.5.3.(7)]
s_2	=	202.84	[MPa]	Total equivalent stress	[4.5.3.(7)]
b_W	=	0.80		Resistance-dependent coefficient	[4.5.3.(7)]
$\max(s_{\perp}, t_{ }) \cdot \blacklozenge$				0.56 < 1.00	verified (0.56)

Trapezoid plate parallel to the column web

$s_{\perp}$	=	0.00	[MPa]	Normal stress in a weld	[4.5.3.(7)]
$t_{\perp}$	=	0.00	[MPa]	Perpendicular tangent stress	[4.5.3.(7)]
$t_{ }$	=	110.13	[MPa]	Parallel tangent stress	[4.5.3.(7)]
s_2	=	0.00	[MPa]	Total equivalent stress	[4.5.3.(7)]
b_W	=	0.80		Resistance-dependent coefficient	[4.5.3.(7)]
$\max(s_{\perp}, t_{ }) \cdot \blacklozenge$				0.53 < 1.00	verified (0.53)

TRANSVERSAL WELDS OF STIFFENERS

Stiffener parallel to the web (along the extension of the column web)

$s_{\perp}$	=	76.02	[MPa]	Normal stress in a weld	[4.5.3.(7)]
$t_{\perp}$	=	76.02	[MPa]	Perpendicular tangent stress	[4.5.3.(7)]
$t_{ }$	=	69.02	[MPa]	Parallel tangent stress	[4.5.3.(7)]
s_2	=	193.41	[MPa]	Total equivalent stress	[4.5.3.(7)]
b_W	=	0.80		Resistance-dependent coefficient	[4.5.3.(7)]
$\max(s_{\perp}, t_{ }) \cdot \blacklozenge$				0.45 < 1.00	verified (0.45)

Trapezoid plate parallel to the column web

$s_{\perp}$	=	76.02	[MPa]	Normal stress in a weld	[4.5.3.(7)]
$t_{\perp}$	=	76.02	[MPa]	Perpendicular tangent stress	[4.5.3.(7)]
$t_{ }$	=	74.84	[MPa]	Parallel tangent stress	[4.5.3.(7)]
s_2	=	199.79	[MPa]	Total equivalent stress	[4.5.3.(7)]
b_W	=	0.80		Resistance-dependent coefficient	[4.5.3.(7)]
$\max(s_{\perp}, t_{ }) \cdot \blacklozenge$				0.46 < 1.00	verified (0.46)

CONNECTION STIFFNESS

Bending moment $M_{j,Ed,y}$

b_{eff}	=	147	[mm]	Effective width of the bearing pressure zone under the flange	[6.2.5.(3)]
l_{eff}	=	428	[mm]	Effective length of the bearing pressure zone under the flange	[6.2.5.(3)]
$k_{13,y} = E_c \cdot \blacklozenge$					
$k_{13,y}$	=	28	[mm]	Stiffness coeff. of compressed concrete	[Table 6.11]
l_{eff}	=	199	[mm]	Effective length for a single bolt for mode 2	[6.2.6.5]
m	=	44	[mm]	Distance of a bolt from the stiffening edge	[6.2.6.5]
$k_{15,y} = 0.425 \cdot l_{eff} \cdot t_p^3 / (m^3)$					
$k_{15,y}$	=	28	[mm]	Stiffness coeff. of the base plate subjected to tension	[Table 6.11]
L_b	=	346	[mm]	Effective anchorage depth	[Table 6.11]
$k_{16,y} = 1.6 \cdot A_b / L_b$					
$k_{16,y}$	=	3	[mm]	Stiffness coeff. of an anchor subjected to tension	[Table 6.11]
l_0	=	0.47		Column slenderness	[5.2.2.5.(2)]
$S_{I,m,y}$	=	73483.41	[kN*m]	Initial rotational stiffness	[Table 6.12]
$S_{I,rig,y}$	=	383698.95	[kN*m]	Stiffness of a rigid connection	[5.2.2.5]

$$S_{j,ini,y} < S_{j,rig,y} \text{ SEMI-RIGID}$$

[5.2.2.5.(2)]

WEAKEST COMPONENT:

BASE PLATE - BENDING

Connection conforms to the code

Ratio 0.98



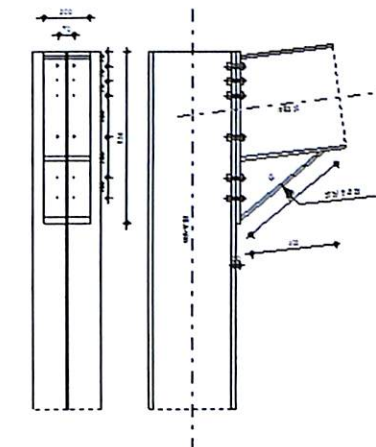
Autodesk Robot Structural Analysis Professional 2016

Design of fixed beam-to-column connection

EN 1993-1-8:2005/AC:2009



Ratio
0.99



GENERAL

Connection no.: 2
Connection name: KS1
Structure node: 2
Structure bars: 2, 5

GEOMETRY

COLUMN

Section: HEA 400
Bar no.: 2
a = -90.0 [Deg] Inclination angle
h_c = 390 [mm] Height of column section
b_{fc} = 300 [mm] Width of column section
t_{wc} = 11 [mm] Thickness of the web of column section
t_{fc} = 19 [mm] Thickness of the flange of column section
r_c = 27 [mm] Radius of column section fillet
A_c = 158.98 [cm²] Cross-sectional area of a column
I_{xc} = 45069.40 [cm⁴] Moment of inertia of the column section
Material: S235
f_{yc} = 235.00 [MPa] Resistance

BEAM

Section: IPE 500
Bar no.: 5
a = 7.0 [Deg] Inclination angle
h_b = 500 [mm] Height of beam section
b_f = 200 [mm] Width of beam section
t_{wb} = 10 [mm] Thickness of the web of beam section
t_{fb} = 16 [mm] Thickness of the flange of beam section
r_b = 21 [mm] Radius of beam section fillet

$r_b =$	21	[mm]	Radius of beam section fillet
$A_b =$	115.52	[cm ²]	Cross-sectional area of a beam
$I_{xb} =$	48198.50	[cm ⁴]	Moment of inertia of the beam section
Material: S235			
$f_{yb} =$	235.00	[MPa]	Resistance

BOLTS

The shear plane passes through the UNTHREADED portion of the bolt.			
$d =$	16	[mm]	Bolt diameter
Class =	6.8		Bolt class
$F_{tRd} =$	67.82	[kN]	Tensile resistance of a bolt
$n_1 =$	2		Number of bolt columns
$n_v =$	6		Number of bolt rows
$h_1 =$	70	[mm]	Distance between first bolt and upper edge of front plate
Horizontal spacing $e_i = 70$ [mm]			
Vertical spacing $p_i = 70;70;200;200;100$ [mm]			

PLATE

$h_p =$	834	[mm]	Plate height
$b_p =$	200	[mm]	Plate width
$t_p =$	20	[mm]	Plate thickness
Material: S 235			
$f_{yp} =$	235.00	[MPa]	Resistance

LOWER STIFFENER

$w_d =$	200	[mm]	Plate width
$t_{fd} =$	16	[mm]	Flange thickness
$h_d =$	300	[mm]	Plate height
$t_{wd} =$	12	[mm]	Web thickness
$l_d =$	400	[mm]	Plate length
$\alpha =$	41.3	[Deg]	Inclination angle
Material: S 235			
$f_{ybu} =$	235.00	[MPa]	Resistance

FILLET WELDS

$a_w =$	8	[mm]	Web weld
$a_f =$	12	[mm]	Flange weld
$a_{fd} =$	5	[mm]	Horizontal weld

MATERIAL FACTORS

$\gamma_{M0} =$	1.00	Partial safety factor	[2.2]
$\gamma_{M1} =$	1.00	Partial safety factor	[2.2]
$\gamma_{M2} =$	1.25	Partial safety factor	[2.2]
$\gamma_{M3} =$	1.25	Partial safety factor	[2.2]

LOADS

Ultimate limit state

Case: 673: ULS/659=1*1.35 + 6*1.35 + 3*1.05 + 10*1.50 (1+6)*1.35+3*1.05+10*1.50

$M_{b1,Ed} =$	318.30	[kN*m]	Bending moment in the right beam
$V_{b1,Ed} =$	120.42	[kN]	Shear force in the right beam
$N_{b1,Ed} =$	-62.59	[kN]	Axial force in the right beam
$M_{c1,Ed} =$	318.30	[kN*m]	Bending moment in the lower column
$V_{c1,Ed} =$	62.59	[kN]	Shear force in the lower column
$N_{c1,Ed} =$	-120.42	[kN]	Axial force in the lower column

RESULTS

BEAM RESISTANCES

COMPRESSION			
$A_b =$	115.52	[cm ²]	Area
$N_{cb,Rd} = A_b f_{yb} / \gamma_{M0}$			
$N_{cb,Rd} =$	2714.77	[kN]	Design compressive resistance of the section
SHEAR			
$A_{vb} =$	95.87	[cm ²]	Shear area

$V_{cb,Rd} = A_{vb} (f_{yb} / \phi)$						
$V_{cb,Rd} =$	1300.79	[kN]	Design sectional resistance for shear			EN1993-1-1:[6.2.6.(2)]
$V_{b1,Ed} / V_{cb,Rd} \leq 1.0$			0.09 < 1.00	verified		(0.09)
BENDING - PLASTIC MOMENT (WITHOUT BRACKETS)						
$W_{plb} =$	2194.26	[cm ³]	Plastic section modulus			EN1993-1-1:[6.2.5.(2)]
$M_{b,pl,Rd} = W_{plb} f_{yb} / \gamma_{M0}$						
$M_{b,pl,Rd} =$	515.65	[kN*m]	Plastic resistance of the section for bending (without stiffeners)			EN1993-1-1:[6.2.5.(2)]
BENDING ON THE CONTACT SURFACE WITH PLATE OR CONNECTED ELEMENT						
$W_{pl} =$	3843.94	[cm ³]	Plastic section modulus			EN1993-1-1:[6.2.5]
$M_{cb,Rd} = W_{pl} f_{yb} / \gamma_{M0}$						
$M_{cb,Rd} =$	903.33	[kN*m]	Design resistance of the section for bending			EN1993-1-1:[6.2.5]
FLANGE AND WEB - COMPRESSION						
$M_{cb,Rd} =$	903.33	[kN*m]	Design resistance of the section for bending			EN1993-1-1:[6.2.5]
$h_f =$	785	[mm]	Distance between the centroids of flanges			[6.2.6.7.(1)]
$F_{c,fb,Rd} = M_{cb,Rd} / h_f$						[6.2.6.7.(1)]
$F_{c,fb,Rd} =$	1150.71	[kN]	Resistance of the compressed flange and web			[6.2.6.7.(1)]
WEB OR BRACKET FLANGE - COMPRESSION - LEVEL OF THE BEAM BOTTOM FLANGE						
Bearing:						
$b =$	7.0	[Deg]	Angle between the front plate and the beam			
$g =$	41.3	[Deg]	Inclination angle of the bracket plate			[6.2.6.2.(1)]
$b_{eff,c,wb} =$	247	[mm]	Effective width of the web for compression			EN1993-1-1:[6.2.6.(3)]
$A_{vb} =$	59.87	[cm ²]	Shear area			[6.2.6.2.(1)]
$w =$	0.90		Reduction factor for interaction with shear			[6.2.6.2.(2)]
$\sigma_{com,Ed} =$	146.08	[MPa]	Maximum compressive stress in web			[6.2.6.2.(2)]
$k_{wc} =$	1.00		Reduction factor conditioned by compressive stresses			
$F_{c,wb,Rd1} = [w k_{wc} b_{eff,c,wb} t_{wb} f_{yb} / \gamma_{M0}] \cos(g) / \sin(g - b)$						[6.2.6.2.(1)]
$F_{c,wb,Rd1} =$	712.50	[kN]	Beam web resistance			[6.2.6.2.(1)]
Buckling:						
$d_{wb} =$	426	[mm]	Height of compressed web			[6.2.6.2.(1)]
$l_p =$	0.99		Plate slenderness of an element			[6.2.6.2.(1)]
$r =$	0.80		Reduction factor for element buckling			[6.2.6.2.(1)]
$F_{c,wb,Rd2} = [w k_{wc} r b_{eff,c,wb} t_{wb} f_{yb} / \gamma_{M1}] \cos(g) / \sin(g - b)$						[6.2.6.2.(1)]
$F_{c,wb,Rd2} =$	573.37	[kN]	Beam web resistance			
Resistance of the bracket flange						
$F_{c,wb,Rd3} = b_b t_b f_{yb} / (0.8 \gamma_{M0})$						[6.2.6.7.(1)]
$F_{c,wb,Rd3} =$	940.00	[kN]	Resistance of the bracket flange			
Final resistance:						
$F_{c,wb,Rd,low} = \text{Min} (F_{c,wb,Rd1} , F_{c,wb,Rd2} , F_{c,wb,Rd3})$						[6.2.6.2.(1)]
$F_{c,wb,Rd,low} =$	573.37	[kN]	Beam web resistance			
COLUMN RESISTANCES						
WEB PANEL - SHEAR						
$M_{b1,Ed} =$	318.30	[kN*m]	Bending moment (right beam)			[5.3.(3)]
$M_{b2,Ed} =$	0.00	[kN*m]	Bending moment (left beam)			[5.3.(3)]
$V_{c1,Ed} =$	62.59	[kN]	Shear force (lower column)			[5.3.(3)]
$V_{c2,Ed} =$	0.00	[kN]	Shear force (upper column)			[5.3.(3)]
$z =$	708	[mm]	Lever arm			[6.2.5]
$V_{wp,Ed} = (M_{b1,Ed} - M_{b2,Ed}) / z - (V_{c1,Ed} - V_{c2,Ed}) / 2$						[5.3.(3)]
$V_{wp,Ed} =$	418.23	[kN]	Shear force acting on the web panel			
$A_{vs} =$	57.33	[cm ²]	Shear area of the column web			EN1993-1-1:[6.2.6.(3)]
$A_{vc} =$	57.33	[cm ²]	Shear area			EN1993-1-1:[6.2.6.(3)]
$V_{wp,Rd} = 0.9 (f_{y,wc} A_{vc} + f_{y,wp} A_{vp} + f_{ys} A_{vd}) / \phi$						[6.2.6.1]
$V_{wp,Rd} =$	700.03	[kN]	Resistance of the column web panel for shear			
$V_{wp,Ed} / V_{wp,Rd} \leq 1.0$			0.60 < 1.00	verified		(0.60)

WEB - TRANSVERSE COMPRESSION - LEVEL OF THE BEAM BOTTOM FLANGE

Bearing:					
t_{wc}	=	11	[mm]	Effective thickness of the column web	[6.2.6.2.(6)]
$b_{eff,c,wc}$	=	325	[mm]	Effective width of the web for compression	[6.2.6.2.(1)]
A_{vc}	=	57.33	[cm ²]	Shear area	EN1993-1-1:[6.2.6.(3)]
w	=	0.81		Reduction factor for interaction with shear	[6.2.6.2.(1)]
$\sigma_{com,Ed}$	=	112.81	[MPa]	Maximum compressive stress in web	[6.2.6.2.(2)]
k_{wc}	=	1.00		Reduction factor conditioned by compressive stresses	[6.2.6.2.(2)]
$F_{c,wc,Rd1} = w k_{wc} b_{eff,c,wc} t_{wc} f_{yc} / \gamma_{M0}$					
$F_{c,wc,Rd1}$	=	685.02	[kN]	Column web resistance	[6.2.6.2.(1)]
Buckling:					
d_{wc}	=	296	[mm]	Height of compressed web	[6.2.6.2.(1)]
l_p	=	0.88		Plate slenderness of an element	[6.2.6.2.(1)]
r	=	0.88		Reduction factor for element buckling	[6.2.6.2.(1)]
$F_{c,wb,Rd2} = w k_{wc} r b_{eff,c,wc} t_{wc} f_{yc} / \gamma_{M1}$					
$F_{c,wb,Rd2}$	=	600.37	[kN]	Column web resistance	[6.2.6.2.(1)]
Final resistance:					
$F_{c,wc,Rd,low} = \text{Min} (F_{c,wc,Rd1} , F_{c,wb,Rd2})$					
$F_{c,wc,Rd}$	=	600.37	[kN]	Column web resistance	[6.2.6.2.(1)]

WEB - TRANSVERSE COMPRESSION - LEVEL OF THE BEAM TOP FLANGE

Bearing:					
t_{wc}	=	11	[mm]	Effective thickness of the column web	[6.2.6.2.(6)]
$b_{eff,c,wc}$	=	320	[mm]	Effective width of the web for compression	[6.2.6.2.(1)]
A_{vc}	=	57.33	[cm ²]	Shear area	EN1993-1-1:[6.2.6.(3)]
w	=	0.82		Reduction factor for interaction with shear	[6.2.6.2.(1)]
$\sigma_{com,Ed}$	=	112.81	[MPa]	Maximum compressive stress in web	[6.2.6.2.(2)]
k_{wc}	=	1.00		Reduction factor conditioned by compressive stresses	[6.2.6.2.(2)]
$F_{c,wc,Rd1} = w k_{wc} b_{eff,c,wc} t_{wc} f_{yc} / \gamma_{M0}$					
$F_{c,wc,Rd1}$	=	677.73	[kN]	Column web resistance	[6.2.6.2.(1)]
Buckling:					
d_{wc}	=	298	[mm]	Height of compressed web	[6.2.6.2.(1)]
l_p	=	0.88		Plate slenderness of an element	[6.2.6.2.(1)]
r	=	0.88		Reduction factor for element buckling	[6.2.6.2.(1)]
$F_{c,wb,Rd2} = w k_{wc} r b_{eff,c,wc} t_{wc} f_{yc} / \gamma_{M1}$					
$F_{c,wb,Rd2}$	=	597.35	[kN]	Column web resistance	[6.2.6.2.(1)]
Final resistance:					
$F_{c,wc,Rd,upp} = \text{Min} (F_{c,wc,Rd1} , F_{c,wb,Rd2})$					
$F_{c,wc,Rd,upp}$	=	597.35	[kN]	Column web resistance	[6.2.6.2.(1)]

GEOMETRICAL PARAMETERS OF A CONNECTION

EFFECTIVE LENGTHS AND PARAMETERS - COLUMN FLANGE

Nr	m	m_x	e	e_x	p	$l_{eff,cp}$	$l_{eff,nc}$	$l_{eff,1}$	$l_{eff,2}$	$l_{eff,cp,g}$	$l_{eff,nc,g}$	$l_{eff,1,g}$	$l_{eff,2,g}$
1	0	-	115	-	70	50	158	50	158	95	105	95	105
2	8	-	115	-	70	50	175	50	175	140	70	70	70
3	8	-	115	-	135	50	175	50	175	140	70	70	70
4	8	-	115	-	200	50	175	50	175	400	200	200	200
5	8	-	115	-	150	50	175	50	175	200	100	100	100
6	8	-	115	-	100	50	175	50	175	125	138	125	138

EFFECTIVE LENGTHS AND PARAMETERS - FRONT PLATE

Nr	m	m_x	e	e_x	p	$l_{eff,cp}$	$l_{eff,nc}$	$l_{eff,1}$	$l_{eff,2}$	$l_{eff,cp,g}$	$l_{eff,nc,g}$	$l_{eff,1,g}$	$l_{eff,2,g}$
1	21	-	65	-	70	131	167	131	167	135	119	119	119
2	21	-	65	-	70	131	165	131	165	140	70	70	70
3	21	-	65	-	135	131	165	131	165	270	135	135	135
4	21	-	65	-	200	131	165	131	165	400	200	200	200
5	21	-	65	-	150	131	165	131	165	300	150	150	150
6	21	-	65	-	100	131	165	131	165	165	132	132	132

- m – Bolt distance from the web
- m_y – Bolt distance from the beam flange

- e – Bolt distance from the outer edge
- e_x – Bolt distance from the horizontal outer edge
- p – Distance between bolts
- l_{eff,cp} – Effective length for a single bolt in the circular failure mode
- l_{eff,nc} – Effective length for a single bolt in the non-circular failure mode
- l_{eff,1} – Effective length for a single bolt for mode 1
- l_{eff,2} – Effective length for a single bolt for mode 2
- l_{eff,cp,g} – Effective length for a group of bolts in the circular failure mode
- l_{eff,nc,g} – Effective length for a group of bolts in the non-circular failure mode
- l_{eff,1,g} – Effective length for a group of bolts for mode 1
- l_{eff,2,g} – Effective length for a group of bolts for mode 2

CONNECTION RESISTANCE FOR COMPRESSION

$$N_{j,Rd} = \min (N_{cb,Rd} 2 F_{c,wb,Rd,low} \cdot 2 F_{c,wc,Rd,low} \cdot 2 F_{c,wc,Rd,upp})$$
$$N_{j,Rd} = 1146.75 \quad [kN] \quad \text{Connection resistance for compression} \quad [6.2]$$
$$N_{b1,Ed} / N_{j,Rd} \leq 1.0 \quad 0.05 < 1.00 \quad \text{verified} \quad (0.05)$$

CONNECTION RESISTANCE FOR BENDING

$$F_{t,Rd} = 67.82 \quad [kN] \quad \text{Bolt resistance for tension} \quad [Table 3.4]$$
$$B_{p,Rd} = 247.55 \quad [kN] \quad \text{Punching shear resistance of a bolt} \quad [Table 3.4]$$

- F_{l,fc,Rd} – column flange resistance due to bending
- F_{l,wc,Rd} – column web resistance due to tension
- F_{t,ep,Rd} – resistance of the front plate due to bending
- F_{l,wb,Rd} – resistance of the web in tension

$$F_{l,fc,Rd} = \min (F_{T,1,fc,Rd} \cdot F_{T,2,fc,Rd} \cdot F_{T,3,fc,Rd}) \quad [6.2.6.4] , [Tab.6.2]$$
$$F_{l,wc,Rd} = w \cdot b_{eff,t,wc} \cdot t_{wc} \cdot f_{yc} / \gamma_{M0} \quad [6.2.6.3.(1)]$$
$$F_{t,ep,Rd} = \min (F_{T,1,ep,Rd} \cdot F_{T,2,ep,Rd} \cdot F_{T,3,ep,Rd}) \quad [6.2.6.5] , [Tab.6.2]$$
$$F_{l,wb,Rd} = b_{eff,t,wb} \cdot t_{wb} \cdot f_{yb} / \gamma_{M0} \quad [6.2.6.8.(1)]$$

RESISTANCE OF THE BOLT ROW NO. 1

F _{t1,Rd,comp} - Formula	F _{t1,Rd,comp}	Component
F _{l,fc,Rd(1)} = 135.65	135.65	Column flange - tension
F _{l,wc,Rd(1)} = 127.56	127.56	Column web - tension
F _{t,ep,Rd(1)} = 135.65	135.65	Front plate - tension
F _{l,wb,Rd(1)} = 314.00	314.00	Beam web - tension
B _{p,Rd} = 495.09	495.09	Bolts due to shear punching
V _{wp,Rd} /b = 700.03	700.03	Web panel - shear
F _{c,wc,Rd} = 600.37	600.37	Column web - compression
F _{c,fb,Rd} = 1150.71	1150.71	Beam flange - compression
F _{c,wb,Rd} = 573.37	573.37	Beam web - compression
F _{t1,Rd} = Min (F _{t1,Rd,comp})	127.56	Bolt row resistance

RESISTANCE OF THE BOLT ROW NO. 2

F _{t2,Rd,comp} - Formula	F _{t2,Rd,comp}	Component
F _{l,fc,Rd(2)} = 135.65	135.65	Column flange - tension
F _{l,wc,Rd(2)} = 127.56	127.56	Column web - tension
F _{t,ep,Rd(2)} = 135.65	135.65	Front plate - tension
F _{l,wb,Rd(2)} = 314.00	314.00	Beam web - tension
B _{p,Rd} = 495.09	495.09	Bolts due to shear punching
V _{wp,Rd} /b - Σ ¹ F _{ij,Rd} = 700.03 - 127.56	572.47	Web panel - shear
F _{c,wc,Rd} - Σ ¹ F _{ij,Rd} = 600.37 - 127.56	472.81	Column web - compression
F _{c,fb,Rd} - Σ ¹ F _{ij,Rd} = 1150.71 - 127.56	1023.15	Beam flange - compression
F _{c,wb,Rd} - Σ ¹ F _{ij,Rd} = 573.37 - 127.56	445.81	Beam web - compression
F _{l,fc,Rd(2 + 1)} - Σ ¹ F _{ij,Rd} = 271.30 - 127.56	143.73	Column flange - tension - group
F _{l,wc,Rd(2 + 1)} - Σ ¹ F _{ij,Rd} = 422.47 - 127.56	294.91	Column web - tension - group
F _{t,ep,Rd(2 + 1)} - Σ ¹ F _{ij,Rd} = 271.30 - 127.56	143.73	Front plate - tension - group
F _{t2,Rd} = Min (F _{t2,Rd,comp})	127.56	Bolt row resistance

F_{t2,Rd,comp} - Formula	F_{t2,Rd,comp}	Component
$F_{t,wb,Rd(2+1)} - \sum_1^1 F_{tj,Rd} = 454.16 - 127.56$	326.60	Beam web - tension - group
$F_{t2,Rd} = \text{Min} (F_{t2,Rd,comp})$	127.56	Bolt row resistance

RESISTANCE OF THE BOLT ROW NO. 3

F_{t3,Rd,comp} - Formula	F_{t3,Rd,comp}	Component
$F_{t,fc,Rd(3)} = 135.65$	135.65	Column flange - tension
$F_{t,wc,Rd(3)} = 127.56$	127.56	Column web - tension
$F_{t,ep,Rd(3)} = 135.65$	135.65	Front plate - tension
$F_{t,wb,Rd(3)} = 314.00$	314.00	Beam web - tension
$B_{p,Rd} = 495.09$	495.09	Bolts due to shear punching
$V_{wp,Rd/b} - \sum_1^2 F_{tj,Rd} = 700.03 - 255.12$	444.91	Web panel - shear
$F_{c,wc,Rd} - \sum_1^2 F_{tj,Rd} = 600.37 - 255.12$	345.25	Column web - compression
$F_{c,fb,Rd} - \sum_1^2 F_{tj,Rd} = 1150.71 - 255.12$	895.58	Beam flange - compression
$F_{c,wb,Rd} - \sum_1^2 F_{tj,Rd} = 573.37 - 255.12$	318.25	Beam web - compression
$F_{t,fc,Rd(3+2)} - \sum_2^2 F_{tj,Rd} = 271.30 - 127.56$	143.73	Column flange - tension - group
$F_{t,wc,Rd(3+2)} - \sum_2^2 F_{tj,Rd} = 346.03 - 127.56$	218.47	Column web - tension - group
$F_{t,fc,Rd(3+2+1)} - \sum_2^1 F_{tj,Rd} = 406.94 - 255.12$	151.82	Column flange - tension - group
$F_{t,wc,Rd(3+2+1)} - \sum_2^1 F_{tj,Rd} = 558.20 - 255.12$	303.07	Column web - tension - group
$F_{t,ep,Rd(3+2)} - \sum_2^2 F_{tj,Rd} = 271.30 - 127.56$	143.73	Front plate - tension - group
$F_{t,wb,Rd(3+2)} - \sum_2^2 F_{tj,Rd} = 491.38 - 127.56$	363.82	Beam web - tension - group
$F_{t,ep,Rd(3+2+1)} - \sum_2^1 F_{tj,Rd} = 406.94 - 255.12$	151.82	Front plate - tension - group
$F_{t,wb,Rd(3+2+1)} - \sum_2^1 F_{tj,Rd} = 777.75 - 255.12$	522.63	Beam web - tension - group
$F_{t3,Rd} = \text{Min} (F_{t3,Rd,comp})$	127.56	Bolt row resistance

RESISTANCE OF THE BOLT ROW NO. 4

F_{t4,Rd,comp} - Formula	F_{t4,Rd,comp}	Component
$F_{t,fc,Rd(4)} = 135.65$	135.65	Column flange - tension
$F_{t,wc,Rd(4)} = 127.56$	127.56	Column web - tension
$F_{t,ep,Rd(4)} = 135.65$	135.65	Front plate - tension
$F_{t,wb,Rd(4)} = 314.00$	314.00	Beam web - tension
$B_{p,Rd} = 495.09$	495.09	Bolts due to shear punching
$V_{wp,Rd/b} - \sum_1^3 F_{tj,Rd} = 700.03 - 382.69$	317.34	Web panel - shear
$F_{c,wc,Rd} - \sum_1^3 F_{tj,Rd} = 600.37 - 382.69$	217.68	Column web - compression
$F_{c,fb,Rd} - \sum_1^3 F_{tj,Rd} = 1150.71 - 382.69$	768.02	Beam flange - compression
$F_{c,wb,Rd} - \sum_1^3 F_{tj,Rd} = 573.37 - 382.69$	190.69	Beam web - compression
$F_{t,fc,Rd(4+3)} - \sum_3^3 F_{tj,Rd} = 271.30 - 127.56$	143.73	Column flange - tension - group
$F_{t,wc,Rd(4+3)} - \sum_3^3 F_{tj,Rd} = 600.94 - 127.56$	473.38	Column web - tension - group
$F_{t,fc,Rd(4+3+2)} - \sum_3^2 F_{tj,Rd} = 406.94 - 255.12$	151.82	Column flange - tension - group
$F_{t,wc,Rd(4+3+2)} - \sum_3^2 F_{tj,Rd} = 705.20 - 255.12$	450.08	Column web - tension - group
$F_{t,fc,Rd(4+3+2+1)} - \sum_3^1 F_{tj,Rd} = 542.59 - 382.69$	159.91	Column flange - tension - group
$F_{t,wc,Rd(4+3+2+1)} - \sum_3^1 F_{tj,Rd} = 824.23 - 382.69$	441.54	Column web - tension - group
$F_{t,ep,Rd(4+3)} - \sum_3^3 F_{tj,Rd} = 271.30 - 127.56$	143.73	Front plate - tension - group
$F_{t,wb,Rd(4+3)} - \sum_3^3 F_{tj,Rd} = 803.00 - 127.56$	675.43	Beam web - tension - group
$F_{t,ep,Rd(4+3+2)} - \sum_3^2 F_{tj,Rd} = 406.94 - 255.12$	151.82	Front plate - tension - group
$F_{t,wb,Rd(4+3+2)} - \sum_3^2 F_{tj,Rd} = 970.78 - 255.12$	715.66	Beam web - tension - group
$F_{t,ep,Rd(4+3+2+1)} - \sum_3^1 F_{tj,Rd} = 542.59 - 382.69$	159.91	Front plate - tension - group
$F_{t,wb,Rd(4+3+2+1)} - \sum_3^1 F_{tj,Rd} = 1257.15 - 382.69$	874.47	Beam web - tension - group
$F_{t4,Rd} = \text{Min} (F_{t4,Rd,comp})$	127.56	Bolt row resistance

RESISTANCE OF THE BOLT ROW NO. 5

F_{t5,Rd,comp} - Formula	F_{t5,Rd,comp}	Component
$F_{t,fc,Rd(5)} = 135.65$	135.65	Column flange - tension
$F_{t,wc,Rd(5)} = 127.56$	127.56	Column web - tension
$F_{t,ep,Rd(5)} = 135.65$	135.65	Front plate - tension
$F_{t,wb,Rd(5)} = 314.00$	314.00	Beam web - tension
$B_{p,Rd} = 495.09$	495.09	Bolts due to shear punching
$F_{t5,Rd} = \text{Min} (F_{t5,Rd,comp})$	63.13	Bolt row resistance

Ft5,Rd,comp - Formula	Ft5,Rd,comp	Component
$V_{wp,Rd}/b - \sum 1^4 F_{ti,Rd} = 700.03 - 510.25$	189.78	Web panel - shear
$F_{c,wc,Rd} - \sum 1^4 F_{ij,Rd} = 600.37 - 510.25$	90.12	Column web - compression
$F_{c,fb,Rd} - \sum 1^4 F_{ij,Rd} = 1150.71 - 510.25$	640.46	Beam flange - compression
$F_{c,wb,Rd} - \sum 1^4 F_{ij,Rd} = 573.37 - 510.25$	63.13	Beam web - compression
$F_{t,fc,Rd}(5+4) - \sum 4^4 F_{ij,Rd} = 271.30 - 127.56$	143.73	Column flange - tension - group
$F_{t,wc,Rd}(5+4) - \sum 4^4 F_{ij,Rd} = 648.33 - 127.56$	520.77	Column web - tension - group
$F_{t,fc,Rd}(5+4+3) - \sum 4^3 F_{ij,Rd} = 406.94 - 255.12$	151.82	Column flange - tension - group
$F_{t,wc,Rd}(5+4+3) - \sum 4^3 F_{ij,Rd} = 743.42 - 255.12$	488.29	Column web - tension - group
$F_{t,fc,Rd}(5+4+3+2) - \sum 4^2 F_{ij,Rd} = 542.59 - 382.69$	159.91	Column flange - tension - group
$F_{t,wc,Rd}(5+4+3+2) - \sum 4^2 F_{ij,Rd} = 819.44 - 382.69$	436.75	Column web - tension - group
$F_{t,fc,Rd}(5+4+3+2+1) - \sum 4^1 F_{ij,Rd} = 678.24 - 510.25$	167.99	Column flange - tension - group
$F_{t,wc,Rd}(5+4+3+2+1) - \sum 4^1 F_{ij,Rd} = 905.32 - 510.25$	395.07	Column web - tension - group
$F_{t,ep,Rd}(5+4) - \sum 4^4 F_{ij,Rd} = 271.30 - 127.56$	143.73	Front plate - tension - group
$F_{t,wb,Rd}(5+4) - \sum 4^4 F_{ij,Rd} = 838.95 - 127.56$	711.39	Beam web - tension - group
$F_{t,ep,Rd}(5+4+3) - \sum 4^3 F_{ij,Rd} = 406.94 - 255.12$	151.82	Front plate - tension - group
$F_{t,wb,Rd}(5+4+3) - \sum 4^3 F_{ij,Rd} = 1162.55 - 255.12$	907.42	Beam web - tension - group
$F_{t,ep,Rd}(5+4+3+2) - \sum 4^2 F_{ij,Rd} = 542.59 - 382.69$	159.91	Front plate - tension - group
$F_{t,wb,Rd}(5+4+3+2) - \sum 4^2 F_{ij,Rd} = 1330.34 - 382.69$	947.65	Beam web - tension - group
$F_{t,ep,Rd}(5+4+3+2+1) - \sum 4^1 F_{ij,Rd} = 678.24 - 510.25$	167.99	Front plate - tension - group
$F_{t,wb,Rd}(5+4+3+2+1) - \sum 4^1 F_{ij,Rd} = 1616.70 - 510.25$	1106.45	Beam web - tension - group
$F_{t5,Rd} = \text{Min} (F_{t5,Rd,comp})$	63.13	Bolt row resistance

RESISTANCE OF THE BOLT ROW NO. 6

Ft6,Rd,comp - Formula	Ft6,Rd,comp	Component
$F_{t,fc,Rd}(6) = 135.65$	135.65	Column flange - tension
$F_{t,wc,Rd}(6) = 127.56$	127.56	Column web - tension
$F_{t,ep,Rd}(6) = 135.65$	135.65	Front plate - tension
$F_{t,wb,Rd}(6) = 314.00$	314.00	Beam web - tension
$B_{p,Rd} = 495.09$	495.09	Bolts due to shear punching
$V_{wp,Rd}/b - \sum 1^5 F_{ti,Rd} = 700.03 - 573.37$	126.66	Web panel - shear
$F_{c,wc,Rd} - \sum 1^5 F_{ij,Rd} = 600.37 - 573.37$	27.00	Column web - compression
$F_{c,fb,Rd} - \sum 1^5 F_{ij,Rd} = 1150.71 - 573.37$	577.34	Beam flange - compression
$F_{c,wb,Rd} - \sum 1^5 F_{ij,Rd} = 573.37 - 573.37$	0.00	Beam web - compression
$F_{t,fc,Rd}(6+5) - \sum 5^5 F_{ij,Rd} = 271.30 - 63.13$	208.17	Column flange - tension - group
$F_{t,wc,Rd}(6+5) - \sum 5^5 F_{ij,Rd} = 545.10 - 63.13$	481.98	Column web - tension - group
$F_{t,fc,Rd}(6+5+4) - \sum 5^4 F_{ij,Rd} = 406.94 - 190.69$	216.26	Column flange - tension - group
$F_{t,wc,Rd}(6+5+4) - \sum 5^4 F_{ij,Rd} = 817.18 - 190.69$	626.49	Column web - tension - group
$F_{t,fc,Rd}(6+5+4+3) - \sum 5^3 F_{ij,Rd} = 542.59 - 318.25$	224.34	Column flange - tension - group
$F_{t,wc,Rd}(6+5+4+3) - \sum 5^3 F_{ij,Rd} = 878.10 - 318.25$	559.85	Column web - tension - group
$F_{t,fc,Rd}(6+5+4+3+2) - \sum 5^2 F_{ij,Rd} = 678.24 - 445.81$	232.43	Column flange - tension - group
$F_{t,wc,Rd}(6+5+4+3+2) - \sum 5^2 F_{ij,Rd} = 926.60 - 445.81$	480.78	Column web - tension - group
$F_{t,fc,Rd}(6+5+4+3+2+1) - \sum 5^1 F_{ij,Rd} = 813.89 - 573.37$	240.51	Column flange - tension - group
$F_{t,wc,Rd}(6+5+4+3+2+1) - \sum 5^1 F_{ij,Rd} = 981.82 - 573.37$	408.45	Column web - tension - group
$F_{t,ep,Rd}(6+5) - \sum 5^5 F_{ij,Rd} = 271.30 - 63.13$	208.17	Front plate - tension - group
$F_{t,wb,Rd}(6+5) - \sum 5^5 F_{ij,Rd} = 676.73 - 63.13$	613.60	Beam web - tension - group
$F_{t,ep,Rd}(6+5+4) - \sum 5^4 F_{ij,Rd} = 406.94 - 190.69$	216.26	Front plate - tension - group
$F_{t,wb,Rd}(6+5+4) - \sum 5^4 F_{ij,Rd} = 1156.13 - 190.69$	965.44	Beam web - tension - group
$F_{t,ep,Rd}(6+5+4+3) - \sum 5^3 F_{ij,Rd} = 542.59 - 318.25$	224.34	Front plate - tension - group
$F_{t,wb,Rd}(6+5+4+3) - \sum 5^3 F_{ij,Rd} = 1479.72 - 318.25$	1161.47	Beam web - tension - group
$F_{t,ep,Rd}(6+5+4+3+2) - \sum 5^2 F_{ij,Rd} = 678.24 - 445.81$	232.43	Front plate - tension - group
$F_{t,wb,Rd}(6+5+4+3+2) - \sum 5^2 F_{ij,Rd} = 1647.51 - 445.81$	1201.70	Beam web - tension - group
$F_{t,ep,Rd}(6+5+4+3+2+1) - \sum 5^1 F_{ij,Rd} = 813.89 - 573.37$	240.51	Front plate - tension - group
$F_{t,wb,Rd}(6+5+4+3+2+1) - \sum 5^1 F_{ij,Rd} = 1933.88 - 573.37$	1360.51	Beam web - tension - group
$F_{t6,Rd} = \text{Min} (F_{t6,Rd,comp})$	0.00	Bolt row resistance

