

STANOGRA D

STUDIO d.o.o.

za nadzor, konzalting i projektiranje

*ul. R. Austrije 7,
Zagreb
tel/fax: 01/375-88-00*

Građevina:

**GOSPODARSKA GRAĐEVINA
SKLADIŠNA HALA
ZA BAZENSKU TEHNIKU
Ivanić-Grad
Nova industrijska cesta
na k.č.br. 3629/1, 4066/2,
k.o. Ivanić-Grad**

Investitor:

**AQUACHEM d.o.o.
za proizvodnju i trgovinu
Ivanić-Grad
Jalševčki odvojak 17
OIB:17436320396**

Projekt:

Građevinski projekt

t.d.

124/16

Mapa:

3

Glavni projekt:

IDEA STUDIO d.o.o.

ZOP:

AH-16

**GRAĐEVINSKI PROJEKT
PROJEKT KONSTRUKCIJE
GOSPODARSKA GRAĐEVINA
SKLADIŠNA HALA ZA BAZENSKU**

Glavni projektant projekta:

Mario Pehnec, dipl.ing.arh.

Projektant konstrukcije:

Marko Gazzari, dipl.ing.grad.

OIB:41805751378

Suradnik:

Ivica Boko, dr.sc.

Direktor:

Marko Gazzari, dipl.ing.grad.

Zagreb, prosinac 2016.

SADRŽAJ:

1. OPĆI DIO

- Popis mapa
- Potvrda o upisu u komoru
- Registracija poduzeća
- Imenovanje projektanta
- Izjava o usklađenosti glavnog projekta

2. PROGRAM KONTROLE I OSIGURANJE KVALITETE

3. PRIMJENA ZAKONA I TEHNIČKI PROPISI

4. TEHNIČKI DIO

- Tehnički opis
- Statički proračun
- čelična konstrukcija
- Betonski elementi (temelji, ploče, grede)
- shema pozicija - čelik

POPIS MAPA GLAVNOG PROJEKTA Z.O.P. AH-16

1. ARHITEKTONSKI PROJEKT, T.D. broj: 01-10/16
IDEA STUDIO d.o.o. Zagreb, Bulićeva 6
Projektant: Mario Pehnec dipl.ing.arh., br.ovl. A-1156
2. GEODETSKI PROJEKT, Poslovni broj: 129/16
Ne-Pok d.o.o., Krešimira IV br 3., Ivanić-Grad,
Projektant: Matko Borić, dipl.ing.geod.et geoinf., br. ovl. GEO 1100.
3. KONSTRUKTERSKI PROJEKT, T.D. broj 124/16
STANOGRAD STUDIO d.o.o., Zagreb, ul. R. Austrije 7
Projektant: Marko Gazzari, dipl.ing.građ., br.ovl. G130
4. PROJEKT STROJARSkih INSTALACIJA, T.D. broj: BP 6091
PRO-ING d.o.o. Zagreb, Trakošćanska 6
Projektant: Ranko Bihler dipl.ing.stroj., br.ovl. S 610
5. GRAĐEVINSKI PROJEKT VODOOPSKRBE I ODVODNJE, T.D. broj: 2166-16-VIK
Ured ovlaštenog inženjera građevinarstava, Zaprešić, Lužnička 10
Projektant: Goran Vučković, dipl.ing.građ., br.ovl. G-886.
6. ELEKTROTEHNIČKI PROJEKT, Mapa: 64/16
TENIKS d.o.o. Zagreb, Domagojeva 26
Projektant: Mr.sc. Nikola Terzija, dipl.ing.el., br.ovl. E 476
7. GRAĐEVINSKI PROJEKT –GRAĐEVINSKA FIZIKA, T.D. broj: 2166-16-FIZ
Ured ovlaštenog inženjera građevinarstava, Zaprešić, Lužnička 10
Projektant: Goran Vučković, dipl.ing.građ., br.ovl. G-886.
8. ELABORAT PRIKAZA ZAŠTITE INSTALACIJA INA-INDUSTRIJA NAFTE D.D., SD ISTRAŽIVANJA I
PROIZVODNJA NAFTE I PLINA U ODNOSU NA PROJEKTIRANU SKLADIŠNU HALU ZA BAZENSKU
TEHNIKU SA PARKIRALIŠTEM I PRATEĆOM KOMUNALNOM INFRASTRUKTUROM,
T.D. broj: 01-10/16-1, IDEA STUDIO d.o.o. Zagreb, Bulićeva 6
Projektant: Mario Pehnec dipl.ing.arh., br.ovl. A-1156
9. GEOTEHNIČKI ELABORAT, Broj elaborata 095/2016
GEO-LAB d.o.o., Zagreb, Č. Truhelke 49
Odgovorni geomehaničar: Ivša Pevec, dipl.ing.građ. br.ovl. G-2264
10. ELABORAT ZAŠTITE OD POŽARA, Broj elaborata: 181116
Flamit d.o.o. Samobor, Jurja Dijanića 24a
Projektant: Željko Mužević, univ.spec.aedif., br.ovl. 64
11. ELABORAT ZAŠTITE NA RADU, Broj elaborata: 191116
Flamit d.o.o. Samobor, Jurja Dijanića 24a
Projektanti: Željko Mužević, univ.spec.aedif., br.ovl. 64