SUMMARY TABLE OF FORCES

Nr	h _j	F _{tj,Rd}	F _{t,fc,Rd}	F _{t,wc,Rd}	F _{t,ep,Rd}	F _{t,wb,Rd}	F _{t,Rd}	B _{p,Rd}
1	743	127.56	135.65	127.56	135.65	314.00	135.65	495.09
2	673	127.56	135.65	127.56	135.65	314.00	135.65	495.09
3	603	127.56	135.65	127.56	135.65	314.00	135.65	495.09
4	403	127.56	135.65	127.56	135.65	314.00	135.65	495.09
5	203	63.13	135.65	127.56	135.65	314.00	135.65	495.09
6	103	-	135.65	127.56	135.65	314.00	135.65	495.09

CONNECTION RESISTANCE FOR BENDING M_{j,Rd}

M _{j,Rd} = ∑ h _j F _{tj,Rd}								
M _{j,Rd} =	321.81	[kN*m]	Connection resistance for bending					[6.2]
M _{b1,Ed} / M _{j,Rd} ≤ 1.0			0.99 < 1.00	verified				(0.99)

CONNECTION RESISTANCE FOR SHEAR

a _v =	0.60		Coefficient for calculation of F _{v,Rd}					[Table 3.4]
b _{lf} =	0.88		Reduction factor for long connections					[3.8]
F _{v,Rd} =	50.67	[kN]	Shear resistance of a single bolt					[Table 3.4]
F _{t,Rd,max} =	67.82	[kN]	Tensile resistance of a single bolt					[Table 3.4]
F _{b,Rd,int} =	218.88	[kN]	Bearing resistance of an intermediate bolt					[Table 3.4]
F _{b,Rd,ext} =	218.88	[kN]	Bearing resistance of an outermost bolt					[Table 3.4]

Nr	F _{tj,Rd,N}	F _{tj,Ed,N}	F _{tj,Rd,M}	F _{tj,Ed,M}	F _{tj,Ed}	F _{vj,Rd}
1	135.65	-10.43	127.56	126.17	115.74	39.58
2	135.65	-10.43	127.56	126.17	115.74	39.58
3	135.65	-10.43	127.56	126.17	115.74	39.58
4	135.65	-10.43	127.56	126.17	115.74	39.58
5	135.65	-10.43	63.13	62.44	52.00	73.59
6	135.65	-10.43	0.00	0.00	-10.43	101.34

F_{tj,Rd,N} – Bolt row resistance for simple tension
F_{tj,Ed,N} – Force due to axial force in a bolt row
F_{tj,Rd,M} – Bolt row resistance for simple bending
F_{tj,Ed,M} – Force due to moment in a bolt row
F_{tj,Ed} – Maximum tensile force in a bolt row
F_{vj,Rd} – Reduced bolt row resistance

F_{tj,Ed,N} = N_{j,Ed} F_{tj,Rd,N} / N_{j,Rd}
F_{tj,Ed,M} = M_{j,Ed} F_{tj,Rd,M} / M_{j,Rd}
F_{tj,Ed} = F_{tj,Ed,N} + F_{tj,Ed,M}
F_{vj,Rd} = Min (n_h F_{v,Ed} (1 - F_{tj,Ed} / (1.4 n_h F_{t,Rd,max}), n_h F_{v,Rd} , n_h F_{b,Rd}))

V _{j,Rd} = n _h ∑ _{i=1} ⁿ F _{vj,Rd}								[Table 3.4]
V _{j,Rd} =	333.23	[kN]	Connection resistance for shear					[Table 3.4]
V _{b1,Ed} / V _{j,Rd} ≤ 1.0			0.36 < 1.00	verified				(0.36)

WELD RESISTANCE

A _w =	213.53	[cm ²]	Area of all welds					[4.5.3.2(2)]
A _{wy} =	100.27	[cm ²]	Area of horizontal welds					[4.5.3.2(2)]
A _{wz} =	113.26	[cm ²]	Area of vertical welds					[4.5.3.2(2)]
I _{wy} =	151126.35	[cm ⁴]	Moment of inertia of the weld arrangement with respect to the hor. axis					[4.5.3.2(5)]
σ _{max=τ_{max}} =	-64.27	[MPa]	Normal stress in a weld					[4.5.3.2(5)]
σ _τ =	-64.27	[MPa]	Stress in a vertical weld					[4.5.3.2(5)]
τ _{ij} =	10.63	[MPa]	Tangent stress					[4.5.3.2(7)]
h _w =	0.80		Correlation coefficient					
ϕ			128.54 < 360.00	verified				(0.36)
ϕ			129.85 < 360.00	verified				(0.36)
σ ≤ 0.9*σ _{fu} /β _{M2}			64.27 < 259.20	verified				(0.25)

CONNECTION STIFFNESS

t _{wash} =	4	[mm]	Washer thickness					[6.2.6.3.(2)]
h _{head} =	12	[mm]	Bolt head height					[6.2.6.3.(2)]
h _{nut} =	16	[mm]	Bolt nut height					[6.2.6.3.(2)]
L _b =	61	[mm]	Bolt length					[6.2.6.3.(2)]

k₁₀ = 4 [mm] Stiffness coefficient of bolts [6.3.2.(1)]

STIFFNESSES OF BOLT ROWS

Nr	h _j	k ₃	k ₄	k ₅	k _{eff,j}	k _{eff,j} h _j	k _{eff,j} h _j ²
1	743	1	621	95	1	6.32	469.55
2	673	1	621	56	1	5.69	382.82
3	603	1	621	104	1	5.13	309.53
4	403	1	621	104	1	3.43	138.27
5	203	1	621	104	1	1.73	35.10
6	103	1	621	104	1	0.88	9.04
Sum						23.17	1344.31

$k_{eff,j} = 1 / (\sum 1 / k_{i,j})$ [6.3.3.1.(2)]

$z_{eq} = \sum k_{eff,j} h_j^2 / \sum k_{eff,j} h_j$
z_{eq} = 580 [mm] Equivalent force arm [6.3.3.1.(3)]

$k_{eq} = \sum k_{eff,j} h_j / z_{eq}$
k_{eq} = 4 [mm] Equivalent stiffness coefficient of a bolt arrangement [6.3.3.1.(1)]

A_{vc} = 57.33 [cm²] Shear area EN1993-1-1:6.2.6.(3)

b = 1.00 Transformation parameter [5.3.(7)]

z = 580 [mm] Lever arm [6.2.5]

k₁ = 4 [mm] Stiffness coefficient of the column web panel subjected to shear [6.3.2.(1)]

b_{eff,c,wc} = 298 [mm] Effective width of the web for compression [6.2.6.2.(1)]

t_{wc} = 11 [mm] Effective thickness of the column web [6.2.6.2.(6)]

d_c = 352 [mm] Height of compressed web [6.2.6.2.(1)]

k₂ = 7 [mm] Stiffness coefficient of the compressed column web [6.3.2.(1)]

S_{j,ini} = E z_{eq}² / ∑ (1 / k₁ + 1 / k₂ + 1 / k_{eq}) [6.3.1.(4)]

S_{j,ini} = 105488.24 [kN*m] Initial rotational stiffness [6.3.1.(4)]

m = 2.90 Stiffness coefficient of a connection [6.3.1.(6)]

S_j = S_{j,ini} / m [6.3.1.(4)]

S_j = 36359.89 [kN*m] Final rotational stiffness [6.3.1.(4)]

Connection classification due to stiffness.

S_{j,rig} = 89305.97 [kN*m] Stiffness of a rigid connection [5.2.2.5]

S_{j,pin} = 5581.62 [kN*m] Stiffness of a pinned connection [5.2.2.5]

S_{j,ini} > S_{j,rig} RIGID

WEAKEST COMPONENT:

BEAM WEB OR BRACKET FLANGE - COMPRESSION

REMARKS

Distance of bolts from an edge is too large. 124 [mm] > 120 [mm]

Connection conforms to the code Ratio 0.99

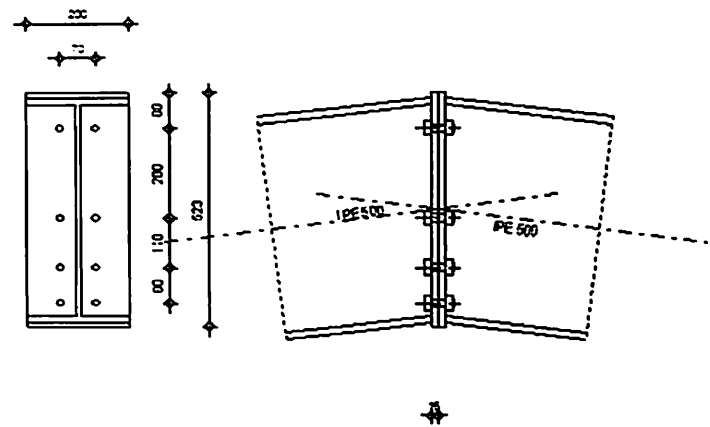


Autodesk Robot Structural Analysis Professional 2016

Design of fixed beam-to-beam connection
EN 1993-1-8:2005/AC:2009



Ratio
0.92



GENERAL

Connection no.: 3
Connection name: Tar
Structure node: 7
Structure bars: 5, 6

GEOMETRY

LEFT SIDE

BEAM

Section: IPE 500
Bar no.: 5
a = -173.0 [Deg] Inclination angle
h_{bl} = 500 [mm] Height of beam section
b_{tbl} = 200 [mm] Width of beam section
t_{wbl} = 10 [mm] Thickness of the web of beam section
t_{tbl} = 16 [mm] Thickness of the flange of beam section
r_{bl} = 21 [mm] Radius of beam section fillet
A_{bl} = 115.52 [cm²] Cross-sectional area of a beam
I_{xbl} = 48198.50 [cm⁴] Moment of inertia of the beam section
Material: S235
f_{yb} = 235.00 [MPa] Resistance

RIGHT SIDE

BEAM

Section: IPE 500
Bar no.: 6
a = -7.0 [Deg] Inclination angle
h_{br} = 500 [mm] Height of beam section
b_{tbr} = 200 [mm] Width of beam section
t_{wbr} = 10 [mm] Thickness of the web of beam section
t_{tbr} = 16 [mm] Thickness of the flange of beam section
r_{br} = 21 [mm] Radius of beam section fillet
A_{br} = 115.52 [cm²] Cross-sectional area of a beam
I_{xbr} = 48198.50 [cm⁴] Moment of inertia of the beam section
Material: S235
f_{yb} = 235.00 [MPa] Resistance

BOLTS

The shear plane passes through the UNTHREADED portion of the bolt.
d = 16 [mm] Bolt diameter
Class = 10.9 Bolt class
F_{tRd} = 113.04 [kN] Tensile resistance of a bolt
n_n = 2 Number of bolt columns
n_v = 4 Number of bolt rows
h₁ = 80 [mm] Distance between first bolt and upper edge of front plate
Horizontal spacing e_i = 70 [mm]
Vertical spacing p_i = 200; 110; 80 [mm]

PLATE

h_{pr} = 523 [mm] Plate height

b _{pr} =	200	[mm]	Plate width
t _{pr} =	16	[mm]	Plate thickness
Material: S 235			
f _{pr} =	235.00	[MPa]	Resistance

Fillet welds

a _w =	5	[mm]	Web weld
a _f =	8	[mm]	Flange weld

Material factors

g _{M0} =	1.00	Partial safety factor	[2.2]
g _{M1} =	1.00	Partial safety factor	[2.2]
g _{M2} =	1.25	Partial safety factor	[2.2]
g _{M3} =	1.25	Partial safety factor	[2.2]

LOADS

Ultimate limit state

Case: 673: ULS/659=1*1.35 + 6*1.35 + 3*1.05 + 10*1.50 (1+6)*1.35+3*1.05+10*1.50

M _{b1,Ed} =	-180.81	[kN*m]	Bending moment in the right beam
V _{b1,Ed} =	5.79	[kN]	Shear force in the right beam
N _{b1,Ed} =	-62.59	[kN]	Axial force in the right beam

RESULTS

BEAM RESISTANCES

COMPRESSION

A _b =	115.52	[cm ²]	Area	EN1993-1-1:[6.2.4]
N _{cb,Rd} = A _b f _{yb} / g _{M0}				
N _{cb,Rd} =	2714.77	[kN]	Design compressive resistance of the section	EN1993-1-1:[6.2.4]

SHEAR

A _{vb} =	59.87	[cm ²]	Shear area	EN1993-1-1:[6.2.6.(3)]
V _{cb,Rd} = A _{vb} (f _{yb} / √3)				
V _{cb,Rd} =	812.35	[kN]	Design sectional resistance for shear	EN1993-1-1:[6.2.6.(2)]
V _{b1,Ed} / V _{cb,Rd} ≤ 1.0	0.01 < 1.00		verified	(0.01)

BENDING - PLASTIC MOMENT (WITHOUT BRACKETS)

W _{plb} =	2194.26	[cm ³]	Plastic section modulus	EN1993-1-1:[6.2.5.(2)]
M _{b,pl,Rd} = W _{plb} f _{yb} / g _{M0}				
M _{b,pl,Rd} =	515.65	[kN*m]	Plastic resistance of the section for bending (without stiffeners)	EN1993-1-1:[6.2.5.(2)]

BENDING ON THE CONTACT SURFACE WITH PLATE OR CONNECTED ELEMENT

W _{pl} =	2194.26	[cm ³]	Plastic section modulus	EN1993-1-1:[6.2.5]
M _{cb,Rd} = W _{pl} f _{yb} / g _{M0}				
M _{cb,Rd} =	515.65	[kN*m]	Design resistance of the section for bending	EN1993-1-1:[6.2.5]

FLANGE AND WEB - COMPRESSION

M _{cb,Rd} =	515.65	[kN*m]	Design resistance of the section for bending	EN1993-1-1:[6.2.5]
h _f =	484	[mm]	Distance between the centroids of flanges	[6.2.6.7.(1)]
F _{c,fb,Rd} = M _{cb,Rd} / h _f				
F _{c,fb,Rd} =	1065.39	[kN]	Resistance of the compressed flange and web	[6.2.6.7.(1)]

GEOMETRICAL PARAMETERS OF A CONNECTION

EFFECTIVE LENGTHS AND PARAMETERS - FRONT PLATE

Nr	m	m _x	e	e _x	p	l _{eff,cp}	l _{eff,nc}	l _{eff,1}	l _{eff,2}	l _{eff,cp,g}	l _{eff,nc,g}	l _{eff,1,g}	l _{eff,2,g}
1	24	-	65	-	80	152	194	152	194	156	145	145	145
2	24	-	65	-	95	152	178	152	178	190	95	95	95
3	24	-	65	-	155	152	178	152	178	310	155	155	155
4	24	-	65	-	200	152	178	152	178	276	189	189	189

- m – Bolt distance from the web
- m_x – Bolt distance from the beam flange
- e – Bolt distance from the outer edge
- e_x – Bolt distance from the horizontal outer edge

- p – Distance between bolts
 $l_{eff,cp}$ – Effective length for a single bolt in the circular failure mode
 $l_{eff,nc}$ – Effective length for a single bolt in the non-circular failure mode
 $l_{eff,1}$ – Effective length for a single bolt for mode 1
 $l_{eff,2}$ – Effective length for a single bolt for mode 2
 $l_{eff,cp,g}$ – Effective length for a group of bolts in the circular failure mode
 $l_{eff,nc,g}$ – Effective length for a group of bolts in the non-circular failure mode
 $l_{eff,1,g}$ – Effective length for a group of bolts for mode 1
 $l_{eff,2,g}$ – Effective length for a group of bolts for mode 2

CONNECTION RESISTANCE FOR COMPRESSION

$N_{j,Rd} = \text{Min} (N_{cb,Rd})$				[6.2]
$N_{j,Rd} =$	2714.77	[kN]	Connection resistance for compression	
$N_{b1,Ed} / N_{j,Rd} \leq 1.0$		0.02 < 1.00	verified	(0.02)

CONNECTION RESISTANCE FOR BENDING

$F_{t,Rd} =$	113.04	[kN]	Bolt resistance for tension	[Table 3.4]
$B_{p,Rd} =$	208.46	[kN]	Punching shear resistance of a bolt	[Table 3.4]
$F_{t,fc,Rd}$	– column flange resistance due to bending			
$F_{t,wc,Rd}$	– column web resistance due to tension			
$F_{t,ep,Rd}$	– resistance of the front plate due to bending			
$F_{t,wb,Rd}$	– resistance of the web in tension			
$F_{t,fc,Rd} = \text{Min} (F_{T,1,fc,Rd} \cdot F_{T,2,fc,Rd} \cdot F_{T,3,fc,Rd})$				[6.2.6.4] , [Tab.6.2]
$F_{t,wc,Rd} = w \cdot b_{eff,t,wc} \cdot t_{wc} \cdot f_{yc} / g_{M0}$				[6.2.6.3.(1)]
$F_{t,ep,Rd} = \text{Min} (F_{T,1,ep,Rd} \cdot F_{T,2,ep,Rd} \cdot F_{T,3,ep,Rd})$				[6.2.6.5] , [Tab.6.2]
$F_{t,wb,Rd} = b_{eff,t,wb} \cdot t_{wb} \cdot f_{yb} / g_{M0}$				[6.2.6.8.(1)]

RESISTANCE OF THE BOLT ROW NO. 1

$F_{t1,Rd,comp}$ - Formula	$F_{t1,Rd,comp}$	Component
$F_{t,ep,Rd(1)} = 226.08$	226.08	Front plate - tension
$F_{t,wb,Rd(1)} = 365.12$	365.12	Beam web - tension
$B_{p,Rd} = 416.92$	416.92	Bolts due to shear punching
$F_{c,fb,Rd} = 1065.39$	1065.39	Beam flange - compression
$F_{t1,Rd} = \text{Min} (F_{t1,Rd,comp})$	226.08	Bolt row resistance

RESISTANCE OF THE BOLT ROW NO. 2

$F_{t2,Rd,comp}$ - Formula	$F_{t2,Rd,comp}$	Component
$F_{t,ep,Rd(2)} = 223.88$	223.88	Front plate - tension
$F_{t,wb,Rd(2)} = 365.12$	365.12	Beam web - tension
$B_{p,Rd} = 416.92$	416.92	Bolts due to shear punching
$F_{c,fb,Rd} - \sum_1^1 F_{ij,Rd} = 1065.39 - 226.08$	839.31	Beam flange - compression
$F_{t,ep,Rd(2+1)} - \sum_1^1 F_{ij,Rd} = 383.46 - 226.08$	157.38	Front plate - tension - group
$F_{t,wb,Rd(2+1)} - \sum_1^1 F_{ij,Rd} = 574.88 - 226.08$	348.80	Beam web - tension - group
$F_{t2,Rd} = \text{Min} (F_{t2,Rd,comp})$	157.38	Bolt row resistance

RESISTANCE OF THE BOLT ROW NO. 3

$F_{t3,Rd,comp}$ - Formula	$F_{t3,Rd,comp}$	Component
$F_{t,ep,Rd(3)} = 223.88$	223.88	Front plate - tension
$F_{t,wb,Rd(3)} = 365.12$	365.12	Beam web - tension
$B_{p,Rd} = 416.92$	416.92	Bolts due to shear punching
$F_{c,fb,Rd} - \sum_1^2 F_{ij,Rd} = 1065.39 - 383.46$	681.94	Beam flange - compression
$F_{t,ep,Rd(3+2)} - \sum_2^2 F_{ij,Rd} = 389.06 - 157.38$	231.69	Front plate - tension - group
$F_{t,wb,Rd(3+2)} - \sum_2^2 F_{ij,Rd} = 599.25 - 157.38$	441.87	Beam web - tension - group
$F_{t,ep,Rd(3+2+1)} - \sum_2^1 F_{ij,Rd} = 594.53 - 383.46$	211.07	Front plate - tension - group
$F_{t,wb,Rd(3+2+1)} - \sum_2^1 F_{ij,Rd} = 946.42 - 383.46$	562.96	Beam web - tension - group
$F_{t3,Rd} = \text{Min} (F_{t3,Rd,comp})$	211.07	Bolt row resistance

Additional reduction of the bolt row resistance

[6.2.7.2.(9)]

$$F_{t3,Rd} = F_{t1,Rd} \cdot h_3/h_1$$

$$F_{t3,Rd} = 131.03 \quad [\text{kN}] \quad \text{Reduced bolt row resistance}$$

RESISTANCE OF THE BOLT ROW NO. 4

$F_{t4,Rd,comp}$ - Formula	$F_{t4,Rd,comp}$	Component
$F_{lep,Rd(4)} = 223.88$	223.88	Front plate - tension
$F_{lwb,Rd(4)} = 365.12$	365.12	Beam web - tension
$B_{p,Rd} = 416.92$	416.92	Bolts due to shear punching
$F_{c,b,Rd} - \sum 1^3 F_{ij,Rd} = 1065.39 - 514.49$	550.90	Beam flange - compression
$F_{lep,Rd(4+3)} - \sum 3^3 F_{ij,Rd} = 440.96 - 131.03$	309.93	Front plate - tension - group
$F_{lwb,Rd(4+3)} - \sum 3^3 F_{ij,Rd} = 824.83 - 131.03$	693.80	Beam web - tension - group
$F_{lep,Rd(4+3+2)} - \sum 3^2 F_{ij,Rd} = 618.95 - 288.41$	330.54	Front plate - tension - group
$F_{lwb,Rd(4+3+2)} - \sum 3^2 F_{ij,Rd} = 1052.55 - 288.41$	764.14	Beam web - tension - group
$F_{lep,Rd(4+3+2+1)} - \sum 3^1 F_{ij,Rd} = 824.42 - 514.49$	309.93	Front plate - tension - group
$F_{lwb,Rd(4+3+2+1)} - \sum 3^1 F_{ij,Rd} = 1399.72 - 514.49$	885.23	Beam web - tension - group
$F_{t4,Rd} = \text{Min}(F_{t4,Rd,comp})$	223.88	Bolt row resistance

Additional reduction of the bolt row resistance

[6.2.7.2.(9)]

$$F_{t4,Rd} = F_{t1,Rd} \cdot h_4/h_1$$

$$F_{t4,Rd} = 30.99 \quad [\text{kN}] \quad \text{Reduced bolt row resistance}$$

SUMMARY TABLE OF FORCES

Nr	h_j	$F_{tj,Rd}$	$F_{t,fc,Rd}$	$F_{t,wc,Rd}$	$F_{t,ep,Rd}$	$F_{t,wb,Rd}$	$F_{t,Rd}$	$B_{p,Rd}$
1 452					226.08	365.12	226.08	416.92
2 372		226.08	-	-	223.88	365.12	226.08	416.92
3 262		157.38	-	-	223.88	365.12	226.08	416.92
4 62		131.03	-	-	223.88	365.12	226.08	416.92
		30.99	-	-				

CONNECTION RESISTANCE FOR BENDING $M_{j,Rd}$

$$M_{j,Rd} = \sum h_j F_{tj,Rd}$$

$$M_{j,Rd} = 196.95 \quad [\text{kN}\cdot\text{m}] \quad \text{Connection resistance for bending}$$

$$M_{b1,Ed} / M_{j,Rd} \leq 1.0 \quad 0.92 < 1.00$$

verified

[6.2]

(0.92)

CONNECTION RESISTANCE FOR SHEAR

$a_v =$	0.60	Coefficient for calculation of $F_{v,Rd}$
$b_{Lr} =$	0.95	Reduction factor for long connections
$F_{v,Rd} =$	91.99	Shear resistance of a single bolt
$F_{t,Rd,max} =$	113.04	Tensile resistance of a single bolt
$F_{b,Rd,int} =$	184.32	Bearing resistance of an intermediate bolt
$F_{b,Rd,ext} =$	179.95	Bearing resistance of an outermost bolt

[Table 3.4]

[3.8]

[Table 3.4]

[Table 3.4]

[Table 3.4]

[Table 3.4]

$F_{Rd,exl} =$		184.32	[kN]	Bearing resistance of an outermost bolt					$F_{vj,Rd}$
		179.95	[kN]	Bearing resistance of an outermost bolt					
N_r					$F_{tj,Ed,M}$		$F_{tj,Ed}$		
1							191.90		72.43
2	$F_{tj,Rd,N}$		$F_{tj,Ed,N}$	$F_{tj,Rd,M}$	207.55		128.83		109.09
3					144.47		104.64		123.15
4					120.29		12.80		176.53
					28.45				

- $F_{tj,Rd,N}$ - Bolt row resistance for simple tension
- $F_{tj,Ed,N}$ - Force due to axial force in a bolt row
- $F_{tj,Rd,M}$ - Bolt row resistance for simple bending
- $F_{tj,Ed,M}$ - Force due to moment in a bolt row
- $F_{tj,Ed}$ - Maximum tensile force in a bolt row
- $F_{vj,Rd}$ - Reduced bolt row resistance

$$F_{tj,Ed,N} = N_{j,Ed} F_{tj,Rd,N} / N_{j,Rd}$$

$$F_{tj,Ed,M} = M_{j,Ed} F_{tj,Rd,M} / M_{j,Rd}$$

$$F_{tj,Ed} = F_{tj,Ed,N} + F_{tj,Ed,M}$$

$$F_{vj,Rd} = \text{Min}(n_h F_{v,Ed} (1 - F_{tj,Ed} / (1.4 n_h F_{t,Rd,max})) \cdot n_h F_{v,Rd} \cdot n_h F_{b,Rd}))$$

$V_{j,Rd} = n \sum_{i=1}^n F_{v,j,Rd}$
 $V_{j,Rd} = 481.21$ [kN] Connection resistance for shear verified
 $V_{b1,Ed} / V_{j,Rd} \leq 1.0$ 0.01 < 1.00
 [Table 3.4]
 [Table 3.4]
 (0.01)

WELD RESISTANCE
 $A_w = 98.57$ [cm²] Area of all welds [4.5.3.2(2)]
 $A_{wH} = 55.65$ [cm²] Area of horizontal welds [4.5.3.2(2)]
 $A_{wV} = 42.92$ [cm²] Area of vertical welds [4.5.3.2(5)]
 $I_{wV} = 40238.84$ [cm⁴] Moment of inertia of the weld arrangement with respect to the hor. axis [4.5.3.2(5)]
 $s_{max} = t_{max} = 87.64$ [MPa] Normal stress in a weld [4.5.3.2(5)]
 $s_{tV} = 74.53$ [MPa] Stress in a vertical weld [4.5.3.2(7)]
 $t_{tV} = 1.35$ [MPa] Tangent stress (0.49)
 $t_{tV} = 0.80$ Correlation coefficient (0.41)
 verified
 verified
 verified
 $s \leq 0.9 \cdot t_{tV} / \text{cm}^2$ 175.29 < 360.00 (0.34)
 149.08 < 360.00
 87.64 < 259.20

CONNECTION STIFFNESS
 $t_{wash} = 4$ [mm] Washer thickness [6.2.6.3.(2)]
 $h_{head} = 12$ [mm] Bolt head height [6.2.6.3.(2)]
 $h_{nut} = 16$ [mm] Bolt nut height [6.2.6.3.(2)]
 $L_b = 54$ [mm] Bolt length [6.3.2.(1)]
 $k_{t0} = 5$ [mm] Stiffness coefficient of bolts

STIFFNESSES OF BOLT ROWS

Nr	h _j	k ₃	k ₄	k ₅	k _{eff,j}	k _{eff,j} h _j	k _{eff,j} h _j ²
1	452	Y	Y	37	4	16.84	761.16
2	372	Y	Y	25	3	12.55	466.83
3	262	Y	Y	39	4	9.86	258.22
4	62	Y	Y	39	4	2.33	14.44
					Sum	41.58	1500.64

$k_{eff,j} = 1 / (\sum_{i=1}^5 (1 / k_{i,j}))$ [6.3.3.1.(3)]
 $z_{eq} = \sum_j k_{eff,j} h_j^2 / \sum_j k_{eff,j} h_j$ [6.3.3.1.(1)]
 $z_{eq} = 361$ [mm] Equivalent force arm
 $k_{eq} = \sum_j k_{eff,j} h_j / z_{eq}$ [6.3.1.(4)]
 $k_{eq} = 12$ [mm] Equivalent stiffness coefficient of a bolt arrangement [6.3.1.(4)]
 $S_{j,ini} = E z_{eq}^2 k_{eq}$ [6.3.1.(6)]
 $S_{j,ini} = 315134.98$ [kN*m] Initial rotational stiffness [6.3.1.(4)]
 $m = S_j = S_{j,ini} / m$ [6.3.1.(4)]
 $S_j = 2.37$ Stiffness coefficient of a connection [5.2.2.5]
 $S_j = 132846.92$ [kN*m] Final rotational stiffness [5.2.2.5]

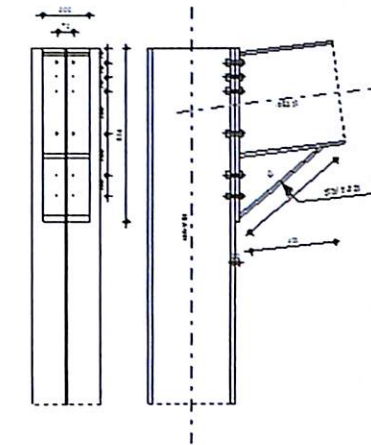
Connection classification due to stiffness.
 $S_{j,rig} = 89305.97$ [kN*m] Stiffness of a rigid connection
 $S_{j,pin} = 5581.62$ [kN*m] Stiffness of a pinned connection
 $S_{j,ini} \geq S_{j,rig}$ RIGID

WEAKEST COMPONENT:
 FRONT PLATE - TENSION
 Connection conforms to the code
 Ratio 0.92



Design of fixed beam-to-column connection

EN 1993-1-8:2005/AC:2009

Ratio
0.92

GENERAL

Connection no.: 4
Connection name: KS2
Structure node: 4
Structure bars: 3, 6

GEOMETRY

COLUMN

Section: HEA 400

Bar no.: 3

a =	-90.0	[Deg]	Inclination angle
h _c =	390	[mm]	Height of column section
b _{fc} =	300	[mm]	Width of column section
t _{wc} =	11	[mm]	Thickness of the web of column section
t _{fc} =	19	[mm]	Thickness of the flange of column section
r _c =	27	[mm]	Radius of column section fillet
A _c =	158.98	[cm ²]	Cross-sectional area of a column
I _{xc} =	45069.40	[cm ⁴]	Moment of inertia of the column section
Material: S235			
f _{yc} =	235.00	[MPa]	Resistance

BEAM

Section: IPE 500

Bar no.: 6

a =	7.0	[Deg]	Inclination angle
h _b =	500	[mm]	Height of beam section
b _f =	200	[mm]	Width of beam section
t _{wb} =	10	[mm]	Thickness of the web of beam section
t _{fb} =	16	[mm]	Thickness of the flange of beam section
r _b =	21	[mm]	Radius of beam section fillet
r _b =	21	[mm]	Radius of beam section fillet
A _b =	115.52	[cm ²]	Cross-sectional area of a beam
I _{xb} =	48198.50	[cm ⁴]	Moment of inertia of the beam section
Material: S235			
f _{yb} =	235.00	[MPa]	Resistance

BOLTS

The shear plane passes through the UNTHREADED portion of the bolt.

d =	16	[mm]	Bolt diameter
Class =	6.8		Bolt class
F _{TRd} =	67.82	[kN]	Tensile resistance of a bolt

$n_1 = 2$ Number of bolt columns
 $n_y = 6$ Number of bolt rows
 $h_1 = 70$ [mm] Distance between first bolt and upper edge of front plate
Horizontal spacing $e_1 = 70$ [mm]
Vertical spacing $p_1 = 70; 70; 200; 200; 100$ [mm]

PLATE

$h_p = 834$ [mm] Plate height
 $b_p = 200$ [mm] Plate width
 $t_p = 20$ [mm] Plate thickness
Material: S 235
 $f_{yp} = 235.00$ [MPa] Resistance

LOWER STIFFENER

$w_d = 200$ [mm] Plate width
 $t_{fd} = 16$ [mm] Flange thickness
 $h_d = 300$ [mm] Plate height
 $t_{wd} = 12$ [mm] Web thickness
 $l_d = 400$ [mm] Plate length
 $\alpha = 41.3$ [Deg] Inclination angle
Material: S 235
 $f_{ybu} = 235.00$ [MPa] Resistance

FILLET WELDS

$a_w = 8$ [mm] Web weld
 $a_f = 12$ [mm] Flange weld
 $a_{fd} = 5$ [mm] Horizontal weld

MATERIAL FACTORS

$\gamma_{M0} = 1.00$	Partial safety factor	[2.2]
$\gamma_{M1} = 1.00$	Partial safety factor	[2.2]
$\gamma_{M2} = 1.25$	Partial safety factor	[2.2]
$\gamma_{M3} = 1.25$	Partial safety factor	[2.2]

LOADS

Ultimate limit state

Case: 692: tk9 (1+6)*1.35+(5+10)*1.50+(8+13)*0.90

$M_{b1,Ed} = 297.34$	[kN*m]	Bending moment in the right beam
$V_{b1,Ed} = 117.41$	[kN]	Shear force in the right beam
$N_{b1,Ed} = -83.29$	[kN]	Axial force in the right beam
$M_{c1,Ed} = 297.34$	[kN*m]	Bending moment in the lower column
$V_{c1,Ed} = 83.29$	[kN]	Shear force in the lower column
$N_{c1,Ed} = -117.41$	[kN]	Axial force in the lower column

RESULTS

BEAM RESISTANCES

COMPRESSION

$A_b = 115.52$	[cm ²]	Area	EN1993-1-1:[6.2.4]
$N_{cb,Rd} = A_b f_{yb} / \gamma_{M0}$			
$N_{cb,Rd} = 2714.77$	[kN]	Design compressive resistance of the section	EN1993-1-1:[6.2.4]

SHEAR

$A_{vb} = 95.87$	[cm ²]	Shear area	EN1993-1-1:[6.2.6.(3)]
$V_{cb,Rd} = A_{vb} (f_{yb} / \sqrt{3})$			
$V_{cb,Rd} = 1300.79$	[kN]	Design sectional resistance for shear	EN1993-1-1:[6.2.6.(2)]
$V_{b1,Ed} / V_{cb,Rd} \leq 1.0$		0.09 < 1.00	verified (0.09)

BENDING - PLASTIC MOMENT (WITHOUT BRACKETS)

$W_{plb} = 2194.26$	[cm ³]	Plastic section modulus	EN1993-1-1:[6.2.5.(2)]
$M_{b,pl,Rd} = W_{plb} f_{yb} / \gamma_{M0}$			
$M_{b,pl,Rd} = 515.65$	[kN*m]	Plastic resistance of the section for bending (without stiffeners)	EN1993-1-1:[6.2.5.(2)]

BENDING ON THE CONTACT SURFACE WITH PLATE OR CONNECTED ELEMENT

$W_{pl} = 3843.94$	[cm ³]	Plastic section modulus	EN1993-1-1:[6.2.5]
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$$M_{cb,Rd} = W_{pl} f_{yb} / \gamma_{M0}$$

$$M_{cb,Rd} = 903.33 \quad [kN \cdot m] \quad \text{Design resistance of the section for bending}$$

EN1993-1-1:[6.2.5]

FLANGE AND WEB - COMPRESSION

$$M_{cb,Rd} = 903.33 \quad [kN \cdot m] \quad \text{Design resistance of the section for bending}$$

$$h_f = 785 \quad [mm] \quad \text{Distance between the centroids of flanges}$$

$$F_{c,fb,Rd} = M_{cb,Rd} / h_f$$

$$F_{c,fb,Rd} = 1150.71 \quad [kN] \quad \text{Resistance of the compressed flange and web}$$

EN1993-1-1:[6.2.5]

[6.2.6.7.(1)]

[6.2.6.7.(1)]

WEB OR BRACKET FLANGE - COMPRESSION - LEVEL OF THE BEAM BOTTOM FLANGE

Bearing:

$$b = 7.0 \quad [Deg] \quad \text{Angle between the front plate and the beam}$$

$$g = 41.3 \quad [Deg] \quad \text{Inclination angle of the bracket plate}$$

$$b_{eff,c,wb} = 247 \quad [mm] \quad \text{Effective width of the web for compression}$$

$$A_{vb} = 59.87 \quad [cm^2] \quad \text{Shear area}$$

$$w = 0.90 \quad \text{Reduction factor for interaction with shear}$$

$$s_{com,Ed} = 138.61 \quad [MPa] \quad \text{Maximum compressive stress in web}$$

$$k_{wc} = 1.00 \quad \text{Reduction factor conditioned by compressive stresses}$$

$$F_{c,wb,Rd1} = [w k_{wc} b_{eff,c,wb} t_{wb} f_{yb} / \gamma_{M0}] \cos(g) / \sin(g - b)$$

$$F_{c,wb,Rd1} = 712.50 \quad [kN] \quad \text{Beam web resistance}$$

[6.2.6.2.(1)]
EN1993-1-1:[6.2.6.(3)]

[6.2.6.2.(1)]

[6.2.6.2.(2)]

[6.2.6.2.(2)]

[6.2.6.2.(2)]

[6.2.6.2.(1)]

Buckling:

$$d_{wb} = 426 \quad [mm] \quad \text{Height of compressed web}$$

$$l_p = 0.99 \quad \text{Plate slenderness of an element}$$

$$r = 0.80 \quad \text{Reduction factor for element buckling}$$

$$F_{c,wb,Rd2} = [w k_{wc} r b_{eff,c,wb} t_{wb} f_{yb} / \gamma_{M1}] \cos(g) / \sin(g - b)$$

$$F_{c,wb,Rd2} = 573.37 \quad [kN] \quad \text{Beam web resistance}$$

[6.2.6.2.(1)]

[6.2.6.2.(1)]

[6.2.6.2.(1)]

[6.2.6.2.(1)]

Resistance of the bracket flange

$$F_{c,wb,Rd3} = b_b t_b f_{yb} / (0.8 \gamma_{M0})$$

$$F_{c,wb,Rd3} = 940.00 \quad [kN] \quad \text{Resistance of the bracket flange}$$

[6.2.6.7.(1)]

Final resistance:

$$F_{c,wb,Rd,low} = \min(F_{c,wb,Rd1}, F_{c,wb,Rd2}, F_{c,wb,Rd3})$$

$$F_{c,wb,Rd,low} = 573.37 \quad [kN] \quad \text{Beam web resistance}$$

[6.2.6.2.(1)]

COLUMN RESISTANCES

WEB PANEL - SHEAR

$$M_{b1,Ed} = 297.34 \quad [kN \cdot m] \quad \text{Bending moment (right beam)}$$

$$M_{b2,Ed} = 0.00 \quad [kN \cdot m] \quad \text{Bending moment (left beam)}$$

$$V_{c1,Ed} = 83.29 \quad [kN] \quad \text{Shear force (lower column)}$$

$$V_{c2,Ed} = 0.00 \quad [kN] \quad \text{Shear force (upper column)}$$

$$z = 708 \quad [mm] \quad \text{Lever arm}$$

$$V_{wp,Ed} = (M_{b1,Ed} - M_{b2,Ed}) / z - (V_{c1,Ed} - V_{c2,Ed}) / 2$$

$$V_{wp,Ed} = 378.29 \quad [kN] \quad \text{Shear force acting on the web panel}$$

$$A_{vs} = 57.33 \quad [cm^2] \quad \text{Shear area of the column web}$$

$$A_{vc} = 57.33 \quad [cm^2] \quad \text{Shear area}$$

$$V_{wp,Rd} = 0.9 \cdot (f_{y,wc} \cdot A_{vc} + f_{y,wp} \cdot A_{vp} + f_{ys} \cdot A_{vd}) / \gamma_{M0}$$

$$V_{wp,Rd} = 700.03 \quad [kN] \quad \text{Resistance of the column web panel for shear}$$

[5.3.(3)]

[5.3.(3)]

[5.3.(3)]

[5.3.(3)]

[6.2.5]

[5.3.(3)]

EN1993-1-1:[6.2.6.(3)]
EN1993-1-1:[6.2.6.(3)]

[6.2.6.1]

(0.54)

$$V_{wp,Ed} / V_{wp,Rd} \leq 1.0 \quad 0.54 < 1.00 \quad \text{verified}$$

WEB - TRANSVERSE COMPRESSION - LEVEL OF THE BEAM BOTTOM FLANGE

Bearing:

$$t_{wc} = 11 \quad [mm] \quad \text{Effective thickness of the column web}$$

$$b_{eff,c,wc} = 325 \quad [mm] \quad \text{Effective width of the web for compression}$$

$$A_{vc} = 57.33 \quad [cm^2] \quad \text{Shear area}$$

$$w = 0.81 \quad \text{Reduction factor for interaction with shear}$$

$$s_{com,Ed} = 105.69 \quad [MPa] \quad \text{Maximum compressive stress in web}$$

$$k_{wc} = 1.00 \quad \text{Reduction factor conditioned by compressive stresses}$$

$$F_{c,wc,Rd1} = w k_{wc} b_{eff,c,wc} t_{wc} f_{yc} / \gamma_{M0}$$

$$F_{c,wc,Rd1} = 685.02 \quad [kN] \quad \text{Column web resistance}$$

[6.2.6.2.(6)]
[6.2.6.2.(1)]
EN1993-1-1:[6.2.6.(3)]
[6.2.6.2.(1)]
[6.2.6.2.(2)]
[6.2.6.2.(2)]

[6.2.6.2.(1)]

$M_{cb,Rd} = W_{pl} f_{yb} / \gamma_{M0}$
 $M_{cb,Rd} = 903.33$ [kN*m] Design resistance of the section for bending EN1993-1-1:[6.2.5]

FLANGE AND WEB - COMPRESSION
 $M_{cb,Rd} = 903.33$ [kN*m] Design resistance of the section for bending EN1993-1-1:[6.2.5]
 $h_f = 785$ [mm] Distance between the centroids of flanges [6.2.6.7.(1)]
 $F_{c,fb,Rd} = M_{cb,Rd} / h_f$
 $F_{c,fb,Rd} = 1150.71$ [kN] Resistance of the compressed flange and web [6.2.6.7.(1)]

WEB OR BRACKET FLANGE - COMPRESSION - LEVEL OF THE BEAM BOTTOM FLANGE

Bearing:
 $b = 7.0$ [Deg] Angle between the front plate and the beam
 $g = 41.3$ [Deg] Inclination angle of the bracket plate [6.2.6.2.(1)]
 $b_{eff,c,wb} = 247$ [mm] Effective width of the web for compression EN1993-1-1:[6.2.6.(3)]
 $A_{vb} = 59.87$ [cm²] Shear area [6.2.6.2.(1)]
 $w = 0.90$ Reduction factor for interaction with shear [6.2.6.2.(2)]
 $\sigma_{com,Ed} = 138.61$ [MPa] Maximum compressive stress in web [6.2.6.2.(2)]
 $k_{wc} = 1.00$ Reduction factor conditioned by compressive stresses
 $F_{c,wb,Rd1} = [w k_{wc} b_{eff,c,wb} t_{wb} f_{yb} / \gamma_{M0}] \cos(g) / \sin(g - b)$ [6.2.6.2.(1)]
 $F_{c,wb,Rd1} = 712.50$ [kN] Beam web resistance
Buckling:
 $d_{wb} = 426$ [mm] Height of compressed web [6.2.6.2.(1)]
 $l_p = 0.99$ Plate slenderness of an element [6.2.6.2.(1)]
 $r = 0.80$ Reduction factor for element buckling [6.2.6.2.(1)]
 $F_{c,wb,Rd2} = [w k_{wc} r b_{eff,c,wb} t_{wb} f_{yb} / \gamma_{M1}] \cos(g) / \sin(g - b)$ [6.2.6.2.(1)]
 $F_{c,wb,Rd2} = 573.37$ [kN] Beam web resistance
Resistance of the bracket flange
 $F_{c,wb,Rd3} = b_b t_b f_{yb} / (0.8 \gamma_{M0})$ [6.2.6.7.(1)]
 $F_{c,wb,Rd3} = 940.00$ [kN] Resistance of the bracket flange
Final resistance:
 $F_{c,wb,Rd,low} = \min(F_{c,wb,Rd1}, F_{c,wb,Rd2}, F_{c,wb,Rd3})$ [6.2.6.2.(1)]
 $F_{c,wb,Rd,low} = 573.37$ [kN] Beam web resistance

COLUMN RESISTANCES

WEB PANEL - SHEAR
 $M_{b1,Ed} = 297.34$ [kN*m] Bending moment (right beam) [5.3.(3)]
 $M_{b2,Ed} = 0.00$ [kN*m] Bending moment (left beam) [5.3.(3)]
 $V_{c1,Ed} = 83.29$ [kN] Shear force (lower column) [5.3.(3)]
 $V_{c2,Ed} = 0.00$ [kN] Shear force (upper column) [6.2.5]
 $z = 708$ [mm] Lever arm
 $V_{wp,Ed} = (M_{b1,Ed} - M_{b2,Ed}) / z - (V_{c1,Ed} - V_{c2,Ed}) / 2$ [5.3.(3)]
 $V_{wp,Ed} = 378.29$ [kN] Shear force acting on the web panel
 $A_{vs} = 57.33$ [cm²] Shear area of the column web EN1993-1-1:[6.2.6.(3)]
 $A_{vc} = 57.33$ [cm²] Shear area EN1993-1-1:[6.2.6.(3)]
 $V_{wp,Rd} = 0.9 (f_{y,wc} A_{vc} + f_{y,wp} A_{vp} + f_{ys} A_{vd}) / \phi$ [6.2.6.1]
 $V_{wp,Rd} = 700.03$ [kN] Resistance of the column web panel for shear
 $V_{wp,Ed} / V_{wp,Rd} \leq 1.0$ 0.54 < 1.00 verified (0.54)

WEB - TRANSVERSE COMPRESSION - LEVEL OF THE BEAM BOTTOM FLANGE

Bearing:
 $t_{wc} = 11$ [mm] Effective thickness of the column web
 $b_{eff,c,wc} = 325$ [mm] Effective width of the web for compression
 $A_{vc} = 57.33$ [cm²] Shear area EN1993-1-1:[6.2.6.(3)]
 $w = 0.81$ Reduction factor for interaction with shear [6.2.6.2.(1)]
 $\sigma_{com,Ed} = 105.69$ [MPa] Maximum compressive stress in web [6.2.6.2.(2)]
 $k_{wc} = 1.00$ Reduction factor conditioned by compressive stresses [6.2.6.2.(2)]
 $F_{c,wc,Rd1} = w k_{wc} b_{eff,c,wc} t_{wc} f_{yc} / \gamma_{M0}$
 $F_{c,wc,Rd1} = 685.02$ [kN] Column web resistance [6.2.6.2.(1)]

Buckling: [6.2.6.2.(1)]
 $d_{wc} = 298$ [mm] Height of compressed web [6.2.6.2.(1)]
 $I_p = 0.88$ Plate slenderness of an element [6.2.6.2.(1)]
 $r = 0.88$ Reduction factor for element buckling
 $F_{cwb,Rd2} = w \cdot k_{wc} \cdot r \cdot b_{eff,c,wc} \cdot t_{wc} \cdot f_{yc} / \gamma_{M1}$ [6.2.6.2.(1)]
 $F_{cwc,Rd2} = 600.37$ [kN] Column web resistance

Final resistance: [6.2.6.2.(1)]
 $F_{cwc,Rd,low} = \min(F_{cwc,Rd1}, F_{cwc,Rd2})$
 $F_{cwc,Rd} = 600.37$ [kN] Column web resistance

WEB - TRANSVERSE COMPRESSION - LEVEL OF THE BEAM TOP FLANGE

Bearing: [6.2.6.2.(6)]
 $t_{wc} = 11$ [mm] Effective thickness of the column web [6.2.6.2.(1)]
 $b_{eff,c,wc} = 320$ [mm] Effective width of the web for compression EN1993-1-1:[6.2.6.(3)]
 $A_{vc} = 57.33$ [cm²] Shear area [6.2.6.2.(1)]
 $w = 0.82$ Reduction factor for interaction with shear [6.2.6.2.(2)]
 $\sigma_{com,Ed} = 105.69$ [MPa] Maximum compressive stress in web [6.2.6.2.(2)]
 $k_{wc} = 1.00$ Reduction factor conditioned by compressive stresses [6.2.6.2.(1)]
 $F_{cwc,Rd1} = w \cdot k_{wc} \cdot b_{eff,c,wc} \cdot t_{wc} \cdot f_{yc} / \gamma_{M0}$
 $F_{cwc,Rd1} = 677.73$ [kN] Column web resistance [6.2.6.2.(1)]

Buckling: [6.2.6.2.(1)]
 $d_{wc} = 298$ [mm] Height of compressed web [6.2.6.2.(1)]
 $I_p = 0.88$ Plate slenderness of an element [6.2.6.2.(1)]
 $r = 0.88$ Reduction factor for element buckling [6.2.6.2.(1)]
 $F_{cwb,Rd2} = w \cdot k_{wc} \cdot r \cdot b_{eff,c,wc} \cdot t_{wc} \cdot f_{yc} / \gamma_{M1}$
 $F_{cwc,Rd2} = 597.35$ [kN] Column web resistance [6.2.6.2.(1)]
Final resistance: [6.2.6.2.(1)]
 $F_{cwc,Rd,upp} = \min(F_{cwc,Rd1}, F_{cwc,Rd2})$
 $F_{cwc,Rd,upp} = 597.35$ [kN] Column web resistance

GEOMETRICAL PARAMETERS OF A CONNECTION

EFFECTIVE LENGTHS AND PARAMETERS - COLUMN FLANGE

Nr	m	m_x	e	e_x	p	$l_{eff,cp}$	$l_{eff,nc}$	$l_{eff,1}$	$l_{eff,2}$	$l_{eff,cp,g}$	$l_{eff,nc,g}$	$l_{eff,1,g}$	$l_{eff,2,g}$
1	8	-	115	-	70	50	158	50	158	95	105	70	105
2	8	-	115	-	70	50	175	50	175	140	70	70	70
3	8	-	115	-	135	50	175	50	175	200	200	200	200
4	8	-	115	-	200	50	175	50	175	400	100	100	100
5	8	-	115	-	150	50	175	50	175	200	125	125	138
6	8	-	115	-	100	50	175	50	175	125	138	125	138

EFFECTIVE LENGTHS AND PARAMETERS - FRONT PLATE

Nr	m	m_x	e	e_x	p	$l_{eff,cp}$	$l_{eff,nc}$	$l_{eff,1}$	$l_{eff,2}$	$l_{eff,cp,g}$	$l_{eff,nc,g}$	$l_{eff,1,g}$	$l_{eff,2,g}$
1	21	-	65	-	70	131	167	131	167	135	119	70	119
2	21	-	65	-	70	131	165	131	165	140	70	135	135
3	21	-	65	-	135	131	165	131	165	270	135	200	200
4	21	-	65	-	200	131	165	131	165	400	150	150	150
5	21	-	65	-	150	131	165	131	165	300	132	132	132
6	21	-	65	-	100	131	165	131	165	165	132	132	132

- m - Bolt distance from the web
- m_x - Bolt distance from the beam flange
- e - Bolt distance from the outer edge
- e_x - Bolt distance from the horizontal outer edge
- p - Distance between bolts
- $l_{eff,cp}$ - Effective length for a single bolt in the circular failure mode
- $l_{eff,nc}$ - Effective length for a single bolt in the non-circular failure mode
- $l_{eff,1}$ - Effective length for a single bolt for mode 1
- $l_{eff,2}$ - Effective length for a single bolt for mode 2
- $l_{eff,cp,g}$ - Effective length for a group of bolts in the circular failure mode
- $l_{eff,nc,g}$ - Effective length for a group of bolts in the non-circular failure mode
- $l_{eff,1,g}$ - Effective length for a group of bolts for mode 1
- $l_{eff,2,g}$ - Effective length for a group of bolts for mode 2

CONNECTION RESISTANCE FOR COMPRESSION

$$N_{j,Rd} = \min (N_{cb,Rd} / 2, F_{c,wb,Rd,low} / 2, F_{c,wc,Rd,low} / 2, F_{c,wc,Rd,upp} / 2)$$

$$N_{j,Rd} = 1146.75 \text{ [kN]} \quad \text{Connection resistance for compression}$$

[6.2]

$$N_{b1,Ed} / N_{j,Rd} \leq 1.0$$

$$0.07 < 1.00$$

verified

(0.07)

CONNECTION RESISTANCE FOR BENDING

$$F_{t,Rd} = 57.82 \text{ [kN]} \quad \text{Bolt resistance for tension}$$

$$B_{p,Rd} = 247.55 \text{ [kN]} \quad \text{Punching shear resistance of a bolt}$$

[Table 3.4]

[Table 3.4]

$F_{t,Rd}$ - column flange resistance due to bending
 $F_{t,wc,Rd}$ - column web resistance due to tension
 $F_{t,ep,Rd}$ - resistance of the front plate due to bending
 $F_{t,wb,Rd}$ - resistance of the web in tension

[6.2.6.4] , [Tab.6.2]

[6.2.6.3.(1)]

[6.2.6.5] , [Tab.6.2]

[6.2.6.8.(1)]

$$F_{t,Rd} = \min (F_{t,1,Rd}, F_{t,2,Rd}, F_{t,3,Rd})$$

$$F_{t,wc,Rd} = w_{eff,t,wc} t_{wc} f_{yc} / \gamma_{M0}$$

$$F_{t,ep,Rd} = \min (F_{t,1,ep,Rd}, F_{t,2,ep,Rd}, F_{t,3,ep,Rd})$$

$$F_{t,wb,Rd} = b_{eff,t,wb} t_{wb} f_{yb} / \gamma_{M0}$$

RESISTANCE OF THE BOLT ROW NO. 1

$F_{t1,Rd,comp}$ - Formula

$$F_{t,fc,Rd(1)} = 135.65$$

$$F_{t,wc,Rd(1)} = 127.56$$

$$F_{t,ep,Rd(1)} = 135.65$$

$$F_{t,wb,Rd(1)} = 314.00$$

$$B_{p,Rd} = 495.09$$

$$V_{wp,Rd/b} = 700.03$$

$$F_{c,wc,Rd} = 600.37$$

$$F_{c,fb,Rd} = 1150.71$$

$$F_{c,wb,Rd} = 573.37$$

$$F_{t1,Rd} = \min (F_{t1,Rd,comp})$$

$F_{t1,Rd,comp}$

$$135.65$$

$$127.56$$

$$135.65$$

$$314.00$$

$$495.09$$

$$700.03$$

$$600.37$$

$$1150.71$$

$$573.37$$

$$127.56$$

Component

Column flange - tension
 Column web - tension
 Front plate - tension
 Beam web - tension
 Bolts due to shear punching
 Web panel - shear
 Column web - compression
 Beam flange - compression
 Beam web - compression
 Bolt row resistance

RESISTANCE OF THE BOLT ROW NO. 2

$F_{t2,Rd,comp}$ - Formula

$$F_{t,fc,Rd(2)} = 135.65$$

$$F_{t,wc,Rd(2)} = 127.56$$

$$F_{t,ep,Rd(2)} = 135.65$$

$$F_{t,wb,Rd(2)} = 314.00$$

$$B_{p,Rd} = 495.09$$

$$V_{wp,Rd/b} = 700.03$$

$$F_{c,wc,Rd} = 600.37$$

$$F_{c,fb,Rd} = 1150.71$$

$$F_{c,wb,Rd} = 573.37$$

$$F_{t2,Rd} = \min (F_{t2,Rd,comp})$$

$F_{t2,Rd,comp}$

$$135.65$$

$$127.56$$

$$135.65$$

$$314.00$$

$$495.09$$

$$572.47$$

$$472.81$$

$$1023.15$$

$$445.81$$

$$143.73$$

$$294.91$$

$$143.73$$

$$326.60$$

$$127.56$$

Component

Column flange - tension
 Column web - tension
 Front plate - tension
 Beam web - tension
 Bolts due to shear punching
 Web panel - shear
 Column web - compression
 Beam flange - compression
 Beam web - compression
 Column flange - tension - group
 Column web - tension - group
 Front plate - tension - group
 Beam web - tension - group
 Bolt row resistance

RESISTANCE OF THE BOLT ROW NO. 3

$F_{t3,Rd,comp}$ - Formula

$$F_{t,fc,Rd(3)} = 135.65$$

$$F_{t,wc,Rd(3)} = 127.56$$

$$F_{t,ep,Rd(3)} = 135.65$$

$$F_{t,wb,Rd(3)} = 314.00$$

$$F_{t3,Rd} = \min (F_{t3,Rd,comp})$$

$F_{t3,Rd,comp}$

$$135.65$$

$$127.56$$

$$135.65$$

$$314.00$$

$$127.56$$

Component

Column flange - tension
 Column web - tension
 Front plate - tension
 Beam web - tension
 Bolt row resistance

$F_{t5,Rd,comp}$ - Formula	$F_{t5,Rd,comp}$	Component
$F_{t,wc,Rd}(5+4+3+2) - \sum 4^2 F_{tj,Rd} = 819.44 - 382.69$	436.75	Column web - tension - group
$F_{t,fc,Rd}(5+4+3+2+1) - \sum 4^1 F_{tj,Rd} = 678.24 - 510.25$	167.99	Column flange - tension - group
$F_{t,wc,Rd}(5+4+3+2+1) - \sum 4^1 F_{tj,Rd} = 905.32 - 510.25$	395.07	Column web - tension - group
$F_{t,ep,Rd}(5+4) - \sum 4^4 F_{tj,Rd} = 271.30 - 127.56$	143.73	Front plate - tension - group
$F_{t,wb,Rd}(5+4) - \sum 4^4 F_{tj,Rd} = 838.95 - 127.56$	711.39	Beam web - tension - group
$F_{t,ep,Rd}(5+4+3) - \sum 4^3 F_{tj,Rd} = 406.94 - 255.12$	151.82	Front plate - tension - group
$F_{t,wb,Rd}(5+4+3) - \sum 4^3 F_{tj,Rd} = 1162.55 - 255.12$	907.42	Beam web - tension - group
$F_{t,ep,Rd}(5+4+3+2) - \sum 4^2 F_{tj,Rd} = 542.59 - 382.69$	159.91	Front plate - tension - group
$F_{t,wb,Rd}(5+4+3+2) - \sum 4^2 F_{tj,Rd} = 1330.34 - 382.69$	947.65	Beam web - tension - group
$F_{t,ep,Rd}(5+4+3+2+1) - \sum 4^1 F_{tj,Rd} = 678.24 - 510.25$	167.99	Front plate - tension - group
$F_{t,wb,Rd}(5+4+3+2+1) - \sum 4^1 F_{tj,Rd} = 1616.70 - 510.25$	1106.45	Beam web - tension - group
$F_{t5,Rd} = \text{Min}(F_{t5,Rd,comp})$	63.13	Bolt row resistance

RESISTANCE OF THE BOLT ROW NO. 6

$F_{t6,Rd,comp}$ - Formula	$F_{t6,Rd,comp}$	Component
$F_{t,fc,Rd}(6) = 135.65$	135.65	Column flange - tension
$F_{t,wc,Rd}(6) = 127.56$	127.56	Column web - tension
$F_{t,ep,Rd}(6) = 135.65$	135.65	Front plate - tension
$F_{t,wb,Rd}(6) = 314.00$	314.00	Beam web - tension
$B_{p,Rd} = 495.09$	495.09	Bolts due to shear punching
$V_{wp,Rd}/b - \sum 1^5 F_{tj,Rd} = 700.03 - 573.37$	126.66	Web panel - shear
$F_{c,wc,Rd} - \sum 1^5 F_{tj,Rd} = 600.37 - 573.37$	27.00	Column web - compression
$F_{c,fb,Rd} - \sum 1^5 F_{tj,Rd} = 1150.71 - 573.37$	577.34	Beam flange - compression
$F_{c,wb,Rd} - \sum 1^5 F_{tj,Rd} = 573.37 - 573.37$	0.00	Beam web - compression
$F_{t,fc,Rd}(6+5) - \sum 5^5 F_{tj,Rd} = 271.30 - 63.13$	208.17	Column flange - tension - group
$F_{t,wc,Rd}(6+5) - \sum 5^5 F_{tj,Rd} = 545.10 - 63.13$	481.98	Column web - tension - group
$F_{t,fc,Rd}(6+5+4) - \sum 5^4 F_{tj,Rd} = 406.94 - 190.69$	216.26	Column flange - tension - group
$F_{t,wc,Rd}(6+5+4) - \sum 5^4 F_{tj,Rd} = 817.18 - 190.69$	626.49	Column web - tension - group
$F_{t,fc,Rd}(6+5+4+3) - \sum 5^3 F_{tj,Rd} = 542.59 - 318.25$	224.34	Column flange - tension - group
$F_{t,wc,Rd}(6+5+4+3) - \sum 5^3 F_{tj,Rd} = 878.10 - 318.25$	559.85	Column web - tension - group
$F_{t,fc,Rd}(6+5+4+3+2) - \sum 5^2 F_{tj,Rd} = 678.24 - 445.81$	232.43	Column flange - tension - group
$F_{t,wc,Rd}(6+5+4+3+2) - \sum 5^2 F_{tj,Rd} = 926.60 - 445.81$	480.78	Column web - tension - group
$F_{t,fc,Rd}(6+5+4+3+2+1) - \sum 5^1 F_{tj,Rd} = 813.89 - 573.37$	240.51	Column flange - tension - group
$F_{t,wc,Rd}(6+5+4+3+2+1) - \sum 5^1 F_{tj,Rd} = 981.82 - 573.37$	408.45	Column web - tension - group
$F_{t,ep,Rd}(6+5) - \sum 5^5 F_{tj,Rd} = 271.30 - 63.13$	208.17	Front plate - tension - group
$F_{t,wb,Rd}(6+5) - \sum 5^5 F_{tj,Rd} = 676.73 - 63.13$	613.60	Beam web - tension - group
$F_{t,ep,Rd}(6+5+4) - \sum 5^4 F_{tj,Rd} = 406.94 - 190.69$	216.26	Front plate - tension - group
$F_{t,wb,Rd}(6+5+4) - \sum 5^4 F_{tj,Rd} = 1156.13 - 190.69$	965.44	Beam web - tension - group
$F_{t,ep,Rd}(6+5+4+3) - \sum 5^3 F_{tj,Rd} = 542.59 - 318.25$	224.34	Front plate - tension - group
$F_{t,wb,Rd}(6+5+4+3) - \sum 5^3 F_{tj,Rd} = 1479.72 - 318.25$	1161.47	Beam web - tension - group
$F_{t,ep,Rd}(6+5+4+3+2) - \sum 5^2 F_{tj,Rd} = 678.24 - 445.81$	232.43	Front plate - tension - group
$F_{t,wb,Rd}(6+5+4+3+2) - \sum 5^2 F_{tj,Rd} = 1647.51 - 445.81$	1201.70	Beam web - tension - group
$F_{t,ep,Rd}(6+5+4+3+2+1) - \sum 5^1 F_{tj,Rd} = 813.89 - 573.37$	240.51	Front plate - tension - group
$F_{t,wb,Rd}(6+5+4+3+2+1) - \sum 5^1 F_{tj,Rd} = 1933.88 - 573.37$	1360.51	Beam web - tension - group
$F_{t6,Rd} = \text{Min}(F_{t6,Rd,comp})$	0.00	Bolt row resistance

SUMMARY TABLE OF FORCES

Nr	h_j	$F_{tj,Rd}$	$F_{t,fc,Rd}$	$F_{t,wc,Rd}$	$F_{t,ep,Rd}$	$F_{t,wb,Rd}$	$F_{t,Rd}$	$B_{p,Rd}$
1	743	127.56	135.65	127.56	135.65	314.00	135.65	495.09
2	673	127.56	135.65	127.56	135.65	314.00	135.65	495.09
3	603	127.56	135.65	127.56	135.65	314.00	135.65	495.09
4	403	127.56	135.65	127.56	135.65	314.00	135.65	495.09
5	203	63.13	135.65	127.56	135.65	314.00	135.65	495.09
6	103	-	135.65	127.56	135.65	314.00	135.65	495.09

CONNECTION RESISTANCE FOR BENDING $M_{j,Rd}$

$$M_{j,Rd} = \sum h_j F_{tj,Rd}$$

$$M_{j,Rd} = 321.81 \text{ [kN}\cdot\text{m]} \quad \text{Connection resistance for bending}$$

$$M_{b1,Ed} / M_{j,Rd} \leq 1,0$$

$$0,92 < 1,00$$

verified

(0,92)

CONNECTION RESISTANCE FOR SHEAR

$a_v =$	0,60	Coefficient for calculation of $F_{v,Rd}$	[Table 3.4]
$b_{Lf} =$	0,88	Reduction factor for long connections	[3.8]
$F_{v,Rd} =$	50,67 [kN]	Shear resistance of a single bolt	[Table 3.4]
$F_{t,Rd,max} =$	67,82 [kN]	Tensile resistance of a single bolt	[Table 3.4]
$F_{b,Rd,int} =$	218,88 [kN]	Bearing resistance of an intermediate bolt	[Table 3.4]
$F_{b,Rd,ext} =$	218,88 [kN]	Bearing resistance of an outermost bolt	[Table 3.4]

Nr	$F_{tj,Rd,N}$	$F_{tj,Ed,N}$	$F_{tj,Rd,M}$	$F_{tj,Ed,M}$	$F_{tj,Ed}$	$F_{vj,Rd}$
1	135,65	-13,88	127,56	117,86	103,98	45,85
2	135,65	-13,88	127,56	117,86	103,98	45,85
3	135,65	-13,88	127,56	117,86	103,98	45,85
4	135,65	-13,88	127,56	117,86	44,44	77,62
5	135,65	-13,88	63,13	58,32	-13,88	101,34
6	135,65	-13,88	0,00	0,00		

- $F_{tj,Rd,N}$ – Bolt row resistance for simple tension
- $F_{tj,Ed,N}$ – Force due to axial force in a bolt row
- $F_{tj,Rd,M}$ – Bolt row resistance for simple bending
- $F_{tj,Ed,M}$ – Force due to moment in a bolt row
- $F_{tj,Ed}$ – Maximum tensile force in a bolt row
- $F_{vj,Rd}$ – Reduced bolt row resistance

$$F_{tj,Ed,N} = N_{j,Ed} F_{tj,Rd,N} / N_{j,Rd}$$

$$F_{tj,Ed,M} = M_{j,Ed} F_{tj,Rd,M} / M_{j,Rd}$$

$$F_{tj,Ed} = F_{tj,Ed,N} + F_{tj,Ed,M}$$

$$F_{vj,Rd} = \min(n_h F_{v,Rd} (1 - F_{tj,Ed} / (1.4 n_h F_{t,Rd,max}), n_h F_{v,Rd} \cdot n_h F_{b,Rd}))$$

[Table 3.4]
[Table 3.4]

$$V_{j,Rd} = n_h \sum_{i=1}^n F_{vj,Rd}$$

$$V_{j,Rd} = 362,36 \text{ [kN]}$$

$$V_{b1,Ed} / V_{j,Rd} \leq 1,0$$

$$0,32 < 1,00$$

verified

(0,32)

WELD RESISTANCE

$A_w =$	213,53 [cm ²]	Area of all welds	[4.5.3.2(2)]
$A_{wz} =$	100,27 [cm ²]	Area of horizontal welds	[4.5.3.2(2)]
$A_{wy} =$	113,26 [cm ²]	Area of vertical welds	[4.5.3.2(5)]
$I_{w,max} =$	151126,35 [cm ⁴]	Moment of inertia of the weld arrangement with respect to the hor. axis	[4.5.3.2(5)]
$\sigma_{\perp} =$	-61,80 [MPa]	Normal stress in a weld	[4.5.3.2(5)]
$\tau_{\parallel} =$	-61,80 [MPa]	Stress in a vertical weld	[4.5.3.2(7)]
$\tau_{\perp} =$	10,37 [MPa]	Tangent stress	(0,34)
$\rho =$	0,80	Correlation coefficient	(0,35)

verified
verified
verified

(0,34)
(0,35)
(0,24)

$$\sigma_{\perp} \leq 0,9 \cdot f_{w,EM2}$$

$$123,60 < 360,00$$

$$124,90 < 360,00$$

$$61,80 < 259,20$$

[6.2.6.3.(2)]
[6.2.6.3.(2)]
[6.2.6.3.(2)]
[6.2.6.3.(2)]
[6.3.2.(1)]

CONNECTION STIFFNESS

$t_{wash} =$	4 [mm]	Washer thickness
$h_{head} =$	12 [mm]	Bolt head height
$h_{nut} =$	16 [mm]	Bolt nut height
$L_b =$	61 [mm]	Bolt length
$K_{10} =$	4 [mm]	Stiffness coefficient of bolts

STIFFNESSES OF BOLT ROWS

Nr	h_j	k_3	k_4	k_5	$k_{eff,j}$	$k_{eff,j} h_j$	$k_{eff,j} h_j^2$
1	743	1	621	95	1	6,32	469,55
2	673	1	621	56	1	5,69	382,82
3	603	1	621	104	1	5,13	309,53
4	493	1	621	104	1	3,43	138,27
5	293	1	621	104	1	1,73	35,10
6	103	1	621	104	1	0,88	9,04
					Sum	23,17	1344,31

$k_{eff,j} = 1 / (\sum 3^5 (1 / k_{i,j}))$			[6.3.3.1.(2)]
$z_{eq} = \sum_j k_{eff,j} h_j^2 / \sum_j k_{eff,j} h_j$			[6.3.3.1.(3)]
$z_{eq} =$	560	[mm]	Equivalent force arm
$k_{eq} = \sum_j k_{eff,j} h_j / z_{eq}$			[6.3.3.1.(1)]
$k_{eq} =$	4	[mm]	Equivalent stiffness coefficient of a bolt arrangement
$A_{vc} =$	57.33	[cm ²]	Shear area
$b =$	1.00		Transformation parameter
$z =$	560	[mm]	Lever arm
$k_1 =$	4	[mm]	Stiffness coefficient of the column web panel subjected to shear
$b_{eff,c,wc} =$	298	[mm]	Effective width of the web for compression
$t_{wc} =$	11	[mm]	Effective thickness of the column web
$d_c =$	352	[mm]	Height of compressed web
$k_2 =$	7	[mm]	Stiffness coefficient of the compressed column web
$S_{j,ini} = E z_{eq}^2 / \sum_i (1 / k_1 + 1 / k_2 + 1 / k_{eq})$			[6.3.1.(4)]
$S_{j,ini} =$	105488.24	[kN*m]	Initial rotational stiffness
$m =$	2.41		Stiffness coefficient of a connection
$S_j = S_{j,ini} / m$			[6.3.1.(4)]
$S_j =$	43701.75	[kN*m]	Final rotational stiffness
			[5.2.2.5]
			[5.2.2.5]
Connection classification due to stiffness.			
$S_{j,rig} =$	89305.97	[kN*m]	Stiffness of a rigid connection
$S_{j,pin} =$	5581.62	[kN*m]	Stiffness of a pinned connection
$S_{j,ini} \geq S_{j,rig}$	RIGID		

WEAKEST COMPONENT:

BEAM WEB OR BRACKET FLANGE - COMPRESSION

REMARKS

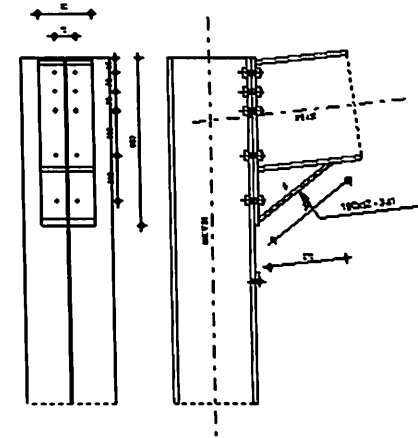
Distance of bolts from an edge is too large. 124 [mm] > 120 [mm]

Ratio 0.92

Connection conforms to the code

OK
Ratio
0.80





GENERAL

Connection no.: 5
Connection name: KS3
Structure node: 6
Structure bars: 4, 7

GEOMETRY

COLUMN

Section: HEA 300
Bar no.: 4
a = -90.0 [Deg] Inclination angle
h_c = 290 [mm] Height of column section
b_{fc} = 300 [mm] Width of column section
t_{wc} = 9 [mm] Thickness of the web of column section
t_{fc} = 14 [mm] Thickness of the flange of column section
r_c = 27 [mm] Radius of column section fillet
A_c = 112.53 [cm²] Cross-sectional area of a column
I_{xc} = 18263.50 [cm⁴] Moment of inertia of the column section
Material: S235
f_{yc} = 235.00 [MPa] Resistance

BEAM

Section: IPE 400
Bar no.: 7
a = 6.3 [Deg] Inclination angle
h_b = 400 [mm] Height of beam section
b_f = 180 [mm] Width of beam section
t_{wb} = 9 [mm] Thickness of the web of beam section
t_{fb} = 14 [mm] Thickness of the flange of beam section
r_b = 21 [mm] Radius of beam section fillet
r_b = 21 [mm] Radius of beam section fillet
A_b = 84.46 [cm²] Cross-sectional area of a beam
I_{xb} = 23128.40 [cm⁴] Moment of inertia of the beam section
Material: S235
f_{yb} = 235.00 [MPa] Resistance

BOLTS

The shear plane passes through the UNTHREADED portion of the bolt.
d = 16 [mm] Bolt diameter
Class = 8.8 Bolt class
F_{IRd} = 90.43 [kN] Tensile resistance of a bolt
n_h = 2 Number of bolt columns
n_v = 5 Number of bolt rows
h₁ = 56 [mm] Distance between first bolt and upper edge of front plate
Horizontal spacing e₁ = 70 [mm]
Vertical spacing p₁ = 70;70;160;160 [mm]

PLATE

h_p = 602 [mm] Plate height
b_p = 180 [mm] Plate width
t_p = 16 [mm] Plate thickness

Material: S 235
 $f_{yp} = 235.00$ [MPa] Resistance

LOWER STIFFENER

$W_d = 180$ [mm] Plate width
 $t_{fd} = 12$ [mm] Flange thickness
 $h_d = 180$ [mm] Plate height
 $t_{wd} = 8$ [mm] Web thickness
 $a = 270$ [mm] Plate length
 $\alpha = 38.0$ [Deg] Inclination angle

Material: S 235
 $f_{yu} = 235.00$ [MPa] Resistance

FILLET WELDS

$a_w = 7$ [mm] Web weld
 $a_f = 10$ [mm] Flange weld
 $a_{fd} = 5$ [mm] Horizontal weld

MATERIAL FACTORS

$\gamma_{M0} = 1.00$ Partial safety factor
 $\gamma_{M1} = 1.00$ Partial safety factor
 $\gamma_{M2} = 1.00$ Partial safety factor
 $\gamma_{M3} = 1.25$ Partial safety factor

LOADS

Ultimate limit state

Case: 692: tk9 $(1+6) \cdot 1.35 + (5+10) \cdot 1.50 + (8+13) \cdot 0.90$

$M_{b1,Ed} = 136.00$ [kN*m] Bending moment in the right beam
 $V_{b1,Ed} = 82.11$ [kN] Shear force in the right beam
 $N_{b1,Ed} = -68.53$ [kN] Axial force in the right beam
 $M_{c1,Ed} = 136.00$ [kN*m] Bending moment in the lower column
 $V_{c1,Ed} = 68.53$ [kN] Shear force in the lower column
 $N_{c1,Ed} = -82.11$ [kN] Axial force in the lower column

RESULTS

BEAM RESISTANCES

COMPRESSION

$A_b = 84.46$ [cm²] Area
 $N_{cb,Rd} = A_b f_{yb} / \gamma_{M0}$
 $N_{cb,Rd} = 1984.89$ [kN] Design compressive resistance of the section

SHEAR

$A_{vb} = 57.09$ [cm²] Shear area
 $V_{cb,Rd} = A_{vb} (f_{yb} / \sqrt{3})$
 $V_{cb,Rd} = 774.64$ [kN] Design sectional resistance for shear
 $V_{b1,Ed} / V_{cb,Rd} \leq 1.0$ verified
 $0.11 < 1.00$

BENDING - PLASTIC MOMENT (WITHOUT BRACKETS)

$W_{plb} = 1307.26$ [cm³] Plastic section modulus
 $M_{pb,Rd} = W_{plb} f_{yb} / \gamma_{M0}$
 $M_{pb,Rd} = 307.21$ [kN*m] Plastic resistance of the section for bending (without stiffeners)

BENDING ON THE CONTACT SURFACE WITH PLATE OR CONNECTED ELEMENT

$W_{pl} = 2129.82$ [cm³] Plastic section modulus
 $M_{cb,Rd} = W_{pl} f_{yb} / \gamma_{M0}$
 $M_{cb,Rd} = 500.51$ [kN*m] Design resistance of the section for bending

FLANGE AND WEB - COMPRESSION

$M_{cb,Rd} = 500.51$ [kN*m] Design resistance of the section for bending
 $h_f = 568$ [mm] Distance between the centroids of flanges
 $F_{c,fb,Rd} = M_{cb,Rd} / h_f$
 $F_{c,fb,Rd} = 881.09$ [kN] Resistance of the compressed flange and web

WEB OR BRACKET FLANGE - COMPRESSION - LEVEL OF THE BEAM BOTTOM FLANGE

EN1993-1-1:[6.2.4]

EN1993-1-1:[6.2.4]

EN1993-1-1:[6.2.6.(3)]

EN1993-1-1:[6.2.6.(2)]
(0.11)

EN1993-1-1:[6.2.5.(2)]

EN1993-1-1:[6.2.5.(2)]

EN1993-1-1:[6.2.5]

EN1993-1-1:[6.2.5]

EN1993-1-1:[6.2.5]
[6.2.6.7.(1)]

[6.2.6.7.(1)]

Bearing:

b =	6.3	[Deg]	Angle between the front plate and the beam	[6.2.6.2.(1)]
g =	38.0	[Deg]	Inclination angle of the bracket plate	EN1993-1-1:[6.2.6.(3)]
b _{eff,c,wb} =	22.4	[mm]	Effective width of the web for compression	[6.2.6.2.(1)]
A _{wb} =	42.69	[cm ²]	Shear area	[6.2.6.2.(2)]
w =	0.89		Reduction factor for interaction with shear	[6.2.6.2.(2)]
s _{com,Ed} =	105.43	[MPa]	Maximum compressive stress in web	
k _{wc} =	1.00		Reduction factor conditioned by compressive stresses	[6.2.6.2.(1)]
F _{c,wb,Rd1} =	$[w \cdot k_{wc} \cdot b_{eff,c,wb} \cdot t_{wb} \cdot f_{yb} / \sigma_{M0}] \cos(g) / \sin(g - b)$			
F _{c,wb,Rd1} =	603.05	[kN]	Beam web resistance	[6.2.6.2.(1)]

Buckling:

d _{wb} =	331	[mm]	Height of compressed web	[6.2.6.2.(1)]
I _p =	0.99		Plate slenderness of an element	[6.2.6.2.(1)]
r =	0.81		Reduction factor for element buckling	[6.2.6.2.(1)]
F _{c,wb,Rd2} =	$[w \cdot k_{wc} \cdot r \cdot b_{eff,c,wb} \cdot t_{wb} \cdot f_{yb} / \sigma_{M1}] \cos(g) / \sin(g - b)$			
F _{c,wb,Rd2} =	487.43	[kN]	Beam web resistance	[6.2.6.7.(1)]

Resistance of the bracket flange

F _{c,wb,Rd3} =	$b_b \cdot t_b \cdot f_{yb} / (0.8 \cdot \sigma_{M0})$			
F _{c,wb,Rd3} =	634.50	[kN]	Resistance of the bracket flange	[6.2.6.2.(1)]

Final resistance:

F _{c,wb,Rd,low} =	Min (F _{c,wb,Rd1} , F _{c,wb,Rd2} , F _{c,wb,Rd3})			
F _{c,wb,Rd,low} =	487.43	[kN]	Beam web resistance	

COLUMN RESISTANCES

WEB PANEL - SHEAR

M _{b1,Ed} =	136.00	[kN*m]	Bending moment (right beam)	[5.3.(3)]
M _{b2,Ed} =	0.00	[kN*m]	Bending moment (left beam)	[5.3.(3)]
V _{c1,Ed} =	68.53	[kN]	Shear force (lower column)	[5.3.(3)]
V _{c2,Ed} =	0.00	[kN]	Shear force (upper column)	[6.2.5]
z =	494	[mm]	Lever arm	[5.3.(3)]
V _{wp,Ed} =	$(M_{b1,Ed} - M_{b2,Ed}) / z - (V_{c1,Ed} - V_{c2,Ed}) / 2$			
V _{wp,Ed} =	240.84	[kN]	Shear force acting on the web panel	EN1993-1-1:[6.2.6.(3)]

A _{vc} =	37.28	[cm ²]	Shear area of the column web	[6.2.6.1]
V _{wp,Rd} =	37.28	[cm ²]	Shear area	
V _{wp,Rd} =	$0.9 \cdot (f_{y,wc} \cdot A_{vc} + f_{y,wp} \cdot A_{vp} + f_{ys} \cdot A_{vd}) / \phi$			
V _{wp,Rd} =	455.20	[kN]	Resistance of the column web panel for shear	(0.53)

0.53 < 1.00

WEB - TRANSVERSE COMPRESSION - LEVEL OF THE BEAM BOTTOM FLANGE

Bearing:

t _{wc} =	9	[mm]	Effective thickness of the column web	[6.2.6.2.(6)]
b _{eff,c,wc} =	281	[mm]	Effective width of the web for compression	[6.2.6.2.(1)]
A _{vc} =	37.28	[cm ²]	Shear area	EN1993-1-1:[6.2.6.(3)]
w =	0.81		Reduction factor for interaction with shear	[6.2.6.2.(2)]
s _{com,Ed} =	84.74	[MPa]	Maximum compressive stress in web	[6.2.6.2.(2)]
k _{wc} =	1.00		Reduction factor conditioned by compressive stresses	[6.2.6.2.(1)]
F _{c,wc,Rd1} =	$[w \cdot k_{wc} \cdot b_{eff,c,wc} \cdot t_{wc} \cdot f_{yc} / \sigma_{M0}]$			
F _{c,wc,Rd1} =	452.73	[kN]	Column web resistance	[6.2.6.2.(1)]

Buckling:

d _{wc} =	208	[mm]	Height of compressed web	[6.2.6.2.(1)]
I _p =	0.89		Plate slenderness of an element	[6.2.6.2.(1)]
r =	0.87		Reduction factor for element buckling	[6.2.6.2.(1)]
F _{c,wc,Rd2} =	$[w \cdot k_{wc} \cdot r \cdot b_{eff,c,wc} \cdot t_{wc} \cdot f_{yc} / \sigma_{M1}]$			
F _{c,wc,Rd2} =	395.63	[kN]	Column web resistance	

Final resistance:

F _{c,wc,Rd,low} =	Min (F _{c,wc,Rd1} , F _{c,wc,Rd2})			
----------------------------	--	--	--	--

$F_{c,wc,Rd} = 395.63$ [kN] Column web resistance

[6.2.6.2.(1)]

WEB - TRANSVERSE COMPRESSION - LEVEL OF THE BEAM TOP FLANGE

Bearing:
 $t_{wc} = 9$ [mm] Effective thickness of the column web
 $b_{eff,c,wc} = 279$ [mm] Effective width of the web for compression
 $A_{wc} = 37.28$ [cm²] Shear area
 $w = 0.81$ Reduction factor for interaction with shear
 $f_{com,Ed} = 84.74$ [MPa] Maximum compressive stress in web
 $k_{wc} = 1.00$ Reduction factor conditioned by compressive stresses
 $F_{c,wc,Rd1} = w k_{wc} b_{eff,c,wc} t_{wc} f_{yc} / \gamma_{M0}$
 $F_{c,wc,Rd1} = 450.98$ [kN] Column web resistance

[6.2.6.2.(6)]
[6.2.6.2.(1)]
EN1993-1-1:6.2.6.(3)]
[6.2.6.2.(1)]
[6.2.6.2.(2)]
[6.2.6.2.(2)]

Buckling:
 $\phi_{wc} =$
 $l_p = 208$ [mm] Height of compressed web
 $r = 0.88$ Plate slenderness of an element
 $\alpha = 0.88$ Reduction factor for element buckling
 $F_{c,wb,Rd2} = w k_{wc} r b_{eff,c,wc} t_{wc} f_{yc} / \gamma_{M1}$
 $F_{c,wb,Rd2} = 394.93$ [kN] Column web resistance

[6.2.6.2.(1)]
[6.2.6.2.(1)]
[6.2.6.2.(1)]
[6.2.6.2.(1)]

Final resistance:
 $F_{c,wc,Rd,upp} = \min(F_{c,wc,Rd1}, F_{c,wb,Rd2})$
 $F_{c,wc,Rd,upp} = 394.93$ [kN] Column web resistance

[6.2.6.2.(1)]

GEOMETRICAL PARAMETERS OF A CONNECTION

EFFECTIVE LENGTHS AND PARAMETERS - COLUMN FLANGE

Nr	m	m_x	e	e_x	p	$l_{eff,cp}$	$l_{eff,nc}$	$l_{eff,1}$	$l_{eff,2}$	$l_{eff,cp,g}$	$l_{eff,nc,g}$	$l_{eff,1,g}$	$l_{eff,2,g}$
1	9	-	115	-	70	57	146	57	146	99	91	91	91
2	9	-	115	-	70	57	180	57	180	140	70	70	70
3	9	-	115	-	115	57	180	57	180	140	160	160	160
4	9	-	115	-	160	57	180	57	180	167	167	167	167
5	9	-	115	-	160	57	177	57	177	189	167	167	167

EFFECTIVE LENGTHS AND PARAMETERS - FRONT PLATE

Nr	m	m_x	e	e_x	p	$l_{eff,cp}$	$l_{eff,nc}$	$l_{eff,1}$	$l_{eff,2}$	$l_{eff,cp,g}$	$l_{eff,nc,g}$	$l_{eff,1,g}$	$l_{eff,2,g}$
1	23	-	55	-	70	143	182	143	182	142	137	137	137
2	23	-	55	-	70	143	160	143	160	140	70	70	70
3	23	-	55	-	115	143	160	143	160	230	115	115	115
4	23	-	55	-	160	143	160	143	160	232	160	160	160
5	23	-	55	-	160	143	160	143	160	232	160	160	160

m - Bolt distance from the web
 m_x - Bolt distance from the beam flange
 e - Bolt distance from the outer edge
 e_x - Bolt distance from the horizontal outer edge
 p - Distance between bolts
 $l_{eff,cp}$ - Effective length for a single bolt in the circular failure mode
 $l_{eff,nc}$ - Effective length for a single bolt in the non-circular failure mode
 $l_{eff,1}$ - Effective length for a single bolt for mode 1
 $l_{eff,2}$ - Effective length for a single bolt for mode 2
 $l_{eff,cp,g}$ - Effective length for a group of bolts in the circular failure mode
 $l_{eff,nc,g}$ - Effective length for a group of bolts in the non-circular failure mode
 $l_{eff,1,g}$ - Effective length for a group of bolts for mode 1
 $l_{eff,2,g}$ - Effective length for a group of bolts for mode 2

CONNECTION RESISTANCE FOR COMPRESSION

[6.2]

$N_{j,Rd} = \min(N_{cb,Rd2}, F_{c,wb,Rd,low}, 2 F_{c,wc,Rd,low}, 2 F_{c,wc,Rd,upp})$
 $N_{j,Rd} = 789.87$ [kN] Connection resistance for compression

(0.09)

$N_{b1,Ed} / N_{j,Rd} \leq 1.0$

0.09 < 1.00

verified

CONNECTION RESISTANCE FOR BENDING

$F_{t,Rd} = 90.43$ [kN] Bolt resistance for tension
 $B_{p,Rd} = 182.40$ [kN] Punching shear resistance of a bolt

[Table 3.4]
 [Table 3.4]

$F_{t,fc,Rd}$ - column flange resistance due to bending
 $F_{t,wc,Rd}$ - column web resistance due to tension
 $F_{t,ep,Rd}$ - resistance of the front plate due to bending
 $F_{t,wb,Rd}$ - resistance of the web in tension

[6.2.6.4] , [Tab.6.2]
 [6.2.6.3.(1)]
 [6.2.6.5] , [Tab.6.2]
 [6.2.6.8.(1)]

$F_{t,Rd} = \min(F_{t,1,fc,Rd} \cdot F_{t,2,fc,Rd} \cdot F_{t,3,fc,Rd})$
 $F_{t,wc,Rd} = w \cdot b_{eff,1,wc} \cdot t_{wc} \cdot f_{yc} / g_{M0}$
 $F_{t,ep,Rd} = \min(F_{t,1,ep,Rd} \cdot F_{t,2,ep,Rd} \cdot F_{t,3,ep,Rd})$
 $F_{t,wb,Rd} = b_{eff,1,wb} \cdot t_{wb} \cdot f_{yb} / g_{M0}$

RESISTANCE OF THE BOLT ROW NO. 1

$F_{t1,Rd,comp}$ - Formula
 $F_{t,fc,Rd(1)} = 180.86$
 $F_{t,wc,Rd(1)} = 113.58$
 $F_{t,ep,Rd(1)} = 180.86$
 $F_{t,wb,Rd(1)} = 289.27$
 $B_{p,Rd} = 364.81$
 $V_{wp,Rd}/b = 455.20$
 $F_{c,wc,Rd} = 395.63$
 $F_{c,fb,Rd} = 881.09$
 $F_{c,wb,Rd} = 487.43$
 $F_{t1,Rd} = \min(F_{t1,Rd,comp})$

$F_{t1,Rd,comp}$	Component
180.86	Column flange - tension
113.58	Column web - tension
180.86	Front plate - tension
289.27	Beam web - tension
364.81	Bolts due to shear punching
455.20	Web panel - shear
395.63	Column web - compression
881.09	Beam flange - compression
487.43	Beam web - compression
113.58	Bolt row resistance

RESISTANCE OF THE BOLT ROW NO. 2

$F_{t2,Rd,comp}$ - Formula
 $F_{t,fc,Rd(2)} = 180.86$
 $F_{t,wc,Rd(2)} = 113.58$
 $F_{t,ep,Rd(2)} = 180.86$
 $F_{t,wb,Rd(2)} = 289.27$
 $B_{p,Rd} = 364.81$
 $V_{wp,Rd}/b - \sum_1^1 F_{tj,Rd} = 455.20 - 113.58$
 $F_{c,wc,Rd} - \sum_1^1 F_{tj,Rd} = 395.63 - 113.58$
 $F_{c,fb,Rd} - \sum_1^1 F_{tj,Rd} = 881.09 - 113.58$
 $F_{c,wb,Rd} - \sum_1^1 F_{tj,Rd} = 487.43 - 113.58$
 $F_{t,fc,Rd(2+1)} - \sum_1^1 F_{tj,Rd} = 361.73 - 113.58$
 $F_{t,wc,Rd(2+1)} - \sum_1^1 F_{tj,Rd} = 295.87 - 113.58$
 $F_{t,ep,Rd(2+1)} - \sum_1^1 F_{tj,Rd} = 322.62 - 113.58$
 $F_{t,wb,Rd(2+1)} - \sum_1^1 F_{tj,Rd} = 418.97 - 113.58$
 $F_{t2,Rd} = \min(F_{t2,Rd,comp})$

$F_{t2,Rd,comp}$	Component
180.86	Column flange - tension
113.58	Column web - tension
180.86	Front plate - tension
289.27	Beam web - tension
364.81	Bolts due to shear punching
341.62	Web panel - shear
282.06	Column web - compression
767.51	Beam flange - compression
373.86	Beam web - compression
248.15	Column flange - tension - group
182.30	Column web - tension - group
209.04	Front plate - tension - group
305.39	Beam web - tension - group
113.58	Bolt row resistance

RESISTANCE OF THE BOLT ROW NO. 3

$F_{t3,Rd,comp}$ - Formula
 $F_{t,fc,Rd(3)} = 180.86$
 $F_{t,wc,Rd(3)} = 113.58$
 $F_{t,ep,Rd(3)} = 180.86$
 $F_{t,wb,Rd(3)} = 289.27$
 $B_{p,Rd} = 364.81$
 $V_{wp,Rd}/b - \sum_1^2 F_{tj,Rd} = 455.20 - 227.15$
 $F_{c,wc,Rd} - \sum_1^2 F_{tj,Rd} = 395.63 - 227.15$
 $F_{c,fb,Rd} - \sum_1^2 F_{tj,Rd} = 881.09 - 227.15$
 $F_{c,wb,Rd} - \sum_1^2 F_{tj,Rd} = 487.43 - 227.15$
 $F_{t,fc,Rd(3+2)} - \sum_2^2 F_{tj,Rd} = 357.57 - 113.58$
 $F_{t,wc,Rd(3+2)} - \sum_2^2 F_{tj,Rd} = 262.79 - 113.58$
 $F_{t,ep,Rd(3+2+1)} - \sum_2^1 F_{tj,Rd} = 542.59 - 227.15$
 $F_{t,wb,Rd(3+2+1)} - \sum_2^1 F_{tj,Rd} = 394.94 - 227.15$
 $F_{t3,Rd} = \min(F_{t3,Rd,comp})$

$F_{t3,Rd,comp}$	Component
180.86	Column flange - tension
113.58	Column web - tension
180.86	Front plate - tension
289.27	Beam web - tension
364.81	Bolts due to shear punching
228.05	Web panel - shear
168.48	Column web - compression
653.93	Beam flange - compression
260.28	Beam web - compression
243.99	Column flange - tension - group
149.21	Column web - tension - group
315.44	Column flange - tension - group
167.79	Column web - tension - group
195.95	Front plate - tension - group
113.58	Bolt row resistance

F_{13,Rd,comp} - Formula

$$F_{lwb,Rd(3+2)} - \sum 2^2 F_{ij,Rd} = 373.88 - 113.58$$

$$F_{lep,Rd(3+2+1)} - \sum 2^1 F_{ij,Rd} = 490.59 - 227.15$$

$$F_{lwb,Rd(3+2+1)} - \sum 2^1 F_{ij,Rd} = 651.38 - 227.15$$

$$F_{13,Rd} = \text{Min} (F_{13,Rd,comp})$$

RESISTANCE OF THE BOLT ROW NO. 4

F_{14,Rd,comp} - Formula

$$F_{lfc,Rd(4)} = 180.86$$

$$F_{lwc,Rd(4)} = 113.58$$

$$F_{lep,Rd(4)} = 180.86$$

$$F_{lwb,Rd(4)} = 289.27$$

$$B_{p,Rd} = 364.81$$

$$V_{wp,Rd/b} - \sum 1^3 F_{ti,Rd} = 455.20 - 340.73$$

$$F_{cwc,Rd} - \sum 1^3 F_{ij,Rd} = 395.63 - 340.73$$

$$F_{cfb,Rd} - \sum 1^3 F_{ij,Rd} = 881.09 - 340.73$$

$$F_{cwb,Rd} - \sum 1^3 F_{ij,Rd} = 487.43 - 340.73$$

$$F_{lfc,Rd(4+3)} - \sum 3^3 F_{ij,Rd} = 361.73 - 113.58$$

$$F_{lwc,Rd(4+3)} - \sum 3^3 F_{ij,Rd} = 394.31 - 113.58$$

$$F_{lfc,Rd(4+3+2)} - \sum 3^2 F_{ij,Rd} = 542.59 - 227.15$$

$$F_{lwc,Rd(4+3+2)} - \sum 3^2 F_{ij,Rd} = 472.52 - 227.15$$

$$F_{lfc,Rd(4+3+2+1)} - \sum 3^1 F_{ij,Rd} = 723.46 - 340.73$$

$$F_{lwc,Rd(4+3+2+1)} - \sum 3^1 F_{ij,Rd} = 547.38 - 340.73$$

$$F_{lep,Rd(4+3)} - \sum 3^3 F_{ij,Rd} = 361.73 - 113.58$$

$$F_{lwb,Rd(4+3)} - \sum 3^3 F_{ij,Rd} = 555.77 - 113.58$$

$$F_{lep,Rd(4+3+2)} - \sum 3^2 F_{ij,Rd} = 503.91 - 227.15$$

$$F_{lwb,Rd(4+3+2)} - \sum 3^2 F_{ij,Rd} = 697.25 - 227.15$$

$$F_{lep,Rd(4+3+2+1)} - \sum 3^1 F_{ij,Rd} = 684.97 - 340.73$$

$$F_{lwb,Rd(4+3+2+1)} - \sum 3^1 F_{ij,Rd} = 974.74 - 340.73$$

$$F_{14,Rd} = \text{Min} (F_{14,Rd,comp})$$

RESISTANCE OF THE BOLT ROW NO. 5

F_{15,Rd,comp} - Formula

$$F_{lfc,Rd(5)} = 180.86$$

$$F_{lwc,Rd(5)} = 113.58$$

$$F_{lep,Rd(5)} = 180.86$$

$$F_{lwb,Rd(5)} = 289.27$$

$$B_{p,Rd} = 364.81$$

$$V_{wp,Rd/b} - \sum 1^4 F_{ti,Rd} = 455.20 - 395.63$$

$$F_{cwc,Rd} - \sum 1^4 F_{ij,Rd} = 395.63 - 395.63$$

$$F_{cfb,Rd} - \sum 1^4 F_{ij,Rd} = 881.09 - 395.63$$

$$F_{cwb,Rd} - \sum 1^4 F_{ij,Rd} = 487.43 - 395.63$$

$$F_{lfc,Rd(5+4)} - \sum 4^4 F_{ij,Rd} = 361.73 - 54.90$$

$$F_{lwc,Rd(5+4)} - \sum 4^4 F_{ij,Rd} = 497.62 - 54.90$$

$$F_{lfc,Rd(5+4+3)} - \sum 4^3 F_{ij,Rd} = 542.59 - 168.48$$

$$F_{lwc,Rd(5+4+3)} - \sum 4^3 F_{ij,Rd} = 551.79 - 168.48$$

$$F_{lfc,Rd(5+4+3+2)} - \sum 4^2 F_{ij,Rd} = 723.46 - 282.06$$

$$F_{lwc,Rd(5+4+3+2)} - \sum 4^2 F_{ij,Rd} = 593.04 - 282.06$$

$$F_{lfc,Rd(5+4+3+2+1)} - \sum 4^1 F_{ij,Rd} = 904.32 - 395.63$$

$$F_{lwc,Rd(5+4+3+2+1)} - \sum 4^1 F_{ij,Rd} = 632.40 - 395.63$$

$$F_{lep,Rd(5+4)} - \sum 4^4 F_{ij,Rd} = 361.73 - 54.90$$

$$F_{lwb,Rd(5+4)} - \sum 4^4 F_{ij,Rd} = 646.59 - 54.90$$

$$F_{lep,Rd(5+4+3)} - \sum 4^3 F_{ij,Rd} = 542.59 - 168.48$$

$$F_{lwb,Rd(5+4+3)} - \sum 4^3 F_{ij,Rd} = 879.01 - 168.48$$

$$F_{lep,Rd(5+4+3+2)} - \sum 4^2 F_{ij,Rd} = 698.25 - 282.06$$

$$F_{lwb,Rd(5+4+3+2)} - \sum 4^2 F_{ij,Rd} = 1020.48 - 282.06$$

$$F_{lep,Rd(5+4+3+2+1)} - \sum 4^1 F_{ij,Rd} = 879.31 - 395.63$$

$$F_{15,Rd} = \text{Min} (F_{15,Rd,comp})$$

F _{13,Rd,comp}	Component
260.31	Beam web - tension - group
263.44	Front plate - tension - group
424.23	Beam web - tension - group
113.58	Bolt row resistance

F _{14,Rd,comp}	Component
180.86	Column flange - tension
113.58	Column web - tension
180.86	Front plate - tension
289.27	Beam web - tension
364.81	Bolts due to shear punching
114.47	Web panel - shear
54.90	Column web - compression
540.36	Beam flange - compression
146.70	Beam web - compression
248.15	Column flange - tension - group
280.73	Column web - tension - group
315.44	Column flange - tension - group
245.37	Column web - tension - group
382.73	Column flange - tension - group
206.65	Column web - tension - group
248.15	Front plate - tension - group
442.20	Beam web - tension - group
276.75	Front plate - tension - group
470.09	Beam web - tension - group
344.24	Front plate - tension - group
634.01	Beam web - tension - group
54.90	Bolt row resistance

F _{15,Rd,comp}	Component
180.86	Column flange - tension
113.58	Column web - tension
180.86	Front plate - tension
289.27	Beam web - tension
364.81	Bolts due to shear punching
59.57	Web panel - shear
0.00	Column web - compression
485.45	Beam flange - compression
91.80	Beam web - compression
306.82	Column flange - tension - group
442.72	Column web - tension - group
374.11	Column flange - tension - group
383.30	Column web - tension - group
441.40	Column flange - tension - group
310.99	Column web - tension - group
508.69	Column flange - tension - group
236.77	Column web - tension - group
306.82	Front plate - tension - group
591.69	Beam web - tension - group
374.11	Front plate - tension - group
710.52	Beam web - tension - group
416.19	Front plate - tension - group
738.42	Beam web - tension - group
483.67	Front plate - tension - group
0.00	Bolt row resistance

$F_{t5,Rd,comp}$ - Formula	$F_{t5,Rd,comp}$	Component
$F_{t5,Rd,comp} = F_{t5,Rd} \cdot (5 + 4 + 3 + 2 + 1) = \sum_{i=1}^5 F_{t5,Rd,i} = 1297.97 - 395.63$	902.34	Beam web - tension - group
$F_{t5,Rd} = \min(F_{t5,Rd,comp})$	0.00	Bolt row resistance

SUMMARY TABLE OF FORCES

Nr	h_j	$F_{tj,Rd}$	$F_{t,fc,Rd}$	$F_{t,wc,Rd}$	$F_{t,ep,Rd}$	$F_{t,wb,Rd}$	$F_{t,Rd}$	$B_{p,Rd}$
1 529						289.27	180.86	364.81
2 459	113.58	180.86	113.58	180.86	289.27	180.86	180.86	364.81
3 389	113.58	180.86	113.58	180.86	289.27	180.86	180.86	364.81
4 229	113.58	180.86	113.58	180.86	289.27	180.86	180.86	364.81
5 69	54.90	180.86	113.58	180.86	289.27	180.86	180.86	364.81
	-	180.86	113.58	180.86	289.27	180.86	180.86	364.81

CONNECTION RESISTANCE FOR BENDING $M_{j,Rd}$		[6.2]
$M_{j,Rd} = \sum h_j F_{tj,Rd}$		
$M_{j,Rd} = 169.11$ [kN*m]	Connection resistance for bending	verified (0.80)
$M_{b1,Ed} / M_{j,Rd} \leq 1.0$	0.80 < 1.00	

CONNECTION RESISTANCE FOR SHEAR		[Table 3.4]
$a_v =$		[3.8]
$b_{t,r} = 0.60$	Coefficient for calculation of $F_{v,Rd}$	[Table 3.4]
$F_{v,Rd} = 0.93$	Reduction factor for long connections	[Table 3.4]
$F_{t,Rd,max} = 71.90$ [kN]	Shear resistance of a single bolt	[Table 3.4]
$F_{b,Rd,int} = 90.43$ [kN]	Tensile resistance of a single bolt	[Table 3.4]
$F_{b,Rd,ext} = 161.28$ [kN]	Bearing resistance of an intermediate bolt	[Table 3.4]
$F_{b,Rd,ext} = 161.28$ [kN]	Bearing resistance of an outermost bolt	

Nr	$F_{tj,Rd,N}$	$F_{tj,Ed,N}$	$F_{tj,Rd,M}$	$F_{tj,Ed,M}$	$F_{tj,Ed}$	$F_{vj,Rd}$
1 180.86			91.34	77.63	99.71	
2 180.86	-13.71	113.58	91.34	77.63	99.71	
3 180.86	-13.71	113.58	91.34	77.63	126.51	
4 180.86	-13.71	113.58	44.15	30.45	143.80	
5 180.86	-13.71	54.90	0.00	-13.71		
	-13.71	0.00				

$F_{tj,Rd,N}$ - Bolt row resistance for simple tension		
$F_{tj,Ed,N}$ - Force due to axial force in a bolt row		
$F_{tj,Rd,M}$ - Bolt row resistance for simple bending		
$F_{tj,Ed,M}$ - Force due to moment in a bolt row		
$F_{tj,Ed}$ - Maximum tensile force in a bolt row		
$F_{vj,Rd}$ - Reduced bolt row resistance		
$F_{tj,Ed,N} = N_{j,Ed} F_{tj,Rd,N} / N_{j,Rd}$		[Table 3.4]
$F_{tj,Ed,M} = M_{j,Ed} F_{tj,Rd,M} / M_{j,Rd}$		[Table 3.4]
$F_{tj,Ed} = F_{tj,Ed,N} + F_{tj,Ed,M}$		
$F_{vj,Rd} = \min(n_h F_{v,Ed} (1 - F_{tj,Ed} / (1.4 n_h F_{t,Rd,max}), n_h F_{v,Rd}, n_h F_{b,Rd}))$		
$V_{j,Rd} = n_h \sum_{i=1}^n F_{vj,Rd}$		(0.14)
$V_{j,Rd} = 569.44$ [kN]	Connection resistance for shear	verified
$V_{b1,Ed} / V_{j,Rd} \leq 1.0$	0.14 < 1.00	

WELD RESISTANCE		[4.5.3.2(2)]
$A_w = 155.37$ [cm ²]	Area of all welds	[4.5.3.2(2)]
$A_{wv} = 85.68$ [cm ²]	Area of horizontal welds	[4.5.3.2(2)]
$A_{wz} = 69.69$ [cm ²]	Area of vertical welds	[4.5.3.2(5)]
$I_{w,max} = 65955.76$ [cm ⁴]	Moment of inertia of the weld arrangement with respect to the hor. axis	[4.5.3.2(5)]
$\sigma_{t,Ed} = -46.09$ [MPa]	Normal stress in a weld	[4.5.3.2(5)]
$\sigma_{v,Ed} = -42.97$ [MPa]	Stress in a vertical weld	[4.5.3.2(7)]
$\tau_{v,Ed} = 11.78$ [MPa]	Tangent stress	(0.26)
$\rho = 0.80$	Correlation coefficient	(0.25)
$\rho \leq 0.9 \cdot f_{t,u} / E_{w2}$		(0.18)
	verified	
	verified	
	verified	
	92.19 < 360.00	
	88.33 < 360.00	
	46.09 < 259.20	

CONNECTION STIFFNESS

$t_{wash} =$	4	[mm]	Washer thickness	[6.2.6.3.(2)]
$h_{head} =$	12	[mm]	Bolt head height	[6.2.6.3.(2)]
$h_{nut} =$	16	[mm]	Bolt nut height	[6.2.6.3.(2)]
$L_b =$	52	[mm]	Bolt length	[6.2.6.3.(2)]
$k_{10} =$	5	[mm]	Stiffness coefficient of bolts	[6.3.2.(1)]

STIFFNESSES OF BOLT ROWS

Nr	h_j	k_3	k_4	k_5	$k_{eff,j}$	$k_{eff,j} h_j$	$k_{eff,j} h_j^2$
1	529	1	185	43	1	5.28	279.74
2	459	1	185	22	1	4.49	206.03
3	389	1	185	36	1	3.87	150.66
4	229	1	185	45	1	2.29	52.56
5	69	1	185	45	1	0.69	4.81
					Sum	16.62	693.79

$$k_{eff,j} = 1 / (\sum 3^5 (1 / k_{i,j}))$$

[6.3.3.1.(2)]

$$z_{eq} = \sum_j k_{eff,j} h_j^2 / \sum_j k_{eff,j} h_j$$

$z_{eq} =$

417

[mm]

Equivalent force arm

[6.3.3.1.(3)]

$$k_{eq} = \sum_j k_{eff,j} h_j / z_{eq}$$

$k_{eq} =$

4

[mm]

Equivalent stiffness coefficient of a bolt arrangement

[6.3.3.1.(1)]

$A_{vc} =$	37.28	[cm ²]	Shear area	EN1993-1-1:[6.2.6.(3)]
$b =$	1.00		Transformation parameter	[5.3.(7)]
$z =$	417	[mm]	Lever arm	[6.2.5]
$k_1 =$	3	[mm]	Stiffness coefficient of the column web panel subjected to shear	[6.3.2.(1)]
$b_{eff,c,wc} =$	260	[mm]	Effective width of the web for compression	[6.2.6.2.(1)]
$t_{wc} =$	9	[mm]	Effective thickness of the column web	[6.2.6.2.(6)]
$d_c =$	262	[mm]	Height of compressed web	[6.2.6.2.(1)]
$k_2 =$	6	[mm]	Stiffness coefficient of the compressed column web	[6.3.2.(1)]

$$S_{j,ini} = E z_{eq}^2 / \sum_i (1 / k_1 + 1 / k_2 + 1 / k_{eq})$$

$S_{j,ini} =$

51171.85

[kN*m]

Initial rotational stiffness

[6.3.1.(4)]

$$m =$$

1.66

Stiffness coefficient of a connection

[6.3.1.(6)]

$$S_j = S_{j,ini} / m$$

$S_j =$

30838.90

[kN*m]

Final rotational stiffness

[6.3.1.(4)]

Connection classification due to stiffness.				[5.2.2.5]
$S_{j,rig} =$	64363.04	[kN*m]	Stiffness of a rigid connection	[5.2.2.5]
$S_{j,pin} =$	4022.69	[kN*m]	Stiffness of a pinned connection	

$$S_{j,pin} \leq S_{j,ini} < S_{j,rig}$$

SEMI-RIGID

WEAKEST COMPONENT:

COLUMN WEB - COMPRESSION

REMARKS	
The thickness of bracket web is less than the thickness of beam web	8 [mm] < 9 [mm]
The thickness of bracket flange is less than the thickness of beam flange	12 [mm] < 14 [mm]

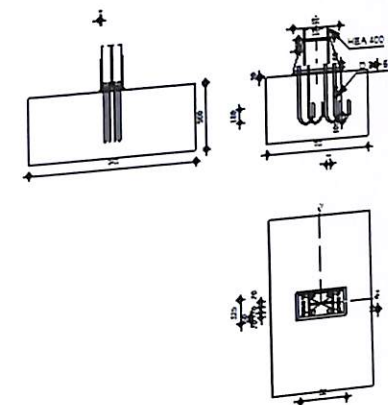
Connection conforms to the code

Ratio 0.80



Fixed column base design

Eurocode 3: EN 1993-1-8:2005/AC:2009



GENERAL

Connection no.: 6
Connection name: OT2
Structure node: 3
Structure bars: 3

GEOMETRY

COLUMN

Section: HEA 400
Bar no.: 3
 $L_c =$

$a =$	7.40	[m]	Column length
$h_c =$	0.0	[Deg]	Inclination angle
$b_{fc} =$	390	[mm]	Height of column section
$t_{wc} =$	300	[mm]	Width of column section
$t_c =$	11	[mm]	Thickness of the web of column section
$r_c =$	19	[mm]	Thickness of the flange of column section
$A_c =$	27	[mm]	Radius of column section fillet
$I_{yc} =$	158.98	[cm ²]	Cross-sectional area of a column
Material: S235	45069.40	[cm ⁴]	Moment of inertia of the column section
$f_{yc} =$	235.00	[MPa]	Resistance
$f_{uc} =$	360.00	[MPa]	Yield strength of a material

COLUMN BASE

$l_{pd} =$			
$b_{pd} =$	690	[mm]	Length
$t_{pd} =$	325	[mm]	Width
Material: S275	30	[mm]	Thickness
$f_{ypd} =$	275.00	[MPa]	Resistance
$f_{upd} =$	430.00	[MPa]	Yield strength of a material

ANCHORAGE

The shear plane passes through the UNTHREADED portion of the bolt.
Class =

$f_{yb} =$	6.8		Anchor class
$f_{ub} =$	480.00	[MPa]	Yield strength of the anchor material
$d =$	600.00	[MPa]	Tensile strength of the anchor material
$A_s =$	30	[mm]	Bolt diameter
$A_v =$	5.61	[cm ²]	Effective section area of a bolt
$n_H =$	7.07	[cm ²]	Area of bolt section
$n_V =$	4		Number of bolt columns
	4		Number of bolt rows

Horizontal spacing $e_{H1} =$ 250; 140 [mm]
Vertical spacing $e_{V1} =$ 70; 70 [mm]

Anchor dimensions

$L_1 =$ 60 [mm]

L₂ = 640 [mm]
L₃ = 210 [mm]
L₄ = 180 [mm]

Washer
l_{wd} = 50 [mm] Length
b_{wd} = 50 [mm] Width
t_{wd} = 5 [mm] Thickness

STIFFENER

l_s = 690 [mm] Length
h_s = 390 [mm] Height
t_s = 16 [mm] Thickness
d₁ = 20 [mm] Cut
d₂ = 20 [mm] Cut

MATERIAL FACTORS

γ_{M0} = 1.00 Partial safety factor
γ_{M2} = 1.25 Partial safety factor
γ_{EC} = 1.50 Partial safety factor

SPREAD FOOTING

L = 1500 [mm] Spread footing length
B = 2400 [mm] Spread footing width
H = 900 [mm] Spread footing height

Concrete

Class C20/25
f_{ck} = 20.00 [MPa] Characteristic resistance for compression

Grout layer

t_g = 30 [mm] Thickness of leveling layer (grout)
f_{ck,g} = 12.00 [MPa] Characteristic resistance for compression
C_{f,d} = 0.30 Coeff. of friction between the base plate and concrete

WELDS

a_p = 7 [mm] Footing plate of the column base
a_s = 5 [mm] Stiffeners

LOADS

Case: 692; t_{k9} (1+6) * 1.35 + (5+10) * 1.50 + (8+13) * 0.90

N_{1,Ed} =
V_{1,Ed,z} = -289.45 [kN] Axial force
M_{1,Ed,y} = -112.34 [kN] Shear force
M_{1,Ed,y} = 309.43 [kN*m] Bending moment

RESULTS

COMPRESSION ZONE

COMPRESSION OF CONCRETE

f_{cd} = 13.33 [MPa] Design compressive resistance
f_j = 22.70 [MPa] Design bearing resistance under the base plate
c = l_p 60 [mm] Additional width of the bearing pressure zone
c = 140 [mm] Effective width of the bearing pressure zone under the flange
b_{eff} = 325 [mm] Effective length of the bearing pressure zone under the flange
A_{c0} = 453.61 [cm²] Area of the joint between the base plate and the foundation
A_{c1} = 4082.45 [cm²] Maximum design area of load distribution
F_{rd,u} = A_{c0} * f_{cd} 1814.42 [kN] Bearing resistance of concrete
F_{rd,u} = b_j * F_{rd,u} / (b_{eff} * l_{eff}) 0.67 Reduction factor for compression
f_{rd} = 26.67 [MPa] Design bearing resistance

EN 1992-1:3.1.6.(1)
[6.2.5.(7)]

[6.2.5.(4)]

[6.2.5.(3)]

[6.2.5.(3)]

EN 1992-1:6.7.(3)

EN 1992-1:6.7.(3)

EN 1992-1:6.7.(3)

[6.2.5.(7)]

[6.2.5.(7)]

$A_{c,n} =$	1809.79	[cm ²]	Bearing area for compression	[6.2.8.2.(1)]
$A_{c,y} =$	584.20	[cm ²]	Bearing area for bending My	[6.2.8.3.(1)]
$F_{c,Rd,i} = A_{c,i} \cdot f_{jd}$				[6.2.8.2.(1)]
$F_{c,Rd,n} =$	4826.12	[kN]	Bearing resistance of concrete for compression	[6.2.8.3.(1)]
$F_{c,Rd,y} =$	1557.88	[kN]	Bearing resistance of concrete for bending My	

COLUMN FLANGE AND WEB IN COMPRESSION				EN 1993-1-1:[5.5.2]
$CL =$	1.00		Section class	EN1993-1-1:[6.2.5.(2)]
$W_{ply} =$	6370.77	[cm ³]	Plastic section modulus	EN1993-1-1:[6.2.5]
$M_{c,Rd,y} =$	1497.13	[kN*m]	Design resistance of the section for bending	[6.2.6.7.(1)]
$h_{fy} =$	422	[mm]	Distance between the centroids of flanges	
$F_{c,fc,Rd,y} = M_{c,Rd,y} / h_{fy}$				[6.2.6.7.(1)]
$F_{c,fc,Rd,y} =$	3545.55	[kN]	Resistance of the compressed flange and web	

RESISTANCES OF SPREAD FOOTING IN THE COMPRESSION ZONE				[6.2.8.2.(1)]
$N_{j,Rd} = F_{c,Rd,n}$				
$N_{j,Rd} =$	4826.12	[kN]	Resistance of a spread footing for axial compression	[6.2.8.3]
$F_{c,Rd,y} = \min(F_{c,Rd,y}, F_{c,fc,Rd,y})$				
$F_{c,Rd,y} =$	1557.88	[kN]	Resistance of spread footing in the compression zone	

TENSION ZONE				[Table 3.4]
STEEL FAILURE				[Table 3.4]
$A_b =$			Effective anchor area	[3.6.1.(3)]
$f_{ub} =$	5.61	[cm ²]	Tensile strength of the anchor material	
$\beta =$	0.85	[MPa]	Reduction factor of anchor resistance	[Table 3.4]
$F_{t,Rd,s1} = \beta \cdot f_{ub} \cdot A_b / \gamma_{M2}$				
$F_{t,Rd,s1} =$	206.00	[kN]	Anchor resistance to steel failure	
$F_{t,Rd,s} = F_{t,Rd,s1}$				
$F_{t,Rd,s} =$	206.00	[kN]	Anchor resistance to steel failure	EN 1992-1:[3.1.2]

PULL-OUT FAILURE				
$f_{ck} =$	20.00	[MPa]	Characteristic compressive strength of concrete	EN 1992-1:[8.4.2.(2)]
$f_{cd} = 0.7 \cdot 0.3 \cdot f_{ck}^{2/3} / \gamma_c$				EN 1992-1:[8.4.2.(2)]
$f_{cd} =$	1.03	[MPa]	Design tensile resistance	EN 1992-1:[8.4.2.(2)]
$h_1 =$	1.00		Coeff. related to the quality of the bond conditions and concreting conditions	
$h_2 =$	1.00		Coeff. related to the bar diameter	EN 1992-1:[8.4.2.(2)]
$f_{bd} = 2.25 \cdot h_1 \cdot h_2 \cdot f_{cd}$				EN 1992-1:[8.4.2.(2)]
$f_{bd} =$	2.32	[MPa]	Design value of the ultimate bond stress	
$h_{ef} =$	640	[mm]	Effective anchorage depth	
$F_{t,Rd,p} = p \cdot d \cdot h_{ef} \cdot f_{bd}$				
$F_{t,Rd,p} =$	140.00	[kN]	Design uplift capacity	

TENSILE RESISTANCE OF AN ANCHOR				
$F_{t,Rd} = \min(F_{t,Rd,s}, F_{t,Rd,p})$				
$F_{t,Rd} =$	140.00	[kN]	Tensile resistance of an anchor	[6.2.6.5]

BENDING OF THE BASE PLATE				
Bending moment $M_{j,Ed,y}$				
$l_{eff,1} =$	472	[mm]	Effective length for a single bolt for mode 1	[6.2.6.5]
$l_{eff,2} =$	472	[mm]	Effective length for a single bolt for mode 2	[6.2.6.5]
$m =$	107	[mm]	Distance of a bolt from the stiffening edge	[6.2.4]
$M_{pl,1,Rd} =$	29.21	[kN*m]	Plastic resistance of a plate for mode 1	[6.2.4]
$M_{pl,2,Rd} =$	29.21	[kN*m]	Plastic resistance of a plate for mode 2	[6.2.4]
$F_{T,1,Rd} =$	1091.12	[kN]	Resistance of a plate for mode 1	
$F_{T,2,Rd} =$	553.40	[kN]	Resistance of a plate for mode 2	[6.2.4]
$F_{T,3,Rd} =$	559.98	[kN]	Resistance of a plate for mode 3	
$F_{t,pl,Rd,y} = \min(F_{T,1,Rd}, F_{T,2,Rd}, F_{T,3,Rd})$				
$F_{t,pl,Rd,y} =$	553.40	[kN]	Tension resistance of a plate	[6.2.8.3]

RESISTANCES OF SPREAD FOOTING IN THE TENSION ZONE				
$F_{t,Rd,y} = F_{t,pl,Rd,y}$				
$F_{t,Rd,y} =$	553.40	[kN]	Resistance of a column base in the tension zone	

CONNECTION CAPACITY CHECK

$N_{j,Ed} / N_{j,Rd} \leq 1.0$ (6.24)	0.06 < 1.00	verified	(0.06)
$e_y = 1069$ [mm]	Axial force eccentricity		[6.2.8.3]
$z_{cy} = 211$ [mm]	Lever arm $F_{C,Rd,y}$		[6.2.8.1.(2)]
$z_{ly} = 265$ [mm]	Lever arm $F_{T,Rd,y}$		[6.2.8.1.(3)]
$M_{j,Rd,y} = 328.33$ [kN*m]	Connection resistance for bending	verified	(0.94)
$M_{j,Ed,y} / M_{j,Rd,y} \leq 1.0$ (6.23)	0.94 < 1.00		

SHEAR

BEARING PRESSURE OF AN ANCHOR BOLT ONTO THE BASE PLATE				[Table 3.4]
Shear force $V_{j,Ed,z}$				[Table 3.4]
$a_{d,z} = 1.21$	Coeff. taking account of the bolt position - in the direction of shear			[Table 3.4]
$a_{b,z} = 1.00$	Coeff. for resistance calculation $F_{1,vb,Rd}$			
$k_{1,z} = 1.36$	Coeff. taking account of the bolt position - perpendicularly to the direction of shear			[6.2.2.(7)]
$F_{1,vb,Rd,z} = k_{1,z} \cdot a_{b,z} \cdot f_{ub} \cdot d \cdot t_p / \gamma_{M2}$	Resistance of an anchor bolt for bearing pressure onto the base plate			[6.2.2.(7)]
$F_{1,vb,Rd,z} = 421.83$ [kN]				[6.2.2.(7)]
SHEAR OF AN ANCHOR BOLT				[6.2.2.(7)]
$a_b = 0.30$	Coeff. for resistance calculation $F_{2,vb,Rd}$			[6.2.2.(7)]
$A_{vb} = 7.07$ [cm ²]	Area of bolt section			[6.2.2.(7)]
$f_{ub} = 600.00$ [MPa]	Tensile strength of the anchor material			[6.2.2.(7)]
$\gamma_{M2} = 1.25$	Partial safety factor			[6.2.2.(7)]
$F_{2,vb,Rd} = a_b \cdot f_{ub} \cdot A_{vb} / \gamma_{M2}$	Shear resistance of a bolt - without lever arm			[6.2.2.(6)]
$F_{2,vb,Rd} = 100.43$ [kN]				[6.2.2.(6)]
SPLITTING RESISTANCE				[6.2.2.(6)]
$C_{f,d} = 0.30$	Coeff. of friction between the base plate and concrete			[6.2.2.(6)]
$N_{c,Ed} = 289.45$ [kN]	Compressive force			
$F_{f,Rd} = C_{f,d} \cdot N_{c,Ed}$	Slip resistance			
$F_{f,Rd} = 86.84$ [kN]				

SHEAR CHECK

$V_{j,Rd,z} = n_b \cdot \min(F_{1,vb,Rd,z}, F_{2,vb,Rd}) + F_{f,Rd}$	Connection resistance for shear	verified	(0.09)
$V_{j,Rd,z} = 1292.00$ [kN]			
$V_{j,Ed,z} / V_{j,Rd,z} \leq 1.0$	0.09 < 1.00		

STIFFENER CHECK

Trapezoid plate parallel to the column web				
$M_1 = 21.33$ [kN*m]	Bending moment acting on a stiffener			EN 1993-1-1:[6.2.1.(5)]
$Q_1 = 284.38$ [kN]	Shear force acting on a stiffener			EN 1993-1-1:[6.2.1.(5)]
$z_n = 128$ [mm]	Location of the neutral axis (from the plate base)			EN 1993-1-1:[6.2.1.(5)]
$I_s = 20658.38$ [cm ⁴]	Moment of inertia of a stiffener			EN 1993-1-1:[6.2.1.(5)]
$\sigma_d = 10.12$ [MPa]	Normal stress on the contact surface between stiffener and plate			(0.29)
$\sigma_g = 30.15$ [MPa]	Normal stress in upper fibers			
$\tau = 45.57$ [MPa]	Tangent stress in a stiffener			
$\sigma_z = 79.58$ [MPa]	Equivalent stress on the contact surface between stiffener and plate	verified		[4.5.3.(7)]
$\max(\sigma_g, \tau / (0.58), \sigma_z) / (f_{yp} / \gamma_{M0}) \leq 1.0$ (6.1)	0.29 < 1.00			[4.5.3.(7)]

WELDS BETWEEN THE COLUMN AND THE BASE PLATE

$\sigma = 55.67$ [MPa]	Normal stress in a weld			[4.5.3.(7)]
$\tau_{\perp} = 55.67$ [MPa]	Perpendicular tangent stress			[4.5.3.(7)]
$\tau_{\parallel} = 0.00$ [MPa]	Tangent stress parallel to $V_{j,Ed,y}$			[4.5.3.(7)]
$\tau_{\parallel} = -6.00$ [MPa]	Tangent stress parallel to $V_{j,Ed,z}$			[4.5.3.(7)]
$\beta = 0.80$	Resistance-dependent coefficient	verified		(0.21)
$\beta / (0.9 \cdot f_{u} / \gamma_{M2}) \leq 1.0$ (4.1)	0.21 < 1.00	verified		(0.31)
	0.31 < 1.00	verified		(0.28)
	0.28 < 1.00			

VERTICAL WELDS OF STIFFENERS

Trapezoid plate parallel to the column web					
$s_{\perp}$ =	0.00	[MPa]	Normal stress in a weld		[4.5.3.(7)]
$t_{\perp}$ =	0.00	[MPa]	Perpendicular tangent stress		[4.5.3.(7)]
$t_{\parallel}$ =	167.71	[MPa]	Parallel tangent stress		[4.5.3.(7)]
s_z =	0.00	[MPa]	Total equivalent stress		[4.5.3.(7)]
b_W =	0.80		Resistance-dependent coefficient		[4.5.3.(7)]
$\max (s_{\perp}, t_{\parallel})$ 			0.81 < 1.00	verified	(0.81)

TRANSVERSAL WELDS OF STIFFENERS

Trapezoid plate parallel to the column web					[4.5.3.(7)]
$s_{\perp}$	=	134.06	[MPa]	Normal stress in a weld	[4.5.3.(7)]
$t_{\perp}$	=	134.06	[MPa]	Perpendicular tangent stress	[4.5.3.(7)]
$t_{\parallel}$	=	83.31	[MPa]	Parallel tangent stress	[4.5.3.(7)]
s_2	=	304.48	[MPa]	Total equivalent stress	[4.5.3.(7)]
b_W	=	0.80		Resistance-dependent coefficient	(0.71)
$\max(s_{\perp}, t_{\parallel})$				0.71 < 1.00	verified

CONNECTION STIFFNESS

Bending moment $M_{j,Ed,y}$					[6.2.5.(3)]
b_{eff}	=	140	[mm]	Effective width of the bearing pressure zone under the flange	[6.2.5.(3)]
l_{eff}	=	325	[mm]	Effective length of the bearing pressure zone under the flange	
$k_{13,y} = E_c$					[Table 6.11]
$k_{13,y}$	=	24	[mm]	Stiffness coeff. of compressed concrete	[6.2.6.5]
l_{eff}	=	472	[mm]	Effective length for a single bolt for mode 2	[6.2.6.5]
m	=	107	[mm]	Distance of a bolt from the stiffening edge	
$k_{15,y} = 0.850 \cdot l_{eff} \cdot t_p^3 / (m^3)$					[Table 6.11]
$k_{15,y}$	=	9	[mm]	Stiffness coeff. of the base plate subjected to tension	[Table 6.11]
L_b	=	320	[mm]	Effective anchorage depth	[Table 6.11]
$k_{16,y} = 1.6 \cdot A_b / L_b$					[Table 6.11]
$k_{16,y}$	=	3	[mm]	Stiffness coeff. of an anchor subjected to tension	
$l_{0,y}$	=	0.47		Column slenderness	[5.2.2.5.(2)]
$S_{j,ini,y}$	=	43547.32	[kN*m]	Initial rotational stiffness	[Table 6.12]
$S_{j,rig,y}$	=	383698.95	[kN*m]	Stiffness of a rigid connection	[5.2.2.5]
$S_{j,ini,y} < S_{j,rig,y}$ SEMI-RIGID					[5.2.2.5.(2)]

WEAKEST COMPONENT:

BASE PLATE - BENDING

Connection conforms to the code

Ratio 0.94



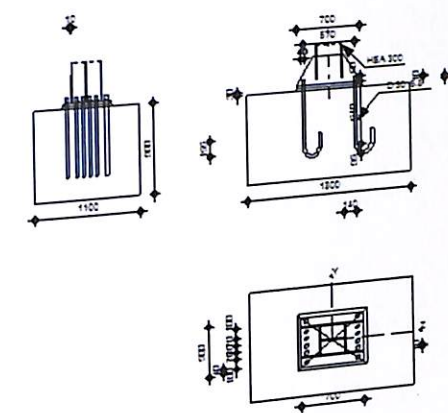
Autodesk Robot Structural Analysis Professional 2016

Fixed column base design

Eurocode 3: EN 1993-1-8:2005/AC:2009 + CEB Design Guide: Design of fastenings in concrete



Ratio
0.90



GENERAL

Connection no.: 7
 Connection name: OT3
 Structure node: 5
 Structure bars: 4

GEOMETRY

COLUMN

Section: HEA 300
 Bar no.: 4

$L_c =$			
$a =$	4.40	[m]	Column length
$h_c =$	0.0	[Deg]	Inclination angle
$b_{fc} =$	290	[mm]	Height of column section
$t_{wc} =$	300	[mm]	Width of column section
$t_{fc} =$	9	[mm]	Thickness of the web of column section
$t_c =$	14	[mm]	Thickness of the flange of column section
$r_c =$	27	[mm]	Radius of column section fillet
$I_{yc} =$	112.53	[cm ²]	Cross-sectional area of a column
$I_{yc} =$	18263.50	[cm ⁴]	Moment of inertia of the column section
Material: S235			
$f_{yc} =$	235.00	[MPa]	Resistance
$f_{uc} =$	360.00	[MPa]	Yield strength of a material

COLUMN BASE

$l_{pd} =$			
$b_{pd} =$	700	[mm]	Length
$t_{pd} =$	500	[mm]	Width
Material: S 235			
$f_{ypd} =$	235.00	[MPa]	Resistance
$f_{upd} =$	360.00	[MPa]	Yield strength of a material

ANCHORAGE

The shear plane passes through the UNTHREADED portion of the bolt.

Class =			
$f_{yb} =$	6.8		Anchor class
$f_{ub} =$	480.00	[MPa]	Yield strength of the anchor material
$d =$	600.00	[MPa]	Tensile strength of the anchor material
$A_s =$	30	[mm]	Bolt diameter
$A_v =$	5.61	[cm ²]	Effective section area of a bolt
$n_H =$	7.07	[cm ²]	Area of bolt section
$n_V =$	2		Number of bolt columns
	6		Number of bolt rows

Horizontal spacing $e_{H1} =$ 570 [mm]
 Vertical spacing $e_{V1} =$ 70; 70; 100 [mm]

Anchor dimensions

$L_1 =$	60	[mm]
$L_2 =$	640	[mm]
$L_3 =$	190	[mm]
$L_4 =$	155	[mm]

Washer

$l_{wq} =$	40	[mm]	Length
------------	----	------	--------

$b_{wd} =$ 40 [mm] Width
 $t_{wd} =$ 10 [mm] Thickness

STIFFENER

$l_s =$ 700 [mm] Length
 $h_s =$ 290 [mm] Height
 $t_s =$ 10 [mm] Thickness
 $d_1 =$ 20 [mm] Cut
 $d_2 =$ 20 [mm] Cut

MATERIAL FACTORS

$\gamma_{M0} =$ 1.00 Partial safety factor
 $\gamma_{M2} =$ 1.25 Partial safety factor
 $\gamma_{M3} =$ 1.50 Partial safety factor

SPREAD FOOTING

$L =$ Spread footing length
 $B =$ 1800 [mm] Spread footing width
 $H =$ 1100 [mm] Spread footing height
 900 [mm]

Concrete
Class C12/15

$f_{ct} =$ 12.00 [MPa] Characteristic resistance for compression

Grout layer

$t_g =$ Thickness of leveling layer (grout)
 $f_{ct,g} =$ 30 [mm] Characteristic resistance for compression
 $C_{f,d} =$ 12.00 [MPa] Characteristic resistance between the base plate and concrete
0.30 Coeff. of friction between the base plate and concrete

WELDS

$a_p =$ Footing plate of the column base
 $a_s =$ 5 [mm] Stiffeners
4 [mm]

LOADS

Case: 692: $t_k 9 (1+6) \cdot 1.35 + (5+10) \cdot 1.50 + (8+13) \cdot 0.90$

$N_{Ed} =$
 $V_{Ed,z} =$ -87.26 [kN] Axial force
 $M_{Ed,y} =$ -69.00 [kN] Shear force
166.57 [kN*m] Bending moment

RESULTS

COMPRESSION ZONE

COMPRESSION OF CONCRETE

$f_{cd} =$ Design compressive resistance
 $f_j =$ 8.00 [MPa]
 $c = t_p \diamond$ 11.96 [MPa] Design bearing resistance under the base plate
 $c =$
 $b_{eff} =$ 72 [mm] Additional width of the bearing pressure zone
 $l_{eff} =$ 157 [mm] Effective width of the bearing pressure zone under the flange
 $A_{c0} =$ 443 [mm] Effective length of the bearing pressure zone under the flange
 $A_{c1} =$ 697.42 [cm²] Area of the joint between the base plate and the foundation
 $F_{Rd} = A_{c0} \cdot f_{cd} \diamond$ 5191.49 [cm²] Maximum design area of load distribution
 $F_{Rd} =$
 $b_j =$ 1522.24 [kN] Bearing resistance of concrete
 $f_{jd} = b_j \cdot F_{Rd} / (b_{eff} \cdot l_{eff})$ 0.67 Reduction factor for compression
 $f_{jd} =$ 14.55 [MPa] Design bearing resistance
 $A_{c,n} =$
 $A_{c,y} =$ 2819.61 [cm²] Bearing area for compression
 $F_{c,Rd,i} = A_{c,i} \cdot f_{jd}$ 1137.75 [cm²] Bearing area for bending My
 $F_{c,Rd,n} =$ 4102.86 [kN] Bearing resistance of concrete for compression
 $F_{c,Rd,y} =$ 1655.57 [kN] Bearing resistance of concrete for bending My

EN 1992-1:[3.1.6.(1)]
[6.2.5.(7)]

[6.2.5.(4)]
[6.2.5.(3)]
[6.2.5.(3)]

EN 1992-1:[6.7.(3)]
EN 1992-1:[6.7.(3)]

EN 1992-1:[6.7.(3)]
[6.2.5.(7)]

[6.2.5.(7)]

[6.2.8.2.(1)]
[6.2.8.3.(1)]

[6.2.8.2.(1)]
[6.2.8.3.(1)]

COLUMN FLANGE AND WEB IN COMPRESSION

CL =	2.00		Section class	EN 1993-1-1:[5.5.2]
W _{pl,y} =	3833.40	[cm ³]	Plastic section modulus	EN1993-1-1:[6.2.5.(2)]
M _{c,Rd,y} =	900.85	[kN*m]	Design resistance of the section for bending	EN1993-1-1:[6.2.5]
h _{f,y} =	380	[mm]	Distance between the centroids of flanges	[6.2.6.7.(1)]
F _{c,fc,Rd,y} = M _{c,Rd,y} / h _{f,y}				
F _{c,fc,Rd,y} =	2367.83	[kN]	Resistance of the compressed flange and web	[6.2.6.7.(1)]

RESISTANCES OF SPREAD FOOTING IN THE COMPRESSION ZONE

N _{j,Rd} = F _{c,Rd,n}				
N _{j,Rd} =	4102.86	[kN]	Resistance of a spread footing for axial compression	[6.2.8.2.(1)]
F _{C,Rd,y} = min(F _{c,Rd,y} , F _{c,fc,Rd,y})				
F _{C,Rd,y} =	1655.57	[kN]	Resistance of spread footing in the compression zone	[6.2.8.3]

TENSION ZONE

STEEL FAILURE

A _b =	5.61	[cm ²]	Effective anchor area	[Table 3.4]
f _{ub} =	600.00	[MPa]	Tensile strength of the anchor material	[Table 3.4]
Beta =	0.85		Reduction factor of anchor resistance	[3.6.1.(3)]
F _{t,Rd,s1} = beta*0.9*f _{ub} *A _b /γ _{M2}				
F _{t,Rd,s1} =	206.00	[kN]	Anchor resistance to steel failure	[Table 3.4]
γ _{M5} =	1.20		Partial safety factor	CEB [3.2.3.2]
f _{yb} =	480.00	[MPa]	Yield strength of the anchor material	CEB [9.2.2]
F _{t,Rd,s2} = f _{yb} *A _b /γ _{M5}				
F _{t,Rd,s2} =	224.40	[kN]	Anchor resistance to steel failure	CEB [9.2.2]
F _{t,Rd,s} = min(F _{t,Rd,s1} , F _{t,Rd,s2})				
F _{t,Rd,s} =	206.00	[kN]	Anchor resistance to steel failure	

PULL-OUT FAILURE

f _{ck} =	12.00	[MPa]	Characteristic compressive strength of concrete	EN 1992-1:[3.1.2]
f _{ctd} = 0.7*0.3*f _{ck} ^{2/3} /γ _C				
f _{ctd} =	0.73	[MPa]	Design tensile resistance	EN 1992-1:[8.4.2.(2)]
h ₁ =	1.00		Coeff. related to the quality of the bond conditions and concreting conditions	EN 1992-1:[8.4.2.(2)]
h ₂ =	1.00		Coeff. related to the bar diameter	EN 1992-1:[8.4.2.(2)]
f _{bd} = 2.25*h ₁ *h ₂ *f _{ctd}				
f _{bd} =	1.65	[MPa]	Design value of the ultimate bond stress	EN 1992-1:[8.4.2.(2)]
h _{ef} =	640	[mm]	Effective anchorage depth	EN 1992-1:[8.4.2.(2)]
F _{t,Rd,p} = p*d*h _{ef} *f _{bd}				
F _{t,Rd,p} =	99.59	[kN]	Design uplift capacity	EN 1992-1:[8.4.2.(2)]

CONCRETE CONE FAILURE

h _{ef} =	410	[mm]	Effective anchorage depth	CEB [9.2.4]
N _{Rk,c} ⁰ = 7.5[N ^{0.5} /mm ^{0.5}]*f _{ck} *h _{ef} ^{1.5}				
N _{Rk,c} ⁰ =	215.69	[kN]	Characteristic resistance of an anchor	CEB [9.2.4]
s _{cr,N} =	1230	[mm]	Critical width of the concrete cone	CEB [9.2.4]
c _{cr,N} =	615	[mm]	Critical edge distance	CEB [9.2.4]
A _{c,N0} =	29520.00	[cm ²]	Maximum area of concrete cone	CEB [9.2.4]
A _{c,N} =	19800.00	[cm ²]	Actual area of concrete cone	CEB [9.2.4]
γ _{A,N} = A _{c,N} /A _{c,N0}				
γ _{A,N} =	0.67		Factor related to anchor spacing and edge distance	CEB [9.2.4]
c =	345	[mm]	Minimum edge distance from an anchor	CEB [9.2.4]
γ _{s,N} = 0.7 + 0.3*c/c _{cr,N} ≤ 1.0				
γ _{s,N} =	0.87		Factor taking account the influence of edges of the concrete member on the distribution of stresses in the concrete	CEB [9.2.4]
γ _{ec,N} =	1.00		Factor related to distribution of tensile forces acting on anchors	CEB [9.2.4]
γ _{re,N} = 0.5 + h _{ef} [mm]/200 ≤ 1.0				
γ _{re,N} =	1.00		Shell spalling factor	CEB [9.2.4]
γ _{ucr,N} =	1.00		Factor taking into account whether the anchorage is in cracked or non-cracked concrete	CEB [9.2.4]
γ _{Mc} =	2.16		Partial safety factor	CEB [3.2.3.1]
F _{t,Rd,c} = N _{Rk,c} ⁰ *γ _{A,N} *γ _{s,N} *γ _{ec,N} *γ _{re,N} *γ _{ucr,N} /γ _{Mc}				
F _{t,Rd,c} =	58.16	[kN]	Design anchor resistance to concrete cone failure	EN 1992-1:[8.4.2.(2)]

SPLITTING FAILURE

$h_{ef} =$	640	[mm]	Effective anchorage depth	CEB [9.2.5]
$N_{Rk,c}^0 = 7.5[N^{0.5}/mm^{0.5}] \cdot f_{ck} \cdot h_{ef}^{1.5}$				CEB [9.2.5]
$N_{Rk,c}^0 =$	420.65	[kN]	Design uplift capacity	CEB [9.2.5]
$s_{cr,N} =$	1280	[mm]	Critical width of the concrete cone	CEB [9.2.5]
$c_{cr,N} =$	640	[mm]	Critical edge distance	CEB [9.2.5]
$A_{c,N0} =$	31265.00	[cm ²]	Maximum area of concrete cone	
$A_{c,N} =$	19800.00	[cm ²]	Actual area of concrete cone	CEB [9.2.5]
$y_{AN} = A_{c,N}/A_{c,N0}$			Factor related to anchor spacing and edge distance	CEB [9.2.5]
$y_{AN} =$	0.63			
$c =$	345	[mm]	Minimum edge distance from an anchor	CEB [9.2.5]
$y_{s,N} = 0.7 + 0.3 \cdot c/c_{cr,N} \leq 1.0$				CEB [9.2.5]
$y_{s,N} =$	0.86		Factor taking account the influence of edges of the concrete member on the distribution of stresses in the concrete	CEB [9.2.5]
$y_{ec,N} =$	1.00		Factor related to distribution of tensile forces acting on anchors	CEB [9.2.5]
$y_{re,N} = 0.5 + h_{ef}[mm]/200 \leq 1.0$				CEB [9.2.5]
$y_{re,N} =$	1.00		Shell spalling factor	
$y_{ucr,N} =$	1.00		Factor taking into account whether the anchorage is in cracked or non-cracked concrete	CEB [9.2.5]
$y_{h,N} = (h/(2 \cdot h_{ef}))^{2/3} \leq 1.2$				CEB [3.2.3.1]
$y_{h,N} =$	0.79		Coeff. related to the foundation height	
$\xi_{M,sp} =$	2.16		Partial safety factor	CEB [9.2.5]
$F_{t,Rd,sp} = N_{Rk,c}^0 \cdot y_{AN} \cdot y_{s,N} \cdot y_{ec,N} \cdot y_{re,N} \cdot y_{ucr,N} \cdot y_{h,N} / \xi_{M,sp}$				
$F_{t,Rd,sp} =$	84.04	[kN]	Design anchor resistance to splitting of concrete	

TENSILE RESISTANCE OF AN ANCHOR

$F_{t,Rd} = \min(F_{t,Rd,s}, F_{t,Rd,p}, F_{t,Rd,c}, F_{t,Rd,sp})$				
$F_{t,Rd} =$	58.16	[kN]	Tensile resistance of an anchor	[6.2.6.5]

BENDING OF THE BASE PLATE

Bending moment $M_{j,Ed,y}$				
$l_{eff,1} =$	481	[mm]	Effective length for a single bolt for mode 1	[6.2.6.5]
$l_{eff,2} =$	481	[mm]	Effective length for a single bolt for mode 2	[6.2.4]
$m =$	109	[mm]	Distance of a bolt from the stiffening edge	[6.2.4]
$M_{pl,1,Rd} =$	22.16	[kN*m]	Plastic resistance of a plate for mode 1	[6.2.4]
$M_{pl,2,Rd} =$	22.16	[kN*m]	Plastic resistance of a plate for mode 2	[6.2.4]
$F_{T,1,Rd} =$	810.68	[kN]	Resistance of a plate for mode 1	
$F_{T,2,Rd} =$	374.00	[kN]	Resistance of a plate for mode 2	[6.2.4]
$F_{T,3,Rd} =$	348.93	[kN]	Resistance of a plate for mode 3	
$F_{t,pl,Rd,y} = \min(F_{T,1,Rd}, F_{T,2,Rd}, F_{T,3,Rd})$				
$F_{t,pl,Rd,y} =$	348.93	[kN]	Tension resistance of a plate	[6.2.8.3]

RESISTANCES OF SPREAD FOOTING IN THE TENSION ZONE

$F_{t,Rd,y} = F_{t,pl,Rd,y}$				
$F_{t,Rd,y} =$	348.93	[kN]	Resistance of a column base in the tension zone	(0.02)

CONNECTION CAPACITY CHECK

$N_{j,Ed} / N_{j,Rd} \leq 1.0$ (6.24)		0.02 < 1.00	verified	[6.2.8.3]
$e_y =$	1909	[mm]	Axial force eccentricity	[6.2.8.1.(2)]
$z_{c,y} =$	190	[mm]	Lever arm $F_{c,Rd,y}$	[6.2.8.1.(3)]
$z_{t,y} =$	285	[mm]	Lever arm $F_{t,Rd,y}$	[6.2.8.3]
$M_{j,Rd,y} =$	184.18	[kN*m]	Connection resistance for bending	(0.90)
$M_{j,Ed,y} / M_{j,Rd,y} \leq 1.0$ (6.23)		0.90 < 1.00	verified	

SHEAR

BEARING PRESSURE OF AN ANCHOR BOLT ONTO THE BASE PLATE

Shear force $V_{j,Ed,z}$				
$a_{b,z} =$	5.69		Coeff. taking account of the bolt position - in the direction of shear	[Table 3.4]
$k_{1,z} =$	1.00		Coeff. for resistance calculation $F_{1,vb,Rd}$	[Table 3.4]
$F_{1,vb,Rd,z} = k_{1,z} \cdot a_{b,z} \cdot f_{up} \cdot d \cdot t_p / \xi_{M2}$			Coeff. taking account of the bolt position - perpendicularly to the direction of shear	[Table 3.4]
$F_{1,vb,Rd,z} =$	329.62	[kN]	Resistance of an anchor bolt for bearing pressure onto the base plate	[6.2.2.(7)]

SHEAR OF AN ANCHOR BOLT

$a_b =$	0.30		Coeff. for resistance calculation $F_{2,vb,Rd}$	[6.2.2.(7)]
$A_{vb} =$	7.07	[cm ²]	Area of bolt section	[6.2.2.(7)]
$f_{ub} =$	600.00	[MPa]	Tensile strength of the anchor material	[6.2.2.(7)]
$\gamma_{M2} =$	1.25		Partial safety factor	[6.2.2.(7)]

$F_{2,vb,Rd} = a_b \cdot f_{ub} \cdot A_{vb} / \gamma_{M2}$				
$F_{2,vb,Rd} =$	100.43	[kN]	Shear resistance of a bolt - without lever arm	CEB [9.3.2.2]

$a_M =$	2.00		Factor related to the fastening of an anchor in the foundation	CEB [9.3.2.2]
$M_{Rk,s} =$	1.46	[kN*m]	Characteristic bending resistance of an anchor	CEB [9.3.2.2]
$l_{sm} =$	59	[mm]	Lever arm length	CEB [3.2.3.2]
$\gamma_{Ms} =$	1.20		Partial safety factor	CEB [9.3.1]

$F_{v,Rd,sm} = a_M \cdot M_{Rk,s} / (l_{sm} \cdot \gamma_{Ms})$				
$F_{v,Rd,sm} =$	41.28	[kN]	Shear resistance of a bolt - with lever arm	CEB [9.2.4]

CONCRETE PRY-OUT FAILURE				
$N_{Rk,c} =$	125.62	[kN]	Design uplift capacity	CEB [9.3.3]
$k_3 =$	2.00		Factor related to the anchor length	CEB [3.2.3.1]
$\gamma_{Mc} =$	2.16		Partial safety factor	CEB [9.3.1]

$F_{v,Rd,cp} = k_3 \cdot N_{Rk,c} / \gamma_{Mc}$				
$F_{v,Rd,cp} =$	116.31	[kN]	Concrete resistance for pry-out failure	CEB [9.3.4.(a)]

CONCRETE EDGE FAILURE				
Shear force $V_{j,Ed,z}$				
$V_{Rk,c,z}^0 = 1099.14$	[kN]	Characteristic resistance of an anchor	CEB [9.3.4]	

$y_{A,V,z} =$	0.36		Factor related to anchor spacing and edge distance	CEB [9.3.4.(d)]
$y_{h,V,z} =$	1.01		Factor related to the foundation thickness	CEB [9.3.4.(e)]
$y_{s,V,z} =$	0.81		Factor related to the influence of edges parallel to the shear load direction	CEB [9.3.4.(f)]
$y_{ec,V,z} =$	1.00		Factor taking account a group effect when different shear loads are acting on the individual anchors in a group	CEB [9.3.4.(g)]
$y_{a,V,z} =$	1.00		Factor related to the angle at which the shear load is applied	CEB [3.2.3.1]
$y_{ucr,V,z} =$	1.00		Factor related to the type of edge reinforcement used	
$\gamma_{Mc} =$	2.16		Partial safety factor	CEB [9.3.1]

$F_{v,Rd,c,z} = V_{Rk,c,z}^0 \cdot y_{A,V,z} \cdot y_{h,V,z} \cdot y_{s,V,z} \cdot y_{ec,V,z} \cdot y_{a,V,z} \cdot y_{ucr,V,z} / \gamma_{Mc}$				
$F_{v,Rd,c,z} =$	152.04	[kN]	Concrete resistance for edge failure	[6.2.2.(6)]

SPLITTING RESISTANCE				
$C_{f,d} =$	0.30		Coeff. of friction between the base plate and concrete	[6.2.2.(6)]
$N_{c,Ed} =$	87.26	[kN]	Compressive force	[6.2.2.(6)]

$F_{f,Rd} = C_{f,d} \cdot N_{c,Ed}$				
$F_{f,Rd} =$	26.18	[kN]	Slip resistance	CEB [9.3.1]

SHEAR CHECK				
$V_{j,Rd,z} = n_b \cdot \min(F_{1,vb,Rd,z}, F_{2,vb,Rd}, F_{v,Rd,sm}, F_{v,Rd,cp}, F_{v,Rd,c,z}) + F_{f,Rd}$				verified
$V_{j,Rd,z} =$	521.49	[kN]	Connection resistance for shear	(0.13)
$V_{j,Ed,z} / V_{j,Rd,z} \leq 1.0$				

STIFFENER CHECK				
Trapezoid plate parallel to the column web				
$M_1 =$	22.09	[kN*m]	Bending moment acting on a stiffener	EN 1993-1-1:[6.2.1.(5)]
$Q_1 =$	157.79	[kN]	Shear force acting on a stiffener	EN 1993-1-1:[6.2.1.(5)]
$z_s =$	59	[mm]	Location of the neutral axis (from the plate base)	EN 1993-1-1:[6.2.1.(5)]
$I_s =$	7322.93	[cm ⁴]	Moment of inertia of a stiffener	EN 1993-1-1:[6.2.1.(5)]
$\sigma_g =$	9.44	[MPa]	Normal stress on the contact surface between stiffener and plate	(0.40)
$\sigma =$	78.04	[MPa]	Normal stress in upper fibers	
$\sigma_z =$	54.41	[MPa]	Tangent stress in a stiffener	
$\max(\sigma_g, \sigma_1 / (0.58), \sigma_z) / (f_{yp} / \gamma_{M0}) \leq 1.0$			Equivalent stress on the contact surface between stiffener and plate	verified
$\max(\sigma_g, \sigma_1 / (0.58), \sigma_z) / (f_{yp} / \gamma_{M0}) =$	94.71	[MPa]		0.40 < 1.00

WELDS BETWEEN THE COLUMN AND THE BASE PLATE				
$\sigma =$	32.51	[MPa]	Normal stress in a weld	[4.5.3.(7)]
$\tau_{\perp} =$	32.51	[MPa]	Perpendicular tangent stress	[4.5.3.(7)]
$\tau_{\parallel} =$	0.00	[MPa]	Tangent stress parallel to $V_{j,Ed,y}$	[4.5.3.(7)]
$\tau_{\parallel} =$	-4.99	[MPa]	Tangent stress parallel to $V_{j,Ed,z}$	[4.5.3.(7)]

$b_{w1} =$	0.80	Resistance-dependent coefficient	verified	[4.5.3.(7)]
$s_1 / ((0.9 \cdot f_{td} / g_{m2})) \leq 1.0$ (4.1)			verified	(0.13)
$\diamond$			verified	(0.18)
$\diamond$			verified	(0.16)

VERTICAL WELDS OF STIFFENERS

Trapezoid plate parallel to the column web					[4.5.3.(7)]
$s_1 =$	0.00	[MPa]	Normal stress in a weld		[4.5.3.(7)]
$t_1 =$	0.00	[MPa]	Perpendicular tangent stress		[4.5.3.(7)]
$t_{11} =$	81.90	[MPa]	Parallel tangent stress		[4.5.3.(7)]
$s_2 =$	0.00	[MPa]	Total equivalent stress		(0.39)
$b_{w1} =$	0.80		Resistance-dependent coefficient	verified	
$\max(s_1, t_{11}) \cdot \diamond$				0.39 < 1.00	

TRANSVERSAL WELDS OF STIFFENERS

Trapezoid plate parallel to the column web					[4.5.3.(7)]
$s_1 =$	68.03	[MPa]	Normal stress in a weld		[4.5.3.(7)]
$t_1 =$	68.03	[MPa]	Perpendicular tangent stress		[4.5.3.(7)]
$t_{11} =$	88.82	[MPa]	Parallel tangent stress		[4.5.3.(7)]
$s_2 =$	205.38	[MPa]	Total equivalent stress		(0.57)
$b_{w1} =$	0.80		Resistance-dependent coefficient	verified	
$\max(s_1, t_{11}) \cdot \diamond$				0.57 < 1.00	

CONNECTION STIFFNESS

Bending moment $M_{j,Ed,y}$					
$b_{eff} =$	157	[mm]	Effective width of the bearing pressure zone under the flange		[Table 6.11]
$l_{eff} =$	443	[mm]	Effective length of the bearing pressure zone under the flange		[6.2.5.(3)]
$k_{13,y} = E_c \cdot \diamond$					[6.2.5.(3)]
$k_{13,y} =$	27	[mm]	Stiffness coeff. of compressed concrete		[6.2.5.(3)]
$l_{eff} =$	481	[mm]	Effective length for a single bolt for mode 2		[Table 6.11]
$m =$	109	[mm]	Distance of a bolt from the stiffening edge		[Table 6.11]
$k_{15,y} = 0.850 \cdot l_{eff} \cdot t_p^3 / (m^3)$					[Table 6.11]
$k_{15,y} =$	7	[mm]	Stiffness coeff. of the base plate subjected to tension		[Table 6.11]
$L_b =$	323	[mm]	Effective anchorage depth		[5.2.2.5.(2)]
$k_{16,y} = 1.6 \cdot A_b / L_b$					[Table 6.12]
$k_{16,y} =$	3	[mm]	Stiffness coeff. of an anchor subjected to tension		[5.2.2.5]
$l_{0,y} =$	0.37		Column slenderness		[5.2.2.5.(2)]
$S_{j,ini,y} =$	41773.89	[kN*m]	Initial rotational stiffness		
$S_{j,rig,y} =$	261500.11	[kN*m]	Stiffness of a rigid connection		
$S_{j,ini,y} < S_{j,rig,y}$			SEMI-RIGID		

WEAKEST COMPONENT:

FOUNDATION - CONCRETE CONE PULL-OUT FAILURE Ratio 0.90

Connection conforms to the code

OK
Ratio 0.60



$b_{wy} =$	0.80	Resistance-dependent coefficient	verified	[4.5.3.(7)]
$s_r / (0.9 \cdot f_{td} / \gamma_{M2}) \leq 1.0$ (4.1)		0.13 < 1.00	verified	(0.13)
$\diamond$		0.18 < 1.00	verified	(0.18)
$\diamond$		0.16 < 1.00	verified	(0.16)

VERTICAL WELDS OF STIFFENERS

Trapezoid plate parallel to the column web					[4.5.3.(7)]
$s_r =$	0.00	[MPa]	Normal stress in a weld		[4.5.3.(7)]
$t_r =$	0.00	[MPa]	Perpendicular tangent stress		[4.5.3.(7)]
$t_{ } =$	81.90	[MPa]	Parallel tangent stress		[4.5.3.(7)]
$s_2 =$	0.00	[MPa]	Total equivalent stress		(0.39)
$b_{wy} =$	0.80		Resistance-dependent coefficient	verified	
$\max(s_r, t_{ }) \cdot \diamond$			0.39 < 1.00		

TRANSVERSAL WELDS OF STIFFENERS

Trapezoid plate parallel to the column web					[4.5.3.(7)]
$s_r =$	68.03	[MPa]	Normal stress in a weld		[4.5.3.(7)]
$t_r =$	68.03	[MPa]	Perpendicular tangent stress		[4.5.3.(7)]
$t_{ } =$	88.82	[MPa]	Parallel tangent stress		[4.5.3.(7)]
$s_2 =$	205.38	[MPa]	Total equivalent stress		(0.57)
$b_{wy} =$	0.80		Resistance-dependent coefficient	verified	
$\max(s_r, t_{ }) \cdot \diamond$			0.57 < 1.00		

CONNECTION STIFFNESS

Bending moment $M_{j,Ed,y}$					[6.2.5.(3)]
$b_{eff} =$	157	[mm]	Effective width of the bearing pressure zone under the flange		[6.2.5.(3)]
$l_{eff} =$	443	[mm]	Effective length of the bearing pressure zone under the flange		[Table 6.11]
$k_{13,y} = E_c \cdot \diamond$					[6.2.6.5]
$k_{13,y} =$	27	[mm]	Stiffness coeff. of compressed concrete		[6.2.6.5]
$l_{eff} =$	481	[mm]	Effective length for a single bolt for mode 2		[Table 6.11]
$m =$	109	[mm]	Distance of a bolt from the stiffening edge		[Table 6.11]
$k_{15,y} = 0.850 \cdot l_{eff} \cdot t_p^3 / (m^3)$			Stiffness coeff. of the base plate subjected to tension		[Table 6.11]
$k_{15,y} =$	7	[mm]			[Table 6.11]
$L_b =$	323	[mm]	Effective anchorage depth		[5.2.2.5.(2)]
$k_{16,y} = 1.6 \cdot A_b / L_b$			Stiffness coeff. of an anchor subjected to tension		[Table 6.12]
$k_{16,y} =$	3	[mm]			[5.2.2.5]
$I_{0,y} =$	0.37		Column slenderness		[5.2.2.5.(2)]
$S_{j,ini,y} =$	41773.89	[kN*m]	Initial rotational stiffness		
$S_{j,rig,y} =$	261500.11	[kN*m]	Stiffness of a rigid connection		
$S_{j,ini,y} < S_{j,rig,y}$			SEMI-RIGID		

WEAKEST COMPONENT:

FOUNDATION - CONCRETE CONE PULL-OUT FAILURE

Ratio 0.90

Connection conforms to the code

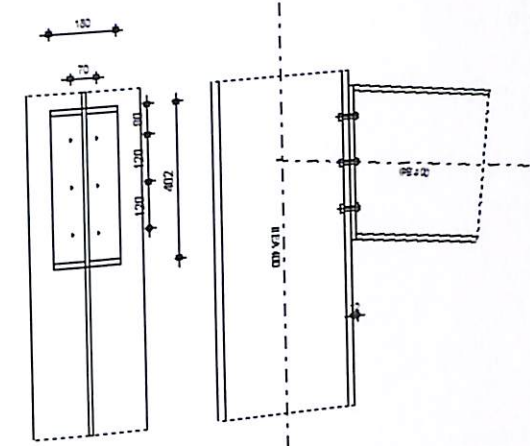
OK
Ratio
0.60



Autodesk Robot Structural Analysis Professional 2016

Design of fixed beam-to-column connection

EN 1993-1-8:2005/AC:2009



GENERAL

Connection no.: 8
 Connection name: FG1
 Structure node: 8
 Structure bars: 3, 7

GEOMETRY

COLUMN

Section: HEA 400
 Bar no.: 3

a =			Inclination angle
$h_c =$	-90.0	[Deg]	Height of column section
$b_{fc} =$	390	[mm]	Width of column section
$t_{wc} =$	300	[mm]	Thickness of the web of column section
$t_{fc} =$	11	[mm]	Thickness of the flange of column section
$r_c =$	19	[mm]	Radius of column section fillet
$A_c =$	27	[mm]	Cross-sectional area of a column
$I_{xc} =$	158.98	[cm ²]	Moment of inertia of the column section
Material: S235	45069.40	[cm ⁴]	
$f_{yc} =$	235.00	[MPa]	Resistance

BEAM

Section: IPE 400
 Bar no.: 7

a =			Inclination angle
$h_b =$	-6.3	[Deg]	Height of beam section
$b_f =$	400	[mm]	Width of beam section
$t_{wb} =$	180	[mm]	Thickness of the web of beam section
$t_{fb} =$	9	[mm]	Thickness of the flange of beam section
$r_b =$	14	[mm]	Radius of beam section fillet
$A_b =$	21	[mm]	Radius of beam section fillet
$I_{xb} =$	21	[mm]	Cross-sectional area of a beam
Material: S235	84.46	[cm ²]	Moment of inertia of the beam section
$f_{yb} =$	23128.40	[cm ⁴]	
	235.00	[MPa]	Resistance

BOLTS

The shear plane passes through the UNTHREADED portion of the bolt.

d =	10	[mm]	Bolt diameter
Class =	6.8		Bolt class
$F_{tRd} =$	25.06	[kN]	Tensile resistance of a bolt
$n_h =$	2		Number of bolt columns
$n_v =$	3		Number of bolt rows
$h_1 =$	80	[mm]	Distance between first bolt and upper edge of front plate
Horizontal spacing $e_1 =$	70	[mm]	
Vertical spacing $p_1 =$	120; 120	[mm]	

PLATE

$h_p =$	402	[mm]	Plate height
$b_p =$	180	[mm]	Plate width

$t_p = 12$ [mm] Plate thickness
Material: S 235
 $f_{yp} = 235.00$ [MPa] Resistance

FILLET WELDS

$a_w = 4$ [mm] Web weld
 $a_f = 4$ [mm] Flange weld

MATERIAL FACTORS

$\gamma_{M0} = 1.00$ Partial safety factor
 $\gamma_{M1} = 1.00$ Partial safety factor
 $\gamma_{M2} = 1.25$ Partial safety factor
 $\gamma_{M3} = 1.25$ Partial safety factor

LOADS

Ultimate limit state

Case: 560: ULS/546=1*1.35 + 6*1.35 + 3*1.05 + 8*0.90 + 13*0.90 + 11*1.50 (1+6)*1.35+3*1.05+(8+13)*0.90+11*1.50

$V_{b1,Ed} = 78.11$ [kN] Shear force in the right beam
 $N_{b1,Ed} = -5.69$ [kN] Axial force in the right beam
 $M_{c1,Ed} = -54.26$ [kN*m] Bending moment in the lower column
 $V_{c1,Ed} = -51.40$ [kN] Shear force in the lower column
 $N_{c1,Ed} = -182.48$ [kN] Axial force in the lower column
 $M_{c2,Ed} = -54.26$ [kN*m] Bending moment in the upper column
 $V_{c2,Ed} = 57.09$ [kN] Shear force in the upper column
 $N_{c2,Ed} = -104.37$ [kN] Axial force in the upper column

RESULTS

BEAM RESISTANCES

COMPRESSION

$A_b = 84.46$ [cm²] Area
 $N_{cb,Rd} = A_b f_{yb} / \gamma_{M0}$
 $N_{cb,Rd} = 1984.89$ [kN] Design compressive resistance of the section
 $N_{b1,Ed} / N_{cb,Rd} \leq 1.0$ 0.00 < 1.00 verified (0.00)

SHEAR

$A_{vb} = 42.69$ [cm²] Shear area
 $V_{cb,Rd} = A_{vb} (f_{yb} / \sqrt{3})$
 $V_{cb,Rd} = 579.27$ [kN] Design sectional resistance for shear
 $V_{b1,Ed} / V_{cb,Rd} \leq 1.0$ 0.13 < 1.00 verified (0.13)

AXIAL FORCES IN BEAM FLANGES

$h_f = 387$ [mm] Distance between the centroids of flanges
 $e_N = 0$ [mm] Axial force eccentricity
 $N_{upp} = N_{b1,Ed} / 2 + (-N_{b1,Ed} e_N + M_{b1,Ed}) / h_f$
 $N_{upp} = -2.85$ [kN] Axial force in the beam top flange
 $N_{low} = N_{b1,Ed} / 2 - (-N_{b1,Ed} e_N + M_{b1,Ed}) / h_f$
 $N_{low} = -2.85$ [kN] Axial force in the beam bottom flange

COLUMN RESISTANCES

WEB PANEL - SHEAR

$M_{b1,Ed} = 0.00$ [kN*m] Bending moment (right beam)
 $M_{b2,Ed} = 0.00$ [kN*m] Bending moment (left beam)
 $V_{c1,Ed} = -51.40$ [kN] Shear force (lower column)
 $V_{c2,Ed} = 57.09$ [kN] Shear force (upper column)
 $z = 253$ [mm] Lever arm
 $V_{wp,Ed} = (M_{b1,Ed} - M_{b2,Ed}) / z - (V_{c1,Ed} - V_{c2,Ed}) / 2$
 $V_{wp,Ed} = 54.25$ [kN] Shear force acting on the web panel

$A_{vs} = 57.33$ [cm²] Shear area of the column web
 $A_{vc} = 57.33$ [cm²] Shear area
 $V_{wp,Rd} = 0.9 * (f_{y,wc} * A_{vc} + f_{y,wp} * A_{vp} + f_{ys} * A_{vd}) / \sqrt{3}$

EN1993-1-1:[6.2.4]

EN1993-1-1:[6.2.4]

EN1993-1-1:[6.2.6.(3)]

EN1993-1-1:[6.2.6.(2)]

[5.3.(3)]

[5.3.(3)]

[5.3.(3)]

[5.3.(3)]

[6.2.5]

[5.3.(3)]

EN1993-1-1:[6.2.6.(3)]

EN1993-1-1:[6.2.6.(3)]

WELD RESISTANCE

$A_w =$	37.00	[cm ²]	Area of all welds	[4.5.3.2(2)]
$A_{wy} =$	10.35	[cm ²]	Area of horizontal welds	[4.5.3.2(2)]
$A_{wz} =$	26.64	[cm ²]	Area of vertical welds	[4.5.3.2(5)]
$I_{wy} =$	6030.61	[cm ⁴]	Moment of inertia of the weld arrangement with respect to the hor. axis	[4.5.3.2(5)]
$s_{\max} = t_{\max} =$	-1.54	[MPa]	Normal stress in a weld	[4.5.3.2(5)]
$s_{\perp} =$	-1.54	[MPa]	Stress in a vertical weld	[4.5.3.2(7)]
$t_{ } =$	29.32	[MPa]	Tangent stress	
$b_w =$	0.80		Correlation coefficient	(0.01)
☐			3.08 < 360.00	verified (0.14)
☐			50.87 < 360.00	verified (0.01)
$s \leq 0.9 \cdot f_u / \sigma_{M2}$			1.54 < 259.20	

CONNECTION STIFFNESS

CONNECTION STIFFNESS

There are no axial forces or moments in the connection. For such a case, the program does not calculate stiffness of the connection.

WEAKEST COMPONENT:

BOLTS - SHEAR

Ratio 0.60

Connection conforms to the code



SZENT ISTVÁN
EGYETEM

YBL

YBL MIKLÓS ÉPÍTÉSTUDOMÁNYI KAR, BUDAPEST

Mechanika és Tartószerkezeti Szakcsoport

Számítási melléklet II.

Kéthajós gyártó-bemutató, hőszigetelt csarnok kiviteli terve
Tartószerkezeti számítási melléklet
Második rész: acélszerkezetű mellékrajz

Tartalom:

- I. Keret méretezése
- II. Keret vizsgálatai
- III. 'Z' szelemen méretezés
- IV. Trapézlemez méretezés

Készítette: Nagy Erika
SZIE-YMÉK, levelezőtagozat
Neptun kód [REDACTED]
Konzulens: Horváth Katalin

**ACÉLSZERKEZETŰ CSARNOK
MELLÉKHAJÓ MÉRETEZÉSE****I. KERET MÉRETEZÉSE****Állandó terhek**

axis-ból

Rétegrend

külső trapézlemez LTP45/0.7
 z szelemen Z200
 hőszigetelés 15 cm
 párazáró fólia 0.1 cm
 belső trapézlemez LTP45/0.7

$g_{ktl} := 0.07$
 $g_z := 0.072$
 $g_{hsz} := 0.12$
 $g_{sz} := 0.014$
 $g_{btl} := 0.07$

Burkolat súlya

$t := 6.6 \text{ m}$ $t_z := 1.6 \text{ m}$

$$p_{burkolat} := t \cdot (g_{ktl} + g_{hsz} + g_{sz} + g_{btl}) + \frac{(t \cdot g_z)}{t_z} = 2.105 \frac{kN}{m}$$

Állandó jellegű hasznos teher gépészet $F_g := 5 \text{ kN}$ **Hóteher** (vetületi teher) $s_k := 1.25$

$b_1 := 18 \text{ m}$
 $b_2 := 6 \text{ m}$

 $\gamma := 2 \frac{kN}{m^3}$ $c_e := 0.8$ szeles $c_t := 1.0$ $h := 2.33 \text{ m}$ $0.8 \leq \mu_w \leq 4$ $\mu_s := 0$ mivel $\alpha < 15^\circ$

$$\mu_w := \frac{(b_1 + b_2)}{2 h} = 5.1 \bar{5}$$

 $t := 6.6 \text{ m}$

$$\mu_w := \frac{\gamma \cdot h}{s_k} = 3.728$$

 $l_s := 2 \cdot h = 4.66 \text{ m}$

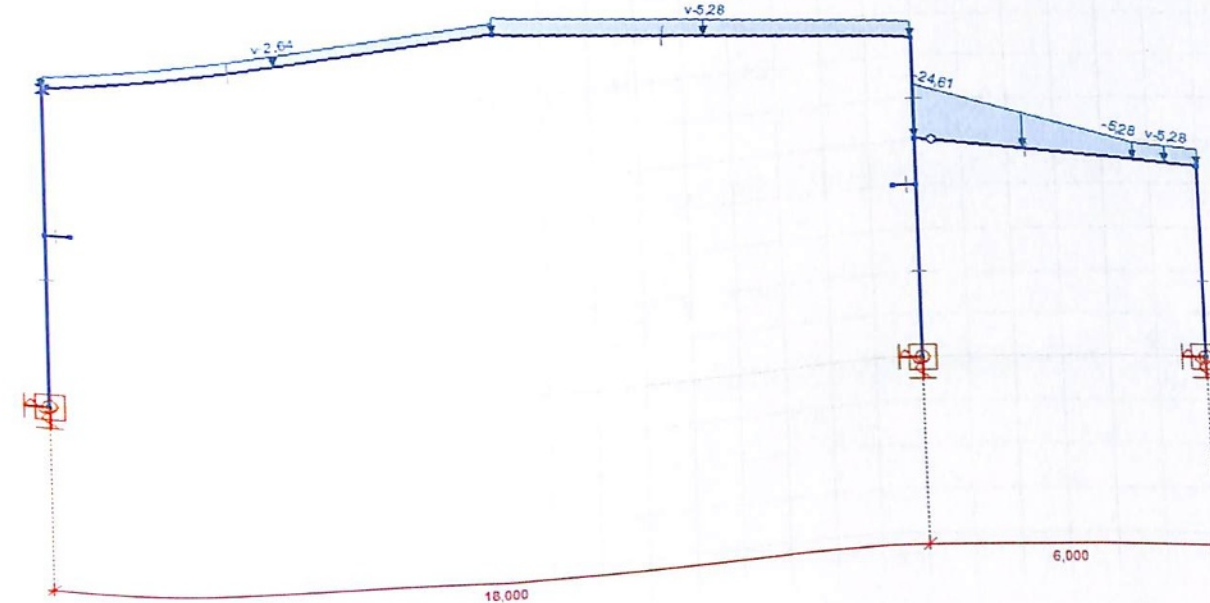
$$\mu_1 := \mu_s + \mu_w = 3.728$$

 $\mu_2 := 0.8$

$$s_1 := c_e \cdot c_t \cdot \mu_1 \cdot s_k = 3.728$$

$$s_2 := c_e \cdot c_t \cdot \mu_2 \cdot s_k = 0.8$$

$$\begin{aligned} \text{psk1} &:= t \cdot s1 = 24.605 \frac{\text{kN}}{\text{m}} & \text{psk1.} &:= t \cdot s2 = 5.28 \frac{\text{kN}}{\text{m}} \\ \text{psk1}' &:= \frac{\text{psk1}}{2} = 12.302 \frac{\text{kN}}{\text{m}} & \text{psk1}' &:= \frac{\text{psk1.}}{2} = 2.64 \frac{\text{kN}}{\text{m}} \end{aligned}$$



Szélteher

Keresztirányú $\theta = 0^\circ$

szél torlónyomás $q_{pz} := 0.2955$

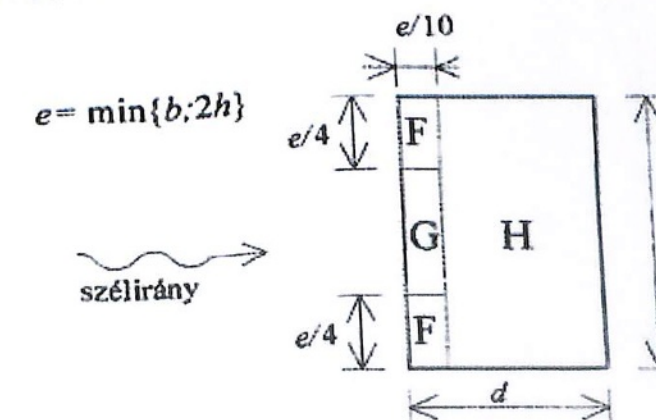
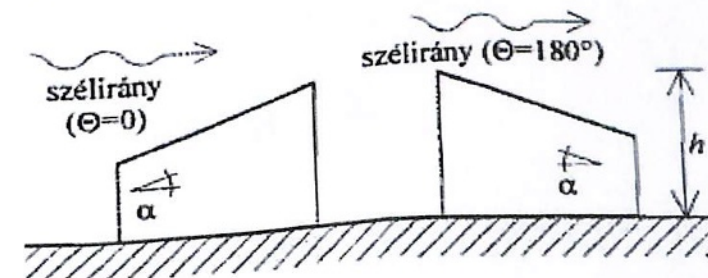
$$z := 5067 + 200 + 200 + 45 = 5512 \text{ mm}$$

$$e := 2 \cdot z = 11024 \text{ mm}$$

$$\frac{e}{4} = 2756 \text{ mm}$$

$$\frac{e}{10} = 1102.4 \text{ mm}$$

Szélirány a tetőfelre merőlegesen



"G" zóna

$$\alpha := 7 \quad t := 6.60 \quad m$$

$$5^\circ \quad c_{pe10} := -1.2$$

$$15^\circ \quad c_{pe10'} := -0.8$$

$$c_{pe1} := \left(\frac{(c_{pe10'} - c_{pe10})}{15 - 5} \right) \cdot (\alpha - 5) + (c_{pe10}) = -1.12$$

$$5^\circ \quad c_{pe.10} := 0.0$$

$$15^\circ \quad c_{pe.10'} := 0.2$$

$$c_{pe2} := \left(\frac{(c_{pe.10'} - c_{pe.10})}{15 - 5} \right) \cdot (\alpha - 5) + (c_{pe.10}) = 0.04$$

$$p_{weszGk'} := t \cdot q_{pz} \cdot c_{pe1} = -2.184 \quad \frac{kN}{m}$$

$$p_{weszGk} := t \cdot q_{pz} \cdot c_{pe2} = 0.078 \quad \frac{kN}{m}$$

"H" zóna

$$\alpha := 7 \quad t := 6.60 \quad m$$

$$5^\circ \quad c_{pe10} := -0.6$$

$$15^\circ \quad c_{pe10'} := -0.3$$

$$c_{pe1} := \left(\frac{(c_{pe10'} - c_{pe10})}{15 - 5} \right) \cdot (\alpha - 5) + (c_{pe10}) = -0.54$$

$$5^\circ \quad c_{pe.10} := 0.0$$

$$15^\circ \quad c_{pe.10'} := 0.2$$

$$c_{pe2} := \left(\frac{(c_{pe.10'} - c_{pe.10})}{15 - 5} \right) \cdot (\alpha - 5) + (c_{pe.10}) = 0.04$$

$$p_{weszHk'} := t \cdot q_{pz} \cdot c_{pe1} = -1.053 \quad \frac{kN}{m}$$

$$p_{weszHk} := t \cdot q_{pz} \cdot c_{pe2} = 0.078 \quad \frac{kN}{m}$$

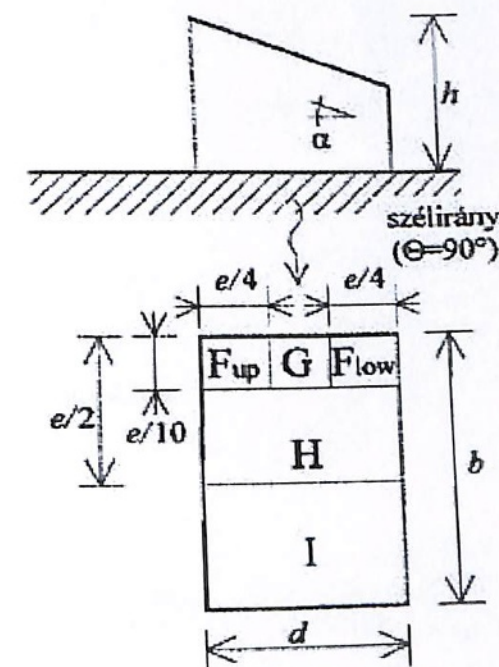
Hosszírányú $\theta = 90^\circ$

$$\frac{e}{4} = 2756 \quad mm$$

$$\frac{e}{10} = 1102.4 \quad mm$$

$$\frac{e}{2} = 5512 \quad mm$$

b) Szélirány a tetőéllel párhuzamosan



"H" zóna

$$\alpha := 7 \quad th := 2.212 \quad m$$

$$5^\circ \quad cpe10 := -0.6$$

$$15^\circ \quad cpe10' := -0.8$$

$$cpe1 := \left(\frac{(cpe10' - cpe10)}{15 - 5} \right) \cdot (\alpha - 5) + (cpe10) = -0.64$$

$$pweszHk' := th \cdot qpz \cdot cpe1 = -0.418 \quad \frac{kN}{m}$$

"I" zóna

$$\alpha := 7 \quad ti := 4.388 \quad m$$

$$5^\circ \quad cpe10 := -0.5$$

$$15^\circ \quad cpe10' := -0.7$$

$$cpe1 := \left(\frac{(cpe10' - cpe10)}{15 - 5} \right) \cdot (\alpha - 5) + (cpe10) = -0.54$$

$$pweszIk' := ti \cdot qpz \cdot cpe1 = -0.7 \quad \frac{kN}{m}$$

$$pweszIi := pweszHk' + pweszIk' = -1.119 \quad \frac{kN}{m}$$

II. Z szelemen méretezése

Z szelemen terhek

- hőteher

$$s1 = 3.728 \frac{kN}{m^2}$$

$$s2 = 3.16 \frac{kN}{m^2}$$

$$Ps := t \cdot s = 22.704 \frac{kN}{m}$$

$$P_{burk} := 0.236 \frac{kN}{m}$$

Teherkombináció ULS

$$P := Ps \cdot 1.5 + P_{burk} \cdot 1.35 = 34.375 \frac{kN}{m}$$

$$s := \frac{s1 + s2}{2} = 3.44 \frac{kN}{m^2}$$

$$s := 3.44 \frac{kN}{m^2}$$

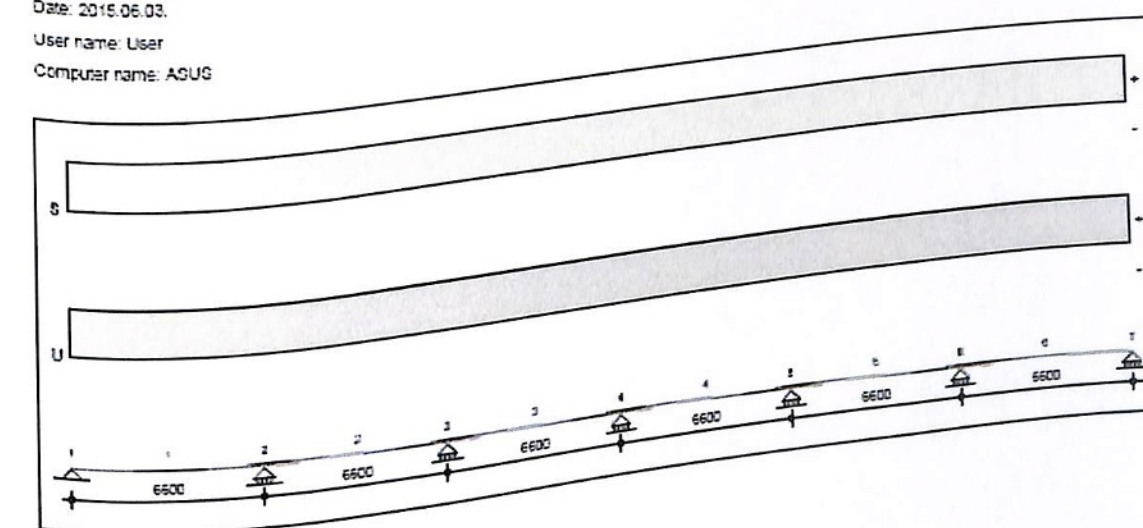
$$t := 0.60 \text{ m}$$

DIMroof 3.5 - F:\Diplomamunka\Erika\z szelemen suritett.dwg

Date: 2015.06.03.

User name: User

Computer name: ASUS



Alapbeállítások

Funkció: Z-gerenda

Ország: Magyarország

Szerkezeti beállítások

Szevény: Z 200

Statikai rendszer: Atrédes

Gyártó: Magyarország

Oldalsó támasz: Mindkét óv

Lemez: Szevény: LTP 46 Vastagság: 0.6

Csavar

Táv: Minden második völgyben

Tipus: Lemez: 4.6 Atrédes: 4.6 Támasz: 5.5

Lehajlás-számítás paraméterei

Lehajlási határ (nyílás): L / 150

Lehajlási határ (konzol): L / 150

Lehajlás típus: Maximális

Eredmények

ULS: 94%

SLS: 61%

Standard: Eurocode

Geometria

No.	Támaszok					Nyílások			Toldóelem
	Pozíció [mm]	Típus	Szélesség [mm]	L1 [mm]	L2 [mm]	Méret [mm]	Vastagság 1 [mm]	Vastagság 2 [mm]	
1	0	H	50	-	-	6600	2,0	-	-
2	6600	O	0	600	1200	6600	2,0	-	-
3	13200	O	0	600	1200	6600	2,0	-	-
4	19800	O	0	600	1200	6600	2,0	-	-
5	26400	O	0	600	1200	6600	2,0	-	-
6	33000	O	0	-	-	-	-	-	-
7	39600	H	0	-	-	-	-	-	-

Terhek

No.	Típus	Kezdőpont	Végpont	Kezdőint.	Végint.	Szélesség	ULS/SLS
							ULS
1	U			3,13			SLS
2	U			2,30			

III. TRAPÉZLEMEZ MÉRETEZÉS

Trapézlemez terhek

Állandó teher

$$g_{ktl} := 0.07 \quad t := 1.0 \text{ m} \quad \alpha := 7$$

$$P_{gk} := t \cdot g_{ktl} = 0.07 \quad \frac{kN}{m}$$

$$P_s := 3.44 \quad \frac{kN}{m}$$

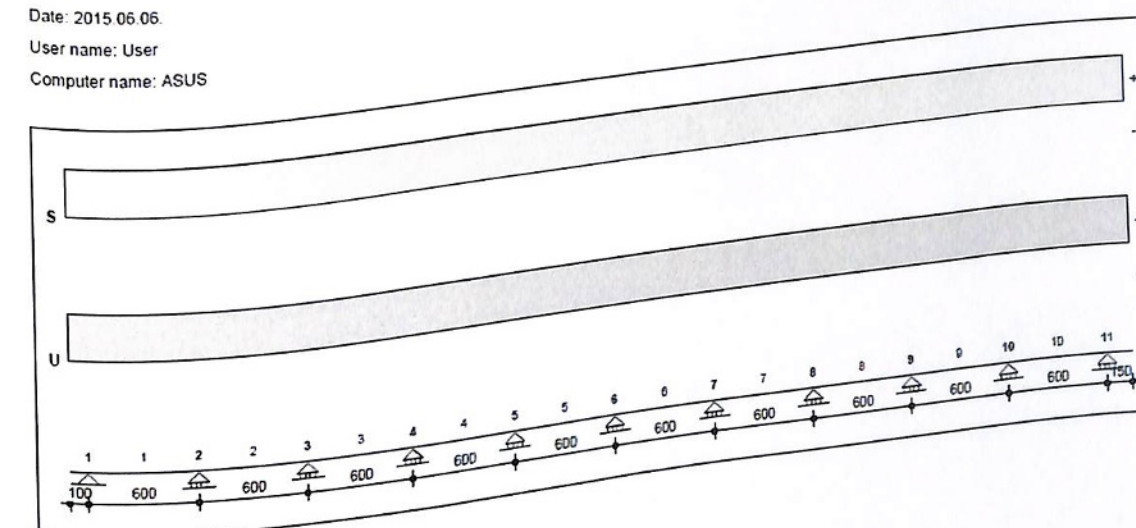
ULS önsúly+hó+szélzivás

$$P_{u1} := (1.35 \cdot P_{gk}) + (1.5 \cdot P_s) + (0.6 \cdot 1.5 \cdot p_{wszGk'}) = 3.289 \quad \frac{kN}{m}$$

SLS

$$P_{s2} := (P_{gk}) + (P_s) + (0.6 \cdot p_{wszGk'}) = 2.199 \quad \frac{kN}{m}$$

Date: 2015.06.06.
User name: User
Computer name: ASUS



Alapbeállítások

Funkció: Tető
Ország: Magyarország

Szerkezeti beállítások

Szelvény: LTP 45
Statikai rendszer: Folytatólagos
Elhelyezés: Szokásos (keskeny)
Átfedés: 1 hullám
Csavar

Táv:

Lehajlás-számítás paraméterei

Lehajlási határ (nyílás): L / 150
Lehajlási határ (konzol): L / 150
Lehajlás típus: Maximális

Eredmények

ULS: 55%
SLS: 6%

Szabvány: Eurocode

Geometria

No.	Támaszok					Nyílások			Csuklók
	Pozíció [mm]	Típus	Szélesség [mm]	L1 [mm]	L2 [mm]	Méret [mm]	Vastagság 1 [mm]	Vastagság 2 [mm]	Pozíció [mm]
1	0	C	100	100	-	600	0,4	-	
2	600	C	50	-	-	600	0,4	-	
3	1200	C	50	-	-	600	0,4	-	
4	1800	C	50	-	-	600	0,4	-	
5	2400	C	50	-	-	600	0,4	-	
6	3000	C	50	-	-	600	0,4	-	
7	3600	C	50	-	-	600	0,4	-	
8	4200	C	50	-	-	600	0,4	-	
9	4800	C	50	-	-	600	0,4	-	
10	5400	C	50	-	150	600	0,4	-	
11	6000	C	50	-	-				

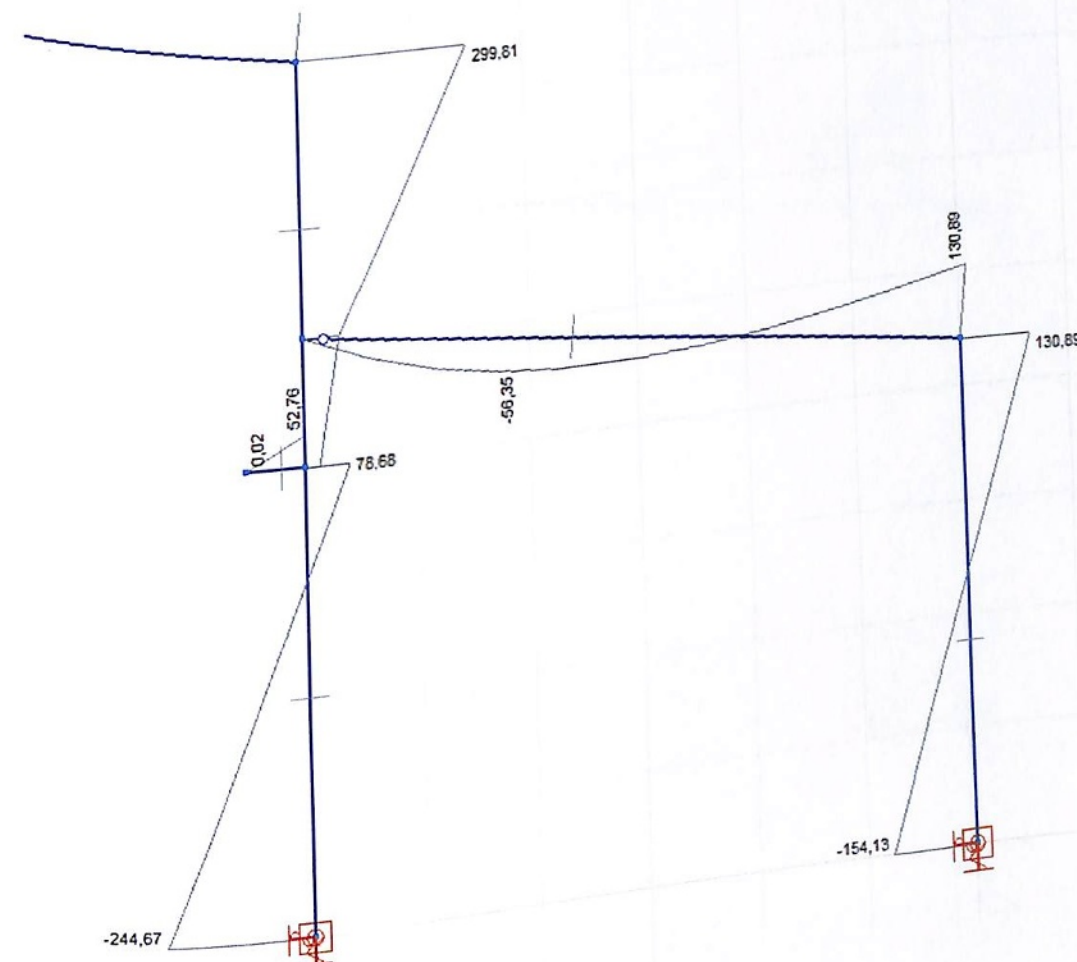
Terhek

No.	Típus	Kezdőpont	Végpont	Kezdőint.	Végint.	Szélesség	ULS/SLS
							ULS SLS
1	U			3,30			
2	U			2,20			

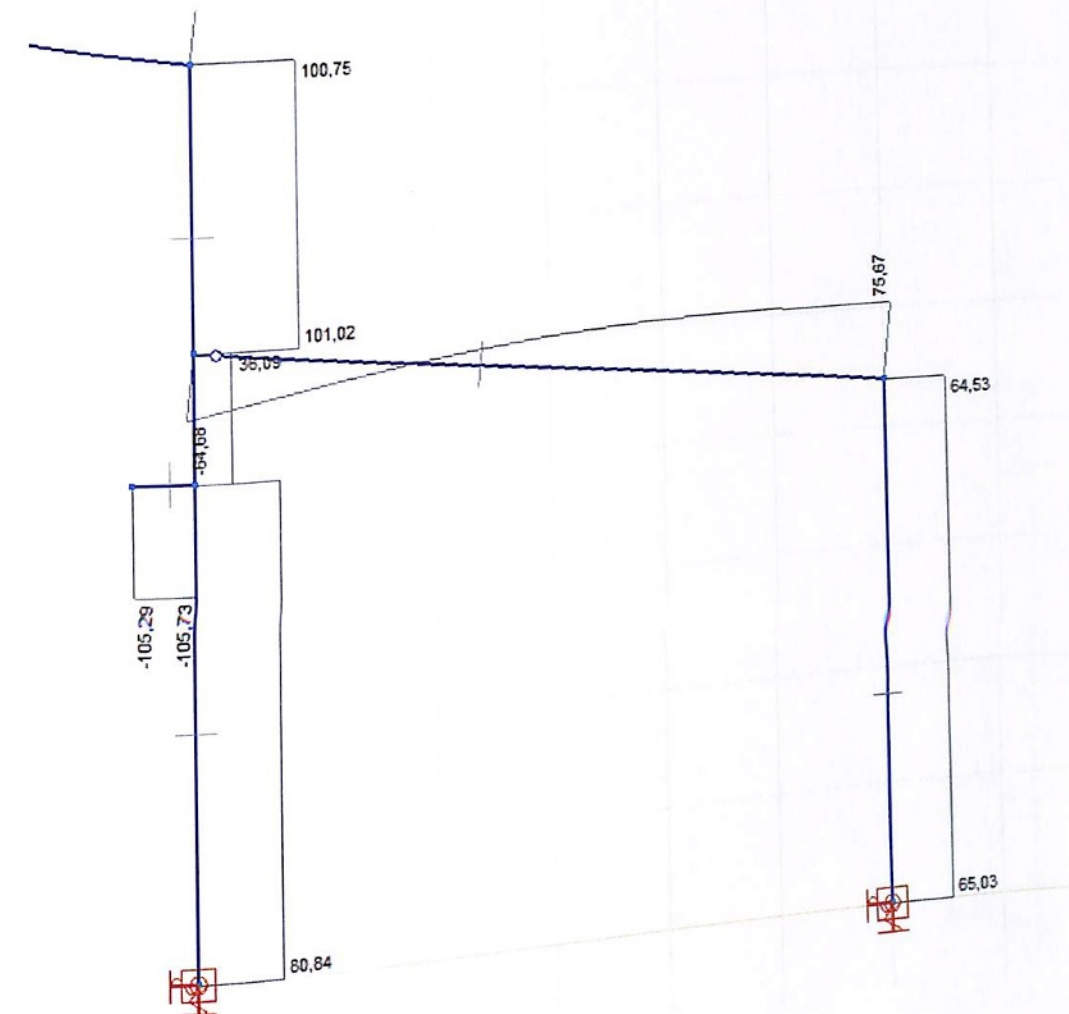
II. KERET VIZSGÁLATAI

Mértékadó igénybevételek

Med

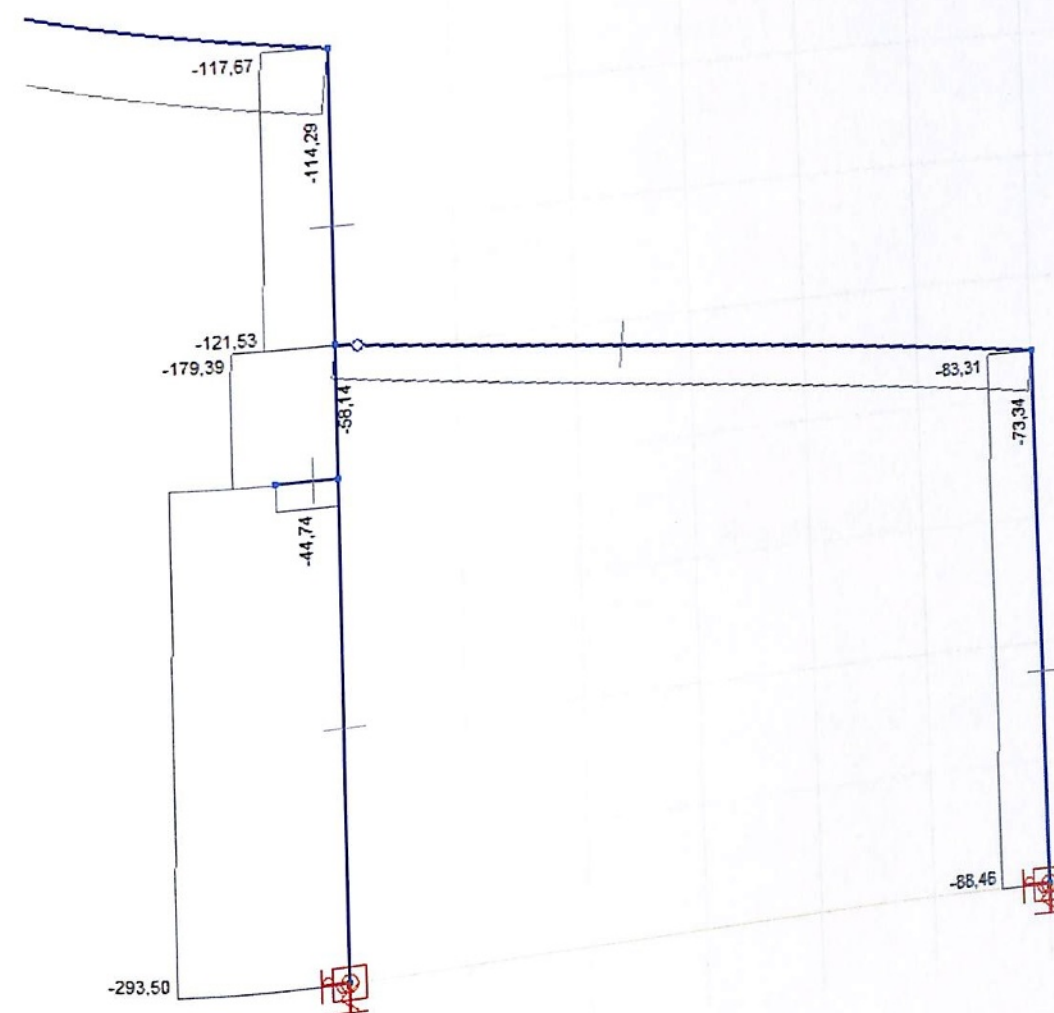


Ved



Acélszerkezetű csarnok méretezése

Ned



Keresztmetszet Ellenőrzések

Oszlop
HEA300

$I_{yo} := 18.2678$
 $i_{yo} := 127 \text{ mm}$
 $i_{zo} := 75 \text{ mm}$
 $A_{yo} := 11250$
 $A_{zo} := 112.5$
 $h_o := 290 \text{ mm}$
 $b_o := 300 \text{ mm}$

$\lambda_e := 93.9$
 $f_y := 235 \frac{N}{mm^2}$
 $\gamma_{m1} := 1.0$
 $\gamma_{mo} := 1.0$
 $t_{wo} := 8.5 \text{ mm}$
 $t_{fo} := 14 \text{ mm}$
 $r_o := 27 \text{ mm}$
 $W_{pl} := 1.383$
 $A_v := 3.728$

Ellenőrzés nyomásra

$$N_{crd} := \frac{A_{yo} \cdot f_y \cdot 10^{-3}}{\gamma_{mo}} = 2643.75 \text{ kN} \quad \square \geq \square \quad N_{ed} := 88.46 \text{ kN} \quad \text{Megfelel}$$

Ellenőrzés tiszta hajlítás

$$M_{crd} := \frac{W_{pl} \cdot f_y}{\gamma_{mo}} = 325.005 \text{ kNm} \quad M_{ed} := 154.13 \text{ kNm} \quad \text{Megfelel}$$

Ellenőrzés nyírásra

$$\frac{h_o}{t_{wo}} = 34.118 \quad \square \leq \square \quad \frac{72 \cdot \varepsilon}{\eta} = 60$$

Nyírási horpadást nem kell vizsgálni

$$V_{crd} := \frac{A_v \cdot f_y}{\sqrt{3} \cdot \gamma_{mo}} = 505.805 \text{ kN}$$

Hajítás +Nyírás kölcsönhatása

$$V_{ed} < \frac{V_{crd}}{2}$$

$V_{ed} := 64.53 \text{ kN}$ 1 pontban $\frac{V_{crd}}{2} = 252.903 \text{ kN}$ Megfelelnek
 $V_{ed} := 65.03 \text{ kN}$ 2 pontban

Normálerő hatása

$$N_{ed} < 0.25 \cdot N_{crd}$$

$$N_{ed} := 88.46 \quad kN$$

1 pontban

$$0.25 \cdot N_{crd} = 660.938 \quad kN \quad \text{Megfelelnek}$$

$$N_{ed} := 83.31 \quad kN$$

2 pontban

Nem kell figyelembe venni

Gerenda
IPE400

$$\lambda_e := 93.9$$

$$\gamma_{m1} := 1.0$$

$$f_y := 235 \quad \frac{N}{mm^2}$$

$$\gamma_{m0} := 1.0$$

$$I_{yg} := 23.134$$

$$i_{yg} := 165 \quad mm$$

$$i_{zg} := 39 \quad mm$$

$$A_{yg} := 8446 \quad mm^2$$

$$A_{zg} := 8446 \quad mm^2$$

$$h_g := 400 \quad mm$$

$$b_g := 180 \quad mm$$

$$t_{wg} := 9 \quad mm$$

$$t_{fg} := 13 \quad mm$$

$$r_g := 21 \quad mm$$

$$W_{pl} := 1.307$$

$$A_v := 4.269$$

Ellenőrzés nyomásra

$$N_{crd} := \frac{A_{yg} \cdot f_y \cdot 10^{-3}}{\gamma_{m0}} = 1984.81 kN$$

$$\sigma \geq \sigma$$

$$N_{ed} := 83.31 \quad kN \quad \text{Megfelel}$$

Ellenőrzés tiszta hajlítás

$$M_{crd} := \frac{W_{pl} \cdot f_y}{\gamma_{m0}} = 307.145 \quad kN$$

$$\sigma \geq \sigma$$

$$M_{ed} := 130.89 \quad kNm \quad \text{Megfelel}$$

Ellenőrzés nyírásra

$$\frac{h_g}{t_{wg}} = 44.444 \quad \sigma \leq \sigma \quad \frac{72 \cdot \varepsilon}{\eta} = 60$$

$$\varepsilon := 1 \quad \eta := 1.2$$

Nyírási horpadást nem kell vizsgálni

$$V_{crd} := \frac{A_v \cdot f_y}{\sqrt{3} \cdot \gamma_{m0}} = 579.206 \quad kN$$

Acélszerkezetű csarnok méretezése

Hajítás + Nyírás kölcsönhatása

$$V_{ed} < \frac{V_{crd}}{2}$$

$$V_{ed} := 75.67 \quad kN$$

1 pontban

$$\frac{V_{crd}}{2} = 289.603 \quad kN$$

Megfelelnek

$$V_{ed} := 64.68 \quad kN$$

2 pontban

Normálerő hatása

$$N_{ed} < 0.25 \cdot N_{crd}$$

$$N_{ed} := 83.31 \quad kN$$

1 pontban

$$0.25 \cdot N_{crd} = 496.203 \quad kN$$

Megfelelnek

$$N_{ed} := 58.14 \quad kN$$

2 pontban

Nem kell figyelembe venni

Hajítás + Nyírás kölcsönhatása

$$V_{ed} < \frac{V_{crd}}{2}$$

$V_{ed} := 75.67$	kN	1 pontban	$\frac{V_{crd}}{2} = 289.603$	kN	Megfelelnek
$V_{ed} := 64.68$	kN	2 pontban			

Normálerő hatása

$$N_{ed} < 0.25 \cdot N_{crd}$$

$N_{ed} := 83.31$	kN	1 pontban	$0.25 \cdot N_{crd} = 496.203$	kN	Megfelelnek
$N_{ed} := 58.14$	kN	2 pontban			Nem kell figyelembe venni

Mellékhajó oszlop stabilitás vizsgálat

Anyagminőség és jellemzői S235

L := 6000 mm H := 4400 mm

$$\lambda_e := 93.9$$

$$\gamma_{m1} := 1.0$$

$$f_y := 235 \frac{N}{mm^2}$$

Oszlop
HEA300

$$I_{yo} := 18.2678$$

$$i_{yo} := 127 \text{ mm}$$

$$i_{zo} := 75 \text{ mm}$$

$$\Lambda_{yo} := 11250$$

$$\Lambda_{zo} := 11250$$

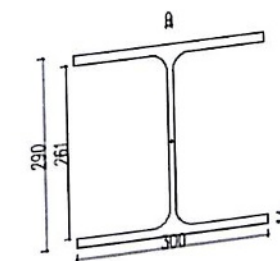
$$h_o := 290 \text{ mm}$$

$$b_o := 300 \text{ mm}$$

$$t_{wo} := 9 \text{ mm}$$

$$t_{fo} := 14 \text{ mm}$$

$$r_o := 27 \text{ mm}$$



Gerenda
IPE400

$$I_{yg} := 23.134$$

$$i_{yg} := 165 \text{ mm}$$

$$i_{zg} := 39 \text{ mm}$$

$$A_{yg} := 8446$$

$$A_{zg} := 8446$$

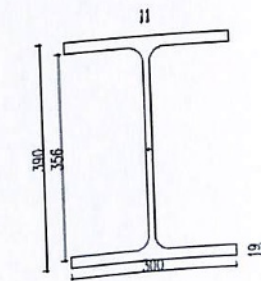
$$h_g := 400 \text{ mm}$$

$$b_g := 180 \text{ mm}$$

$$t_{wg} := 9 \text{ mm}$$

$$t_{fg} := 13 \text{ mm}$$

$$r_g := 21 \text{ mm}$$



Oszlop vizsgálat

Kihajlás vizsgálat keret síkjában "y"

$$c := \frac{I_{yo} \cdot L}{I_{yg} \cdot H} = 1.077 \text{ mm} \quad c \leq 10 \text{ Megfelel}$$

$$\nu_y := \sqrt{1 + 0.35 \cdot c - 0.017 \cdot c^2} = 1.165$$

$$\lambda_y := \frac{\nu_y \cdot H}{i_{yo}} = 40.361$$

$$\lambda := \frac{\lambda_y}{\lambda_e} = 0.43$$

"a" görbe alapján

$$\lambda_1 := 0.4 \quad a_1 := 0.9528$$

$$\lambda_2 := 0.5 \quad a_2 := 0.9243$$

$$\chi := \left(\frac{(a_2 - a_1) \cdot (\lambda - \lambda_1)}{(\lambda_2 - \lambda_1)} \right) + a_1 = 0.9443$$

$$N_{brdy} := \frac{\chi \cdot A_{yo} \cdot f_y \cdot 10^{-3}}{\gamma_{m1}} = 2496.486 \text{ kN}$$

$\geq$

$$N_{ed} = 58.14 \text{ kN} \text{ Megfelel}$$

Kihajlás vizsgálat keret síkjában "z"

$$v_z := 1 \quad l := 1600 \quad mm$$

$$\lambda_y := \frac{v_z \cdot l}{i_{zo}} = 21.333 \quad \lambda := \frac{\lambda_y}{\lambda_e} = 0.227$$

"b" görbe alapján

$$\lambda_1 := 0.2 \quad a_1 := 1$$

$$\lambda_2 := 0.3 \quad a_2 := 0.9641$$

$$\chi := \left(\frac{(a_2 - a_1)}{(\lambda_2 - \lambda_1)} \cdot (\lambda - \lambda_1) \right) + a_1 = 0.9902$$

$$N_{brdz} := \frac{\chi \cdot A_{zo} \cdot f_y \cdot 10^{-3}}{\gamma_{m1}} = 2617.942 \, kN$$

Gerenda vizsgálat

Kihajlás vizsgálat keret síkjában "y"

$$v_y := 1 \quad l := 8062 \quad mm$$

"a" görbe alapján

$$\lambda_y := \frac{v_y \cdot l}{i_{yg}} = 48.861 \quad \lambda := \frac{\lambda_y}{\lambda_e} = 0.52$$

$$\lambda_1 := 0.5 \quad a_1 := 0.9243$$

$$\lambda_2 := 0.6 \quad a_2 := 0.8900$$

$$\chi := \left(\frac{(a_2 - a_1)}{(\lambda_2 - \lambda_1)} \cdot (\lambda - \lambda_1) \right) + a_1 = 0.9173$$

$$N_{brdy} := \frac{\chi \cdot A_{yg} \cdot f_y \cdot 10^{-3}}{\gamma_{m1}} = 1820.708 \, kN$$

$$\geq \quad N_{ed} = 83.5 \quad kN$$

Megfelel

Kihajlás vizsgálat keret síkjában "z"

$$\nu_z := 1 \quad l := 1600 \quad mm$$

$$\lambda_y := \frac{\nu_z \cdot l}{i_{zg}} = 41.026$$

$$\lambda := \frac{\lambda_y}{\lambda_e} = 0.437$$

"b" görbe alapján

$$\begin{aligned} \lambda_1 &:= 0.4 & a_1 &:= 0.9261 \\ \lambda_2 &:= 0.5 & a_2 &:= 0.8842 \end{aligned}$$

$$\chi := \left(\frac{a_2 - a_1}{\lambda_2 - \lambda_1} \right) \cdot (\lambda - \lambda_1) + a_1 = 0.9106$$

$$N_{brdz} := \frac{\chi \cdot A_{zg} \cdot f_y \cdot 10^{-3}}{\gamma_{m1}} = 1807.439 \text{ kN}$$

$\geq$

$$N_{ed} = 85.13 \quad kN$$

Megfelel

Számítási melléklet III.

Acélszerkezetű darupálya méretezése

Tartalom:

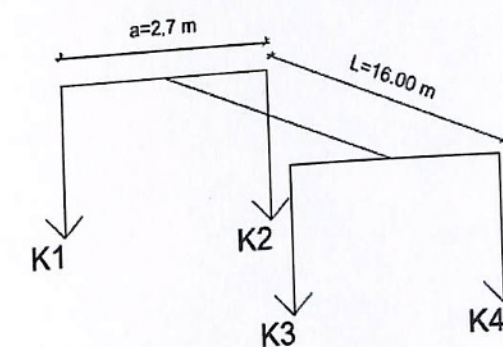
- I. Terhek meghatározása
- II. Darupályatartó igénybevételei
- III. Darupályatartó méretezése
- IV. Darupályatartó ellenőrzés

Készítette: Nagy Erika
SZIE-YMÉK, levelezőtagozat
Neptun kód: [REDACTED]
Konzulens: Horváth Katalin

ACÉLSZERKEZTŰ DARUPÁLYA MÉRETEZÉSE

$$\begin{aligned} K1 &:= 77 \text{ kN} \\ K2 &:= 60 \text{ kN} \\ K3 &:= 35 \text{ kN} \\ K4 &:= 29 \text{ kN} \end{aligned}$$

$$\begin{aligned} a &:= 2.7 \text{ m} \\ L &:= 16 \text{ m} \\ k &:= 6.6 \text{ m} \end{aligned}$$



Maximális keréknyomás

$$Q_{rmax_} := 77 \text{ kN}$$

$$\Sigma Q_{rmax_} := 2 \cdot Q_{rmax_} = 154 \text{ kN}$$

$$Q_{rmax} := 35 \text{ kN}$$

$$\Sigma Q_{rmax} := 2 \cdot Q_{rmax} = 70 \text{ kN}$$

Emelt teher

$$Q_h := 50 \text{ kN}$$

I. Darupálya tartó terhek meghatározása

Daru önsúlya

$$G_{daru} := (K1 + K2 + K3 + K4) - Q_h = 151 \text{ kN}$$

$$\lambda := \frac{G_{daru}}{\Sigma Q_{rmax_} + \Sigma Q_{rmax}} = 0.674$$

Minimális keréknyomás

$$Q_{rmin_} := \lambda \cdot Q_{rmax_} = 51.906 \text{ kN}$$

$$\Sigma Q_{rmin_} := 2 \cdot Q_{rmin_} = 103.813 \text{ kN}$$

$$Q_{rmin} := \lambda \cdot Q_{rmax} = 23.594 \text{ kN}$$

$$\Sigma Q_{rmin} := 2 \cdot Q_{rmin} = 47.188 \text{ kN}$$

Meghajtási erő

suróási tényező

$$\mu := 0.2$$

$$\Sigma K := \mu \cdot \Sigma Q_{rmax_} = 30.8 \text{ kN}$$

Hosszirányú fékező erő

$$nr := 2$$

$$Hl := \frac{\Sigma K}{nr} = 15.4 \quad kN$$

Keresztirányú fékező erő

$$\xi 1 := \frac{\Sigma Q r_{max_}}{\Sigma Q r_{max_} + \Sigma Q r_{max}} = 0.688$$

$$\xi 2 := 1 - \xi 1 = 0.313$$

$$Ht1 := \xi 1 \cdot \Sigma K \cdot L \cdot \frac{\xi 1 - 0.5}{a} = 23.528 \quad kN$$

$$Ht2 := \xi 2 \cdot \Sigma K \cdot L \cdot \frac{\xi 1 - 0.5}{a} = 10.694 \quad kN$$

Oldallökő erő

$$nr := 2$$

$$G_{kocsi} := 15$$

$$\eta_{wc} := 0.5$$

$$Ht3 := \frac{\mu \cdot \eta_{wc} \cdot G_{kocsi}}{nr} = 0.75 \quad kN$$

FERDÉN FUTÁSBÓL származó erő (befeszülési erő)

$$\alpha := 0.015$$

$$f := 0.3 \cdot (1 - e^{-250 \cdot \alpha}) = 0.293 \quad f \leq 0.3$$

$$s := f \cdot \left(1 - \frac{a}{2a}\right) \cdot \Sigma Q r_{max_} = 22.557 \quad kN$$

Acélszerkezetű csarnok méretezése

Első kerekek oldalirányú erői

$$Hs1t := \frac{\xi_1}{nr} \cdot f \cdot \Sigma Q_{rmax_} = 15.508 \quad kN$$

$$Hs2t := \frac{\xi_2}{nr} \cdot f \cdot \Sigma Q_{rmax_} = 7.049 \quad kN$$

Dinamikus tényezők

$$\varphi_1 := 1.1 \quad v := 8.6 \frac{m}{perc} \quad \varphi_{2min} := 1.1 \quad \beta_2 := 0.34$$

$$vh := \frac{v}{60} = 0.143$$

$$\varphi_2 := \varphi_{2min} + \beta_2 \cdot vh = 1.149$$

$$\varphi_4 := 1$$

$$\varphi_5 := 1.5$$

Teherkombináció

Daru önsúlya

$$QC := G_{daru} \cdot \varphi_1 = 166.1 \quad kN$$

Emelt teher

$$QH := Q_h \cdot \varphi_2 = 57.437 \quad kN$$

Hosszirányú fékező erő

$$HL := H_l \cdot \varphi_5 = 23.1 \quad kN$$

Keresztirányú fékező erő

$$HT := H_{t1} \cdot \varphi_5 = 35.292 \quad kN$$

II. Darupálya tartó igénybevételei

Mértékadó erők $\gamma_g := 1.35$

$$Ged := (QC + QH) \cdot \gamma_g = 301.775 \quad kN$$

$$Kössz := K1 + K2 + K3 + K4 = 201 \quad kN$$

$$K1 := \frac{K1}{Kössz} \cdot 100 = 38.308 \quad \%$$

$$K2 := \frac{K2}{Kössz} \cdot 100 = 29.851 \quad \%$$

$$K3 := \frac{K3}{Kössz} \cdot 100 = 17.413 \quad \%$$

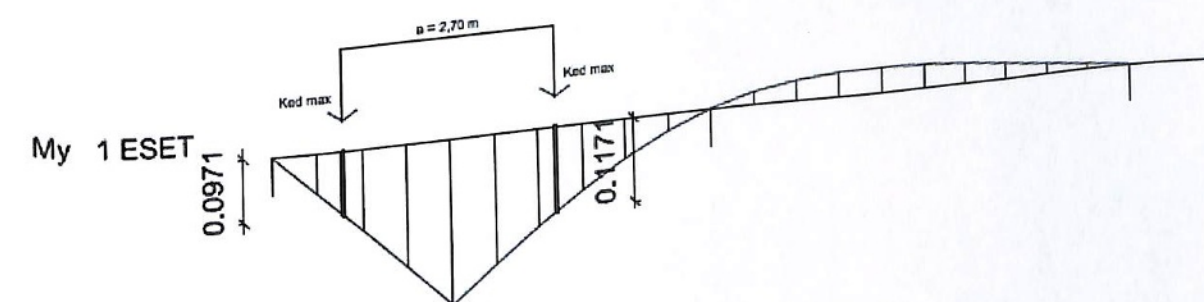
$$K4 := \frac{K4}{Kössz} \cdot 100 = 14.428 \quad \%$$

$$Kedmax := K1 \cdot \frac{Ged}{100} = 115.605 \quad kN$$

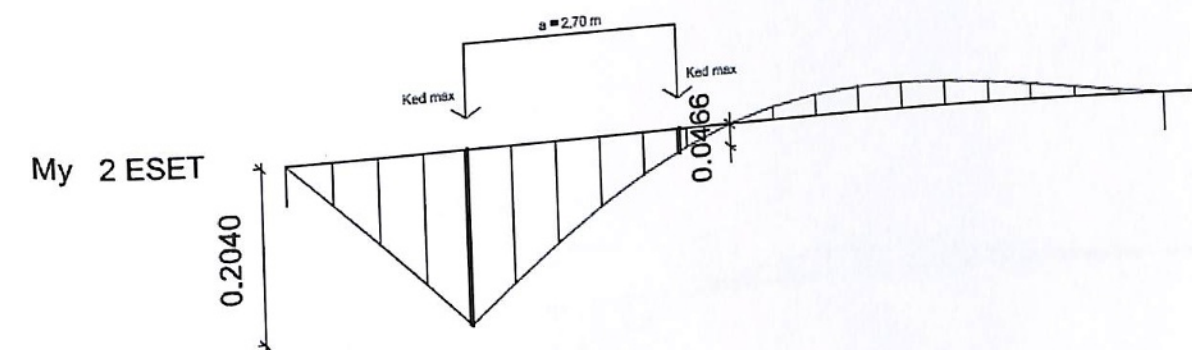
$$Kedmin := K3 \cdot \frac{Ged}{100} = 52.548 \quad kN$$

Függőleges erők minimum

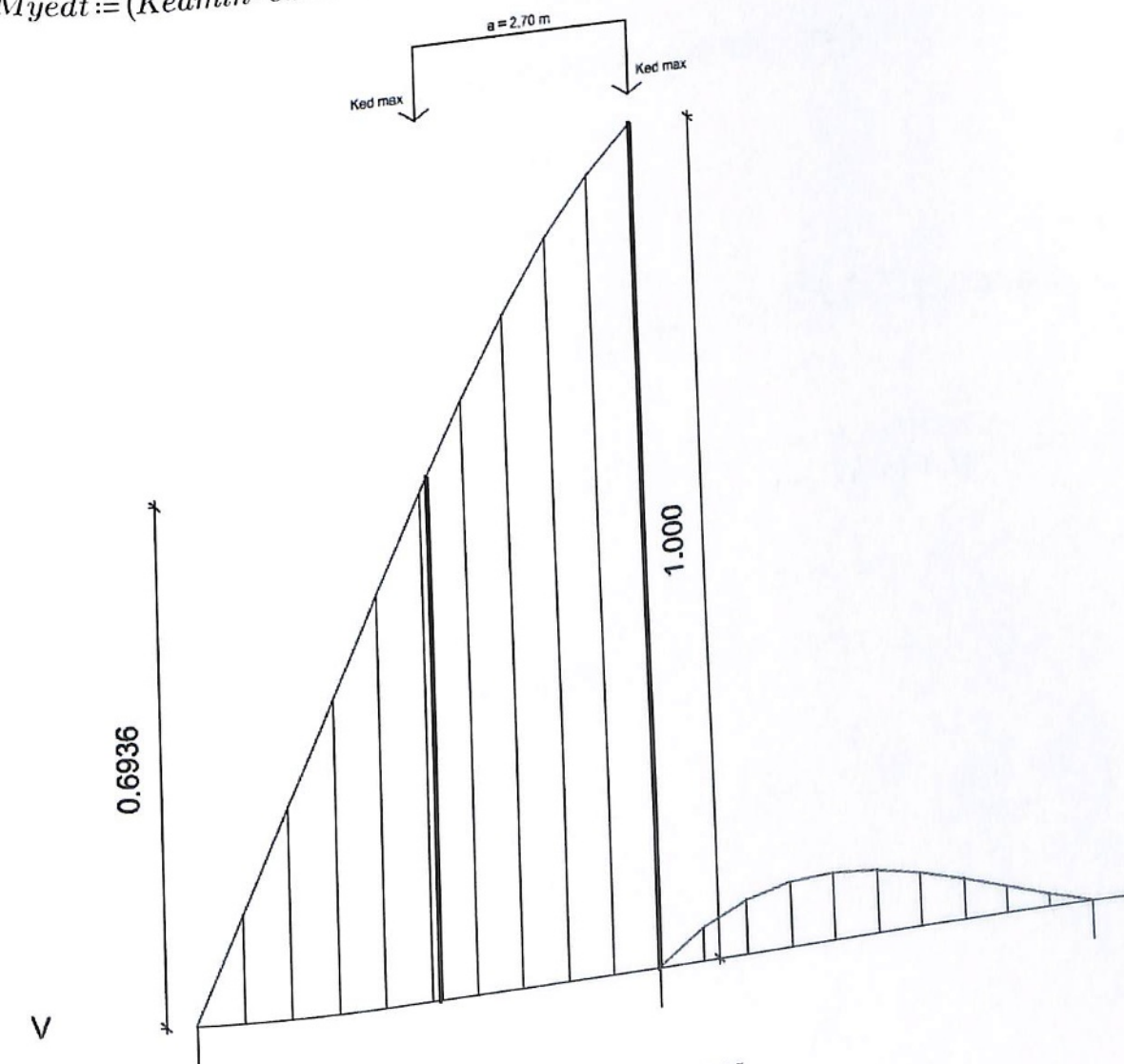
$$k := 6.6$$



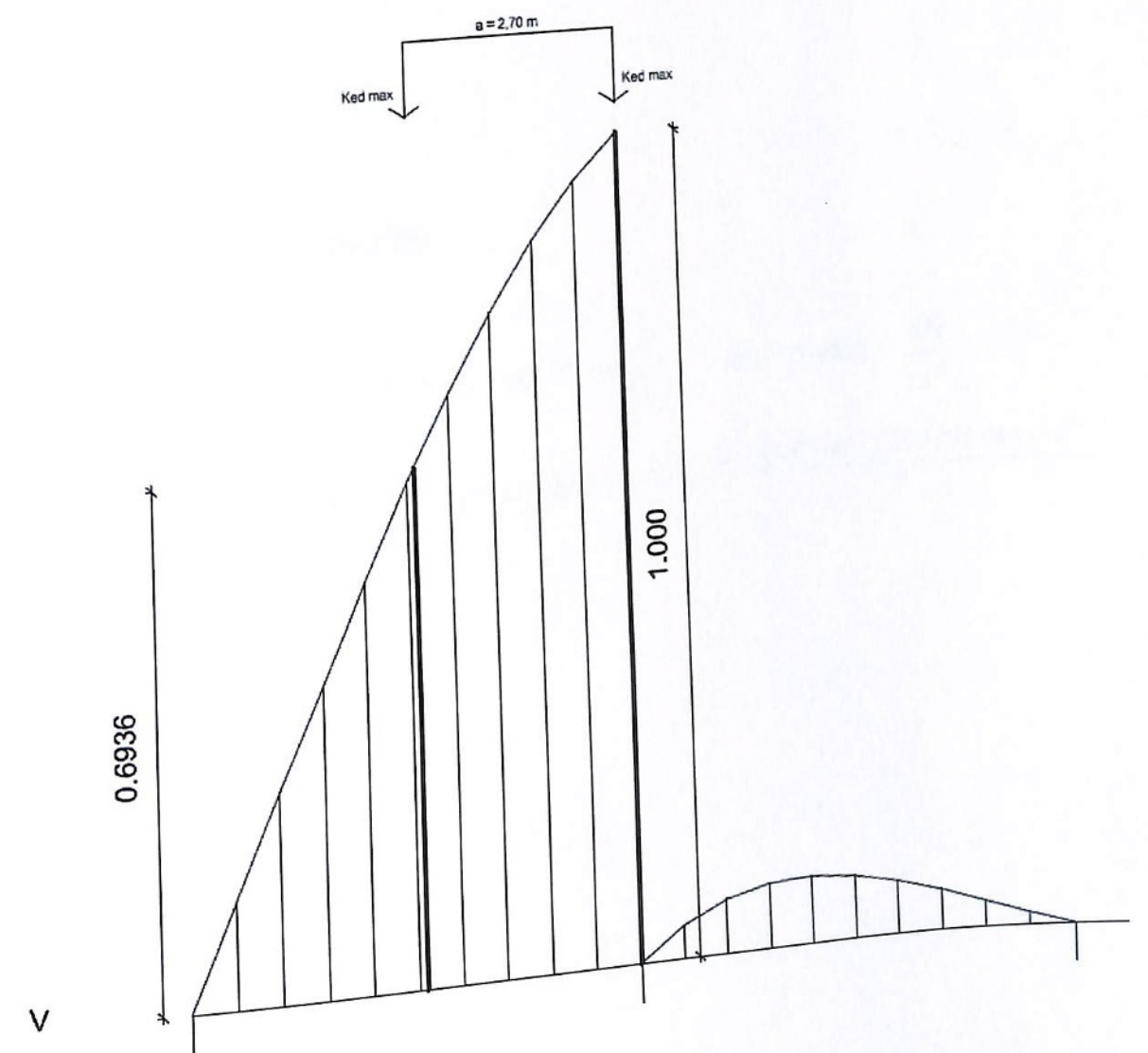
$$Myed := (Kedmin \cdot 0.0971 + Kedmin \cdot 0.1171) \cdot k = 74.288 \quad kNm$$



$$My_{edt} := (Ked_{min} \cdot 0.204 - Ked_{min} \cdot 0.0466) \cdot k = 54.589 \text{ kNm}$$



$$V_{zed} := Ked_{min} \cdot 1 + Ked_{min} \cdot 0.6936 = 88.995 \text{ kN}$$



$$V_{zed} := Ked_{max} \cdot 1 + Ked_{max} \cdot 0,6936 = 195,789 \text{ kN}$$

Vízszintes erők - oldalirányú megtámasztáshoz

$$Fed_{xmax} := Ht1 \cdot \varphi5 = 35,292 \text{ kN}$$

$$Fed_{xmin} := Ht2 \cdot \varphi5 = 16,042 \text{ kN}$$

$$M_{zed1} := (Fed_{xmax} \cdot 0,0971 + Fed_{xmax} \cdot 0,1171) \cdot k = 49,893 \text{ kNm}$$

$$M_{zed2} := (Fed_{xmax} \cdot 0,204 + Fed_{xmax} \cdot 0,0466) \cdot k = 58,371 \text{ kNm}$$

$$V_{yed} := Fed_{xmax} \cdot 1 + Fed_{xmax} \cdot 0,6936 = 59,77 \text{ kN}$$

Acélszerkezetű csarnok méretezése

Normál erő

$$Nx := HL \cdot \varphi 5 = 34.65 \quad kN$$

III. Darupálya tartó méretezése

Lehajlás

$$k := 6.6 \quad m$$

$$l := k \cdot 1000 = 6600 \quad mm$$

$$E := 210000 \quad \frac{kN}{m^2}$$

$$ez := \frac{l^3}{E \cdot I} \cdot (Kedmax \cdot 0.01093 + Kedmax \cdot 0.002364) \rightarrow \frac{2.1039983413297792783 \cdot 10^6}{I}$$

$$ezmax := \frac{l}{600} = 11 \quad mm$$

$$I := \frac{2.10399 \cdot 10^6}{ezmax} = 1.913 \cdot 10^5 \quad mm^4$$

Alkalmazott gerenda

HEB320

$$Ialk := 3.082 \cdot 10^8 \quad mm^4$$

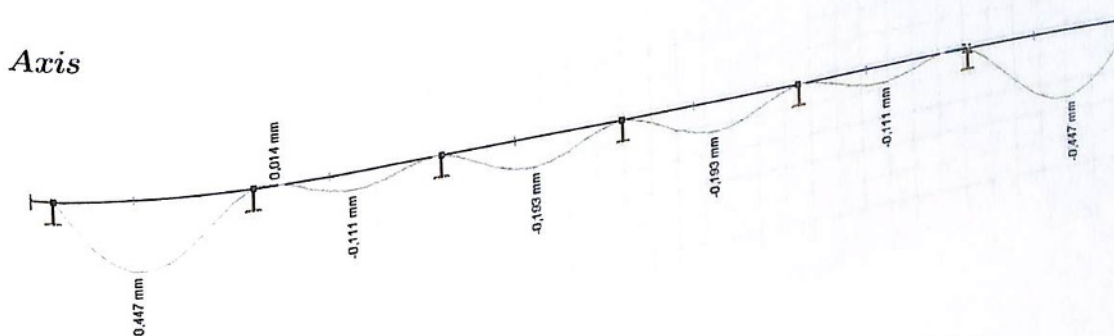
$$ezalk := \frac{l^3}{E \cdot Ialk} \cdot (Kedmax \cdot 0.01093 + Kedmax \cdot 0.002364) \cdot 1000 = 6.827 \quad mm$$

$$ezalk \leq ezmax$$

$$6.827 < 11$$

Lehajlásra megfelel

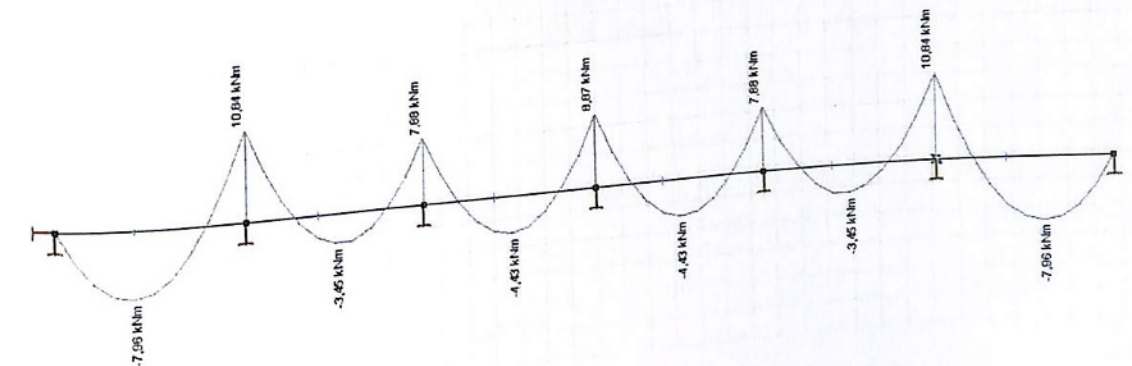
Axis



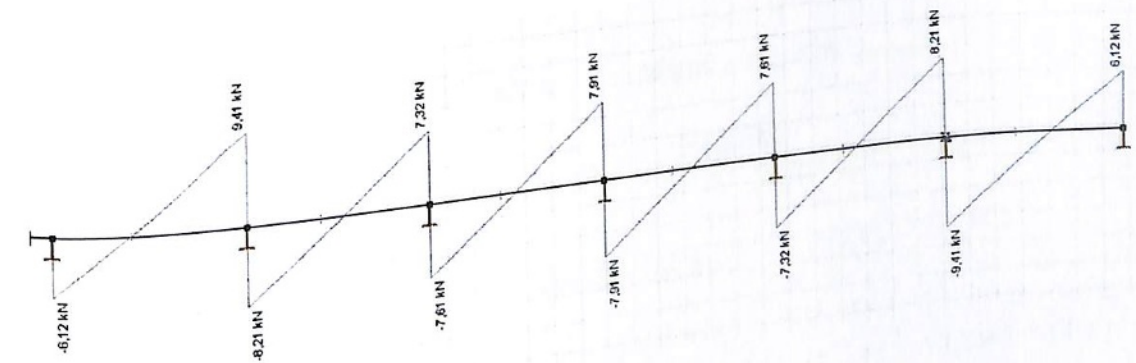
Axisból

Medmező := 7.96 kNm

Medtámasz := 10.8 kN



Ved. := 9.41 kN



Határállapotok

ULS

$Myed_m := Myed_1 + Med_{mező} = 171.393 \text{ kNm}$

$Myed_t := Myed_t + Med_{támasz} = 65.389 \text{ kNm}$

$Vzed := Vzed. + Ved. = 205.199 \text{ kN}$

II. Darupálya tartó ellenőrzése

HEB320 adatai $S235$ $f_y := 235 \frac{N}{mm^2}$

$$\begin{aligned} A &:= 16130 \text{ mm}^2 & b_f &:= 300 \text{ mm} \\ I_y &:= 30820 \cdot 10^4 \text{ mm}^4 & t_f &:= 20.5 \text{ mm} \\ I_z &:= 9239 \cdot 10^4 \text{ mm}^4 & h_w &:= 279 \text{ mm} \\ W_{pl.y} &:= 2149 \cdot 10^3 \text{ mm}^3 & t_w &:= 11.5 \text{ mm} \end{aligned}$$

$$W_{pl.z.öv} := 2 \cdot \left(\frac{b_f}{2} \cdot t_f \cdot \frac{b_f}{2} \right) = 461250 \text{ mm}^3$$

$$A_{vz} := 5177 \text{ mm}^2 \quad \gamma_{m0} := 1$$

Tiszta hajlítás "y" tengely körül $M_{yedm} = 171.393 \text{ kNm}$

$$M_{yrd} := \frac{\left(\frac{W_{pl.y} \cdot f_y}{\gamma_{m0}} \right)}{1000000} = 505.015 \text{ kNm} \quad M_{yrd} \geq M_{yedm}$$

Megfelel

Tiszta hajlítás "z" tengely körül $M_{zed2} = 58.371 \text{ kNm}$

$$M_{zrd} := \frac{\left(\frac{W_{pl.z.öv} \cdot f_y}{\gamma_{m0}} \right)}{1000000} = 108.394 \text{ kNm} \quad M_{zrd} \geq M_{zed2}$$

Megfelel

Tiszta nyomás $Nx = 34.65 \text{ kNm}$

$$N_{xrd} := \frac{\left(\frac{A \cdot f_y}{\gamma_{m0}} \right)}{1000} = 3790.55 \text{ kN}$$

$N_{xrd} \geq Nx$
Megfelel

Normálerő hatásának figyelembe vételének feltételei

$Nx = 34.65 \text{ kN}$ $0.25 \cdot N_{xrd} = 947.638 \text{ kN}$

Nem vesszük figyelembe

$$\frac{\frac{(0.5 \cdot hw \cdot tw \cdot f_y)}{\gamma_{m0}}}{1000} = 376.999 \text{ kN}$$

$Nx = 34.65 \text{ kN}$

Nem vesszük figyelembe

Stabilitás vizsgálatok

Kifordulás

$\lambda_e := 93.9$
 $bf := 300 \text{ mm}$
 $tf := 20.5 \text{ mm}$
 $hw := 279 \text{ mm}$
 $tw := 11.5 \text{ mm}$

$kc := 0.9$
 $Lc := 6600 \text{ mm}$

$$I_z := \left(\frac{tw^3 \cdot hw}{12} \right) + \left(\frac{bf^3 \cdot tf}{12} \right) = 46.131 \cdot 10^6 \text{ mm}^4$$

$$Afz := bf \cdot tf + \frac{hw}{6} = 6196.5 \text{ mm}^2$$

$$ifz := \sqrt{\frac{I_z}{Afz}} = 86.282 \text{ mm}$$

$$\lambda_f := \frac{kc \cdot Lc}{ifz \cdot \lambda_e} = 0.733$$

$\lambda_{co} := 0.5$

Beroppanás vizsgálat

$$\gamma_{m1} := 1$$

$$a := 6600 \text{ mm}$$

$$S_s := 100 \text{ mm}$$

$$\text{Feltételezzük hogy } \lambda_f > 0.5$$

$$m1 := \frac{bf}{tw} = 26.087$$

$$m2 := 0.02 \cdot \left(\frac{hw}{tf} \right)^2 = 3.705$$

$$ly := S_s + 2 \cdot tf \cdot (1 + \sqrt{m1 + m2}) = 364.784 \text{ mm}$$

$$kf := 6 + 2 \cdot \left(\frac{hw}{a} \right) = 6.085$$

$$F_{cr} := \frac{\left(0.9 \cdot kf \cdot E \cdot \frac{tw^3}{hw} \right)}{1000} = 6268.726 \text{ kN}$$

$$\lambda_f := \sqrt{\frac{ly \cdot tw \cdot fy}{F_{cr} \cdot 1000}} = 0.397$$

Nem jól feltételeztük

$$m2 := 0$$

$$ly := S_s + 2 \cdot tf \cdot (1 + \sqrt{m1 + m2}) = 350.409 \text{ mm}$$

$$\lambda_f := \sqrt{\frac{ly \cdot tw \cdot fy}{F_{cr} \cdot 1000}} = 0.389$$

Tehát $\lambda_f = 0.5$

$$\chi_f := \frac{0.5}{\lambda_f} = 1$$

$$L_{eff} := \chi_f \cdot ly = 350.409 \text{ mm}$$

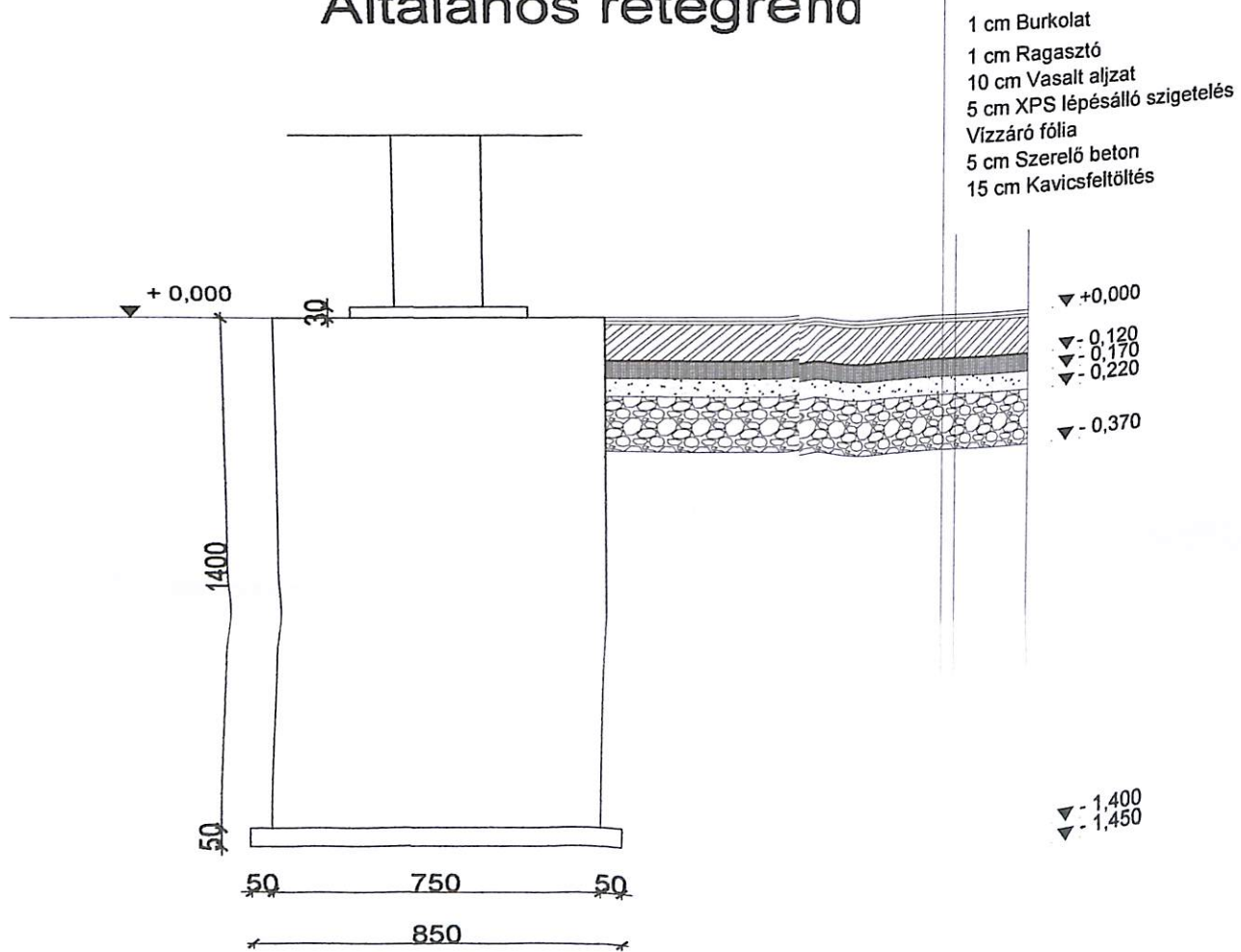
$$F_{rd} := \frac{\left(\frac{fy \cdot L_{eff} \cdot tw}{\gamma_{m1}} \right)}{1000} = 946.981 \text{ kN}$$

$$K_{edmax} = 115.605 \text{ kNm}$$

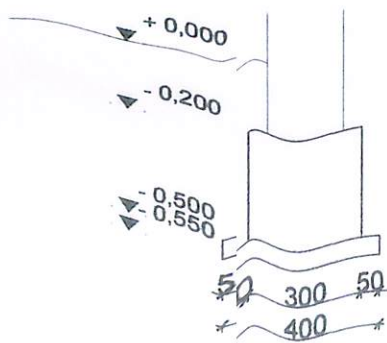
$$F_{rd} \geq K_{edmax}$$

Megfelel

Általános rétegrend



Talpgerenda
készül 116,8 fm

[illegible]

1,80	2,10	12	80	12
1,75	1,90	30	20	28

Megjegyzések:

+0,00 csarnok padlóvonalának síkja

Kapcsolódó tervek:

- A-1 Általános terv 1
- A-2 Általános terv 2
- A-3 Részletes keret terv
- A-4 Lindab terv
- A-5 Merevítés

Betonminőség: C20/25
Acélminőség: S235 JRG2

A betonozás csak műszaki ellenőr építési naplóba írt engedélyével szabad megkezdeni!

Munkavégzés során a vonatkozó balesetvédelmi, biztonságtechnikai, munkavédelmi és tűzrendészeti előírásokat be kell tartani!

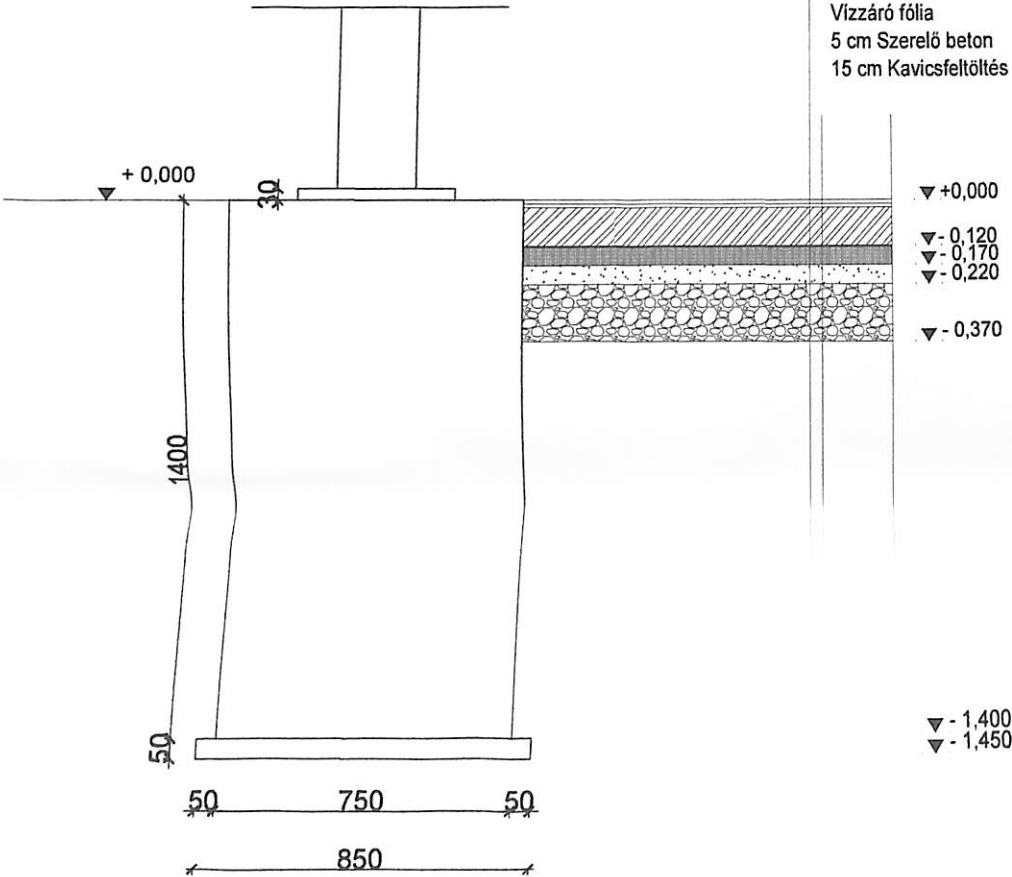
A terv a műszaki leírással együtt érvényes.

Vasalási tervek külön készültek. Nem tartóztak a feladatkiírásba.

Szent István Egyetem Ybl Miklós Építéstudományi Kar Mechanika és Tartószerkezeti Tanszék		
Tárgy: Szakirányú diplomamunka - Magasépítési Szakirány		Rejz. szám: A-6
Projekt: KÉTHAJÓS DARUZOTT IPARI CSARNOK tervezése		Méretarányok: M 1:100 M 1:50
Terv: ALAPOZÁS		Dátum: 2015.06.05
Névképzés/tagozat Nagy Erika Bsc IV. Levelező építőmérnök	Neptun kód: 	Konzulens Horváth Katalin

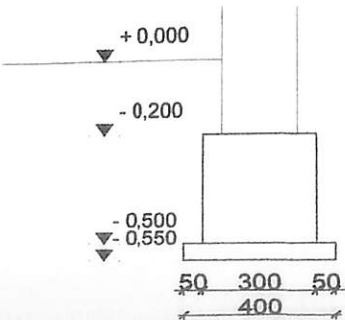
Általános rétegrend

- 1 cm Burkolat
- 1 cm Ragasztó
- 10 cm Vasalt aljzat
- 5 cm XPS lépésálló szigetelés
- Vízzáró fólia
- 5 cm Szerelő beton
- 15 cm Kavicsfeltöltés

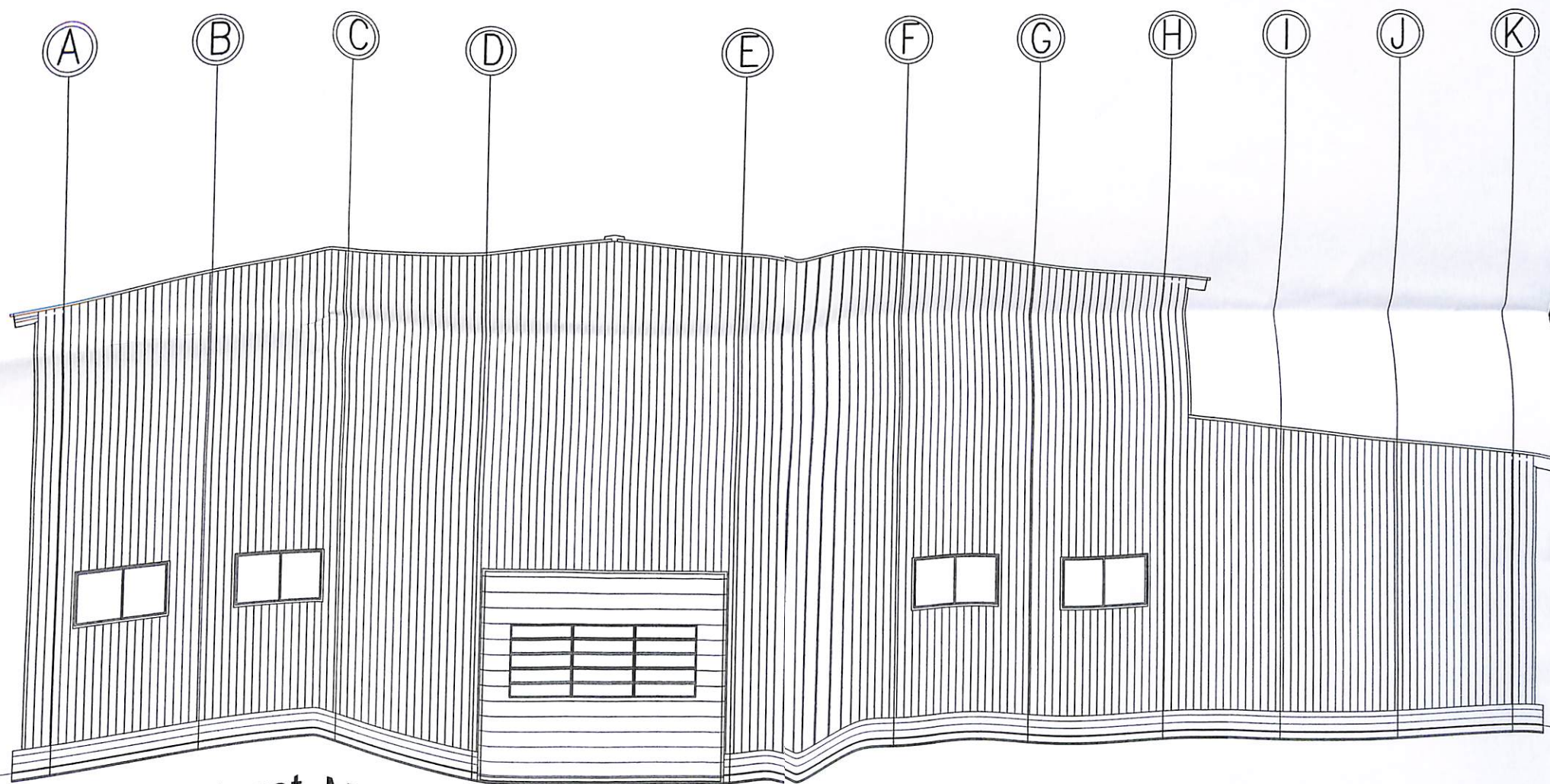


Talpgerenda
készül 116,8 fm

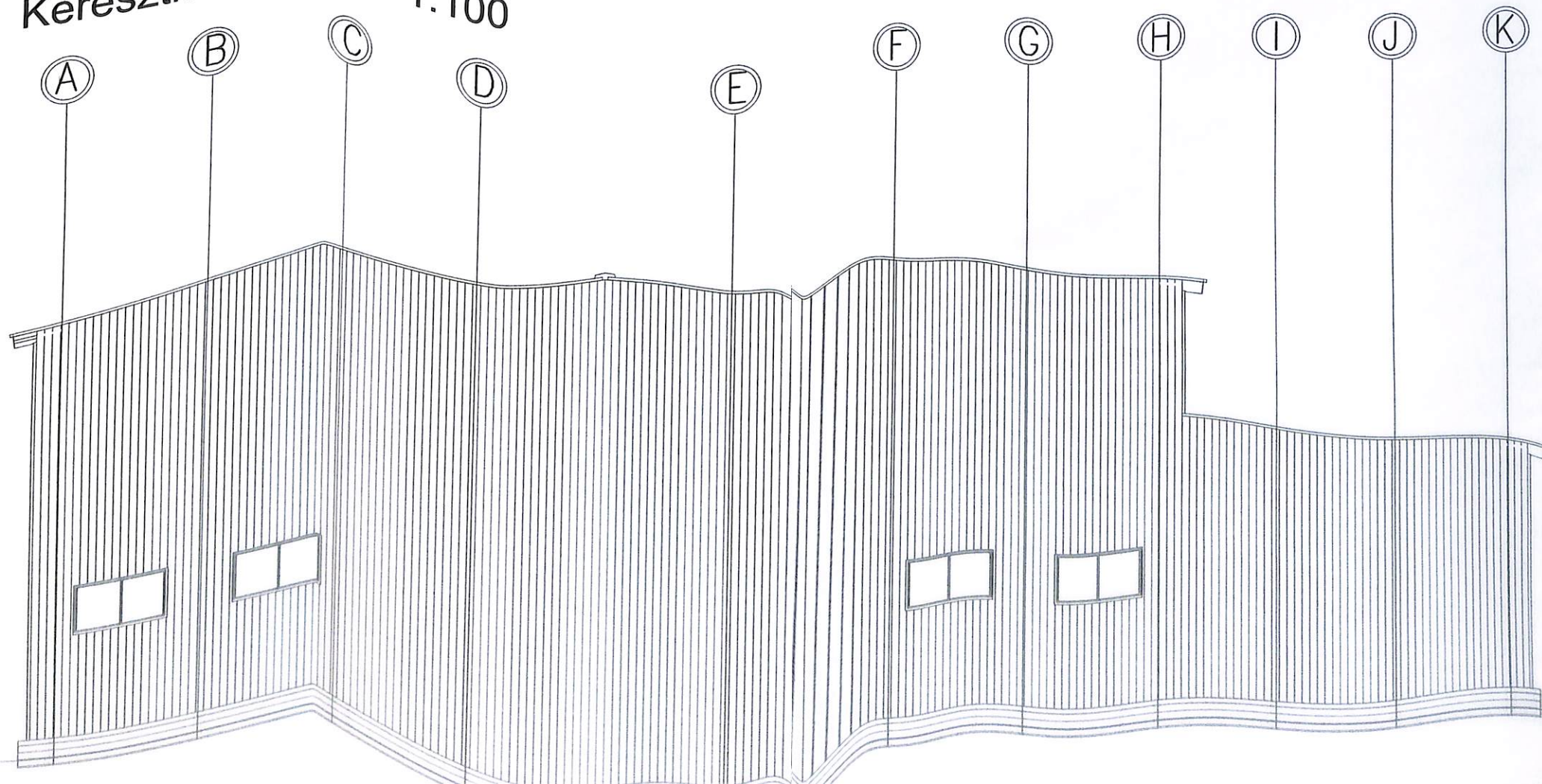
Pd (m3)	Pn (t/m3)	Fi (o)	C (kPa)	E (MN/m2)
1,85	2,20	17	115	20



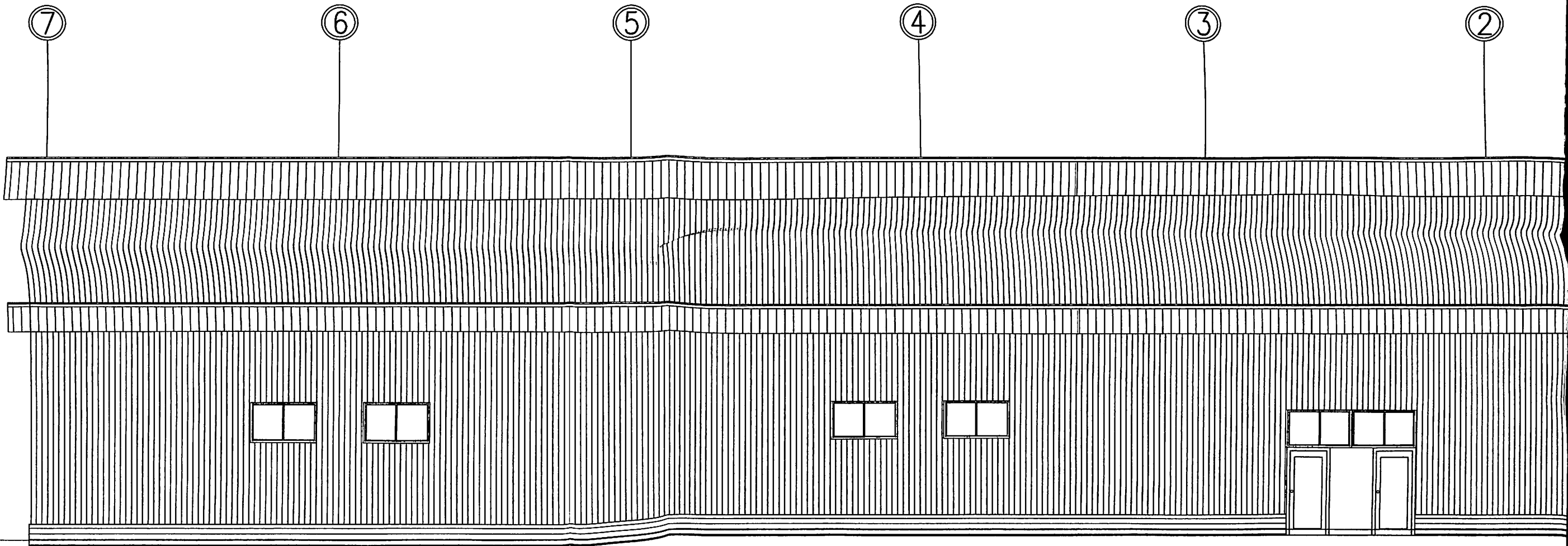
Keresztmetszet M= 1:100



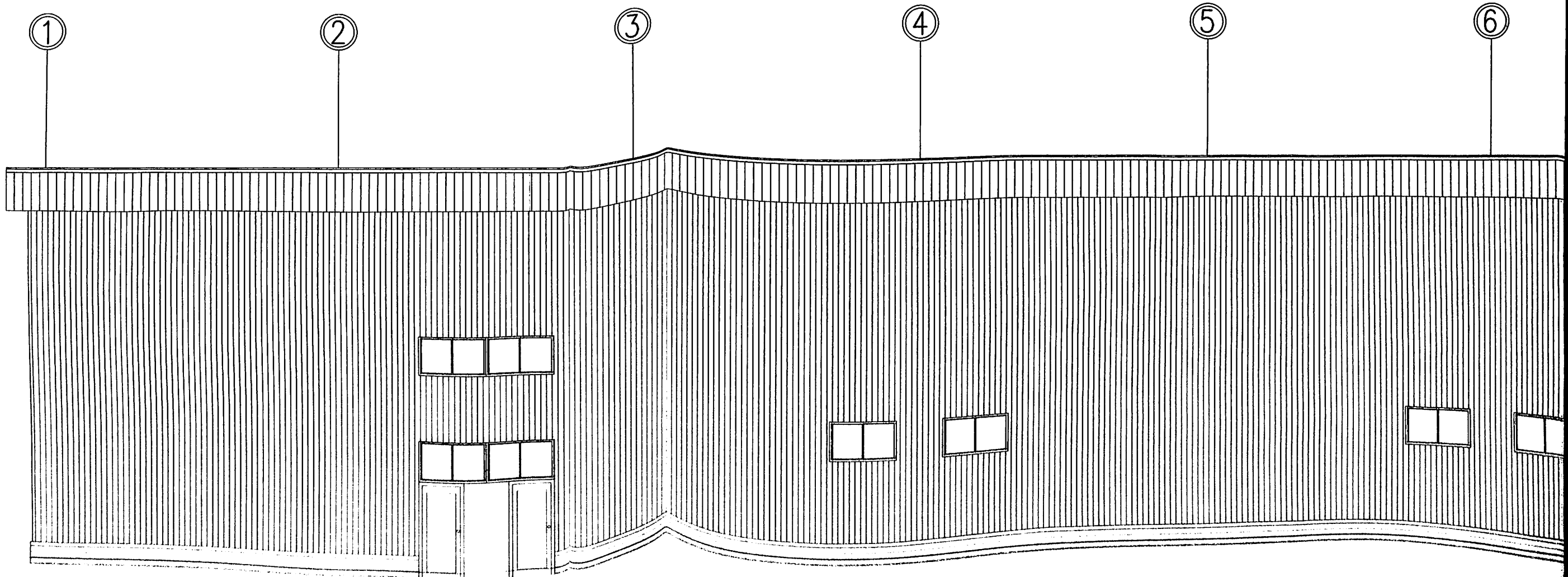
Keresztmetszet M= 1:100

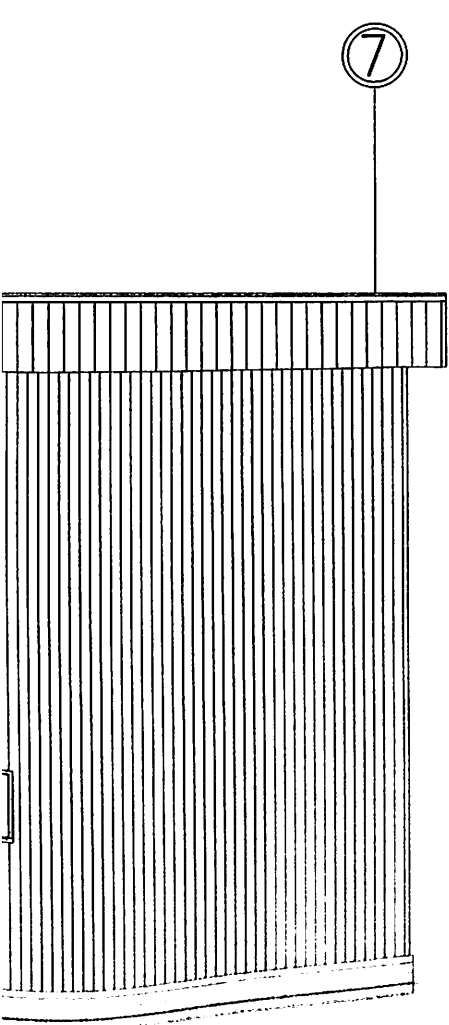
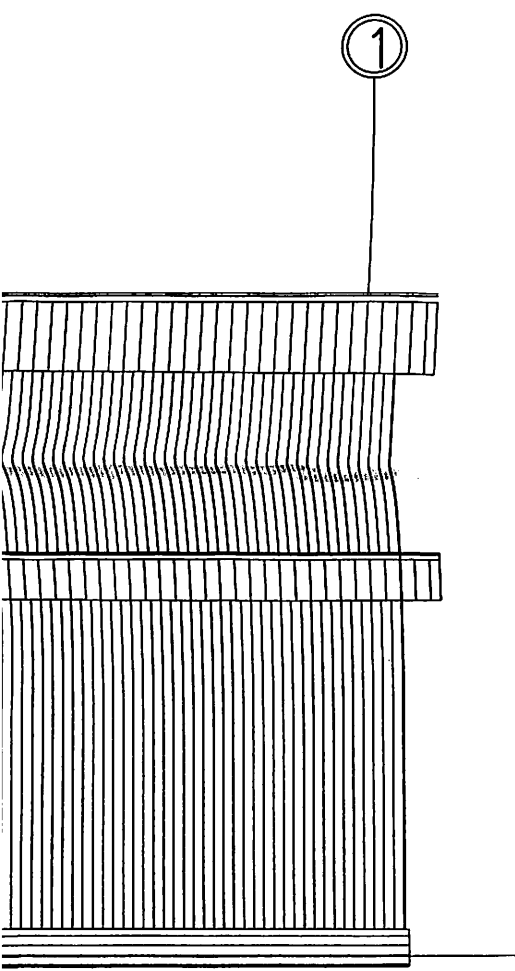


Homlokzat M= 1:100



Homlokzat M= 1:100





Megjegyzések:
+0,00 csarnok Dadlóvonalának síkja

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A-6 Alapozás