REPUBLIKA HRVATSKA
ŽUPANIJSKI SUD U ZAGREBU
Zagreb, Trg Nikole Šubića Zrinskog 5
PREDSJEDNIK SUDA

Broj: 4 Su-1427/14
Zagreb, 15. prosinca 2014. g.

RJEŠENJE

Predsjednik Županijskog suda u Zagrebu, odlučujući povodom zahtjeva Marka Gazzarija za ponovno imenovanje stalnim sudskim vještakom za graditeljstvo i procjenu nekretnina, na temelju članka 126. stavak 4. Zakona o sudovima ("Narodne Novine" 28/13) u vezi s člankom 12. Pravilnika o stalnim sudskim vještacima (NN br. 38/14),

riješio je

Nakon što je utvrđeno da ne postoje zapreke iz čl. 2. st. 2. Pravilnika o stalnim sudskim vještacima Marko Gazzari, dipl.ing.građ. iz Zagreba, Stančićeva 1, ponovno se imenuje stalnim sudskim vještakom za GRADITELJSTVO I PROCJENU NEKRETNINA na vrijeme od četiri godine.



PREDSJEDNIK SUDA
Ivan Turudić, univ.spec.crim.

- O tome obavijest:
1. Marko Gazzari
 2. Ministarstvo pravosuđa
 3. U spis



REPUBLIKA HRVATSKA
HRVATSKA KOMORA
INŽENJERA GRAĐEVINARSTVA
10000 Zagreb, Ulica grada Vukovara 271

KLASA: 102-02/16-01/335
URBROJ: 500-00-16-2
Zagreb, 01. srpnja 2016.

Hrvatska komora inženjera građevinarstva na temelju članka 159. Zakona o općem upravnom postupku ("Narodne novine", br. 47/09), po zahtjevu koji je podnio Marko Gazzari, dipl.ing.građ., Zagreb, STANČIĆEVA 1, izdaje

POTVRDU

1. Uvidom u službenu evidenciju koju vodi Hrvatska komora inženjera građevinarstva razvidno je da je Marko Gazzari, dipl.ing.građ., Zagreb, upisan u Imenik ovlaštenih inženjera građevinarstva, s danom upisa 16.06.1999. godine, pod rednim brojem 130, te je stekao pravo na uporabu strukovnog naziva "ovlašten inženjer građevinarstva", zaposlen u: STANOGRAD STUDIO d.o.o., Zagreb.
2. Ova potvrda se može koristiti samo u svrhu dokazivanja da je imenovani član Hrvatske komore inženjera građevinarstva.
3. Naknada za administrativne troškove u iznosu od 35,00 kn (slovima: trideset pet kuna) po Tar. br. 4. Odluke o naknadama za usluge koje pruža Hrvatska komora inženjera građevinarstva, uplaćena je u korist računa Hrvatske komore inženjera građevinarstva broj: 2360000-1102087559



Glavna tajnica
Hrvatske komore inženjera građevinarstva
Sara Rupić, dipl.iur.

IZVADAK IZ SUDSKOG REGISTRA

SUBJEKT UPISA

MBS:

080415454

OIB:

52281734807

TVRKA:

2 STANOGRAD STUDIO društvo s ograničenom odgovornošću za nadzor i projektiranje

2 STANOGRAD STUDIO d.o.o.

SJEDIŠTE/ADRESA:

4 Zagreb (Grad Zagreb)
Republike Austrije 7

PRAVNI OBLIK:

1 društvo s ograničenom odgovornošću

PREDMET POSLOVANJA:

- 1 * - zastupanje inozemnih tvrtki
- 1 * - kupnja i prodaja robe
- 1 * - obavljanje trgovačkog posredovanja na domaćem i inozemnom tržištu
- 5 * - Poslovi upravljanja nekretninom i održavanje nekretnina
- 5 * - Vještačenje za područje graditeljstva i procjenu nekretnina
- 5 * - Poslovanje nekretninama
- 5 * - Projektiranje i građenje građevina te stručni nadzor građenja
- 5 * - Pružanje usluga u trgovini
- 5 * - Turističke usluge u nautičkom turizmu
- 5 * - Turističke usluge u ostalim oblicima turističke ponude
- 5 * - Ostale turističke usluge
- 5 * - Turističke usluge koje uključuju športsko-rekreativne ili pustolovne aktivnosti
- 5 * - Pripremanje hrane i pružanje usluga prehrane
- 5 * - Pripremanje i usluživanje pića i napitaka
- 5 * - Pružanje usluga smještaja
- 5 * - Pripremanje hrane za potrošnju na drugom mjestu sa ili bez usluživanja (u prijevoznom sredstvu, na priredbama i sl.) i opskrba tom hranom (catering)
- 5 * - Djelatnost javnoga cestovnog prijevoza putnika ili tereta u unutarnjem cestovnom prometu
- 5 * - Prijevoz putnika u unutarnjem cestovnom prometu
- 5 * - Javni prijevoz putnika u međunarodnom linijskom cestovnom prometu
- 5 * - Prijevoz tereta u unutarnjem i međunarodnom

IZVADAK IZ SUDSKOG REGISTRA

SUBJEKT UPISA

PREDMET POSLOVANJA:

- 5 * - cestovnom prometu
- 5 * - Agencijska djelatnosti u cestovnom prometu
- 5 * - Prijevoz za vlastite potrebe
- 5 * - Djelatnost ovlaštenoga carinskog otpremnika
- 5 * - Skladištenje robe

OSNIVAČI/ČLANOVI DRUŠTVA:

- 2 Marko Gazzari, OIB: 41805751378
Zagreb, Stančićeva 1
- 2 - jedini osnivač d.o.o.

OSOBE OVLAŠTENE ZA ZASTUPANJE:

- 2 Marko Gazzari, OIB: 41805751378
Zagreb, Stančićeva 1
- 2 - direktor
- 2 - zastupa pojedinačno i samostalno
- 5 Tihana Sauha, OIB: 74145736512
Zagreb, Stančićeva 1
- 5 - prokurist
- 5 - odlukom osnivača s danom 21.05.2015.godine

TEMELJNI KAPITAL:

- 1 20.000,00 kuna

PRAVNI ODNOSI:

Osnivački akt:

- 1 Izjava o osnivanju društva od 20.prosinca 2001.g.
- 2 Odlukom člana društva od 18.03.2002. izmjenjena je Izjava o osnivanju u cijelosti. Pročišćeni tekst dostavljen u zbirku isprava.
- 4 Odlukom Skupštine od 13.09.2004. stavljena je u cijelosti van snage Izjava društva (pročišćeni tekst) od 18.03.2002., izmjenjena odredba o poslovnoj adresi u sjedištu društva i odredba o predmetu poslovanja - djelatnosti društva, novi tekst Izjave društva od 13.09.2004. dostavljen sudu i uložen u zbirku isprava.
- 5 Izjava društva od 13.09.2004.godine, izmjenjena odlukom Skupštine od 21.05.2015.godine i to čl. 3. c. predmetu poslovanja, usvojen potpuni tekst Izjave društva od 21.05.2015.godine, dostavljen sudu i uložen u zbirku isprava.

FINANCIJSKA IZVJEŠĆA:

Predano God. Za razdoblje Vrata izvješćaja
eu 30.06.15 2014 01.01.14 - 31.12.14 Čvrsto izvješće

Temeljem Zakona o gradnji R.H. (NN RH 153/13) Stanograd Studio d.o.o. donosi :

RJEŠENJE br. 124/16

O IMENOVANJU PROJEKTANTA

MARKO GAZZARI dipl.ing.grad.

sa rješenjem broj UP/I-360-01/99-01/130 od 25.09.1999. upis u imenik "ovlašteni inženjer građevinarstva" imenuje se na dužnost projektanta konstrukcije:

Građevina:

***GOSPODARSKA GRAĐEVINA
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ZA BAZENSKU TEHNIKU
Ivanić-Grad
Nova industrijska cesta
na k.č.br. 3629/1, 4066/2,
k.o. Ivanić-Grad***

Investitor:

***AQUACHEM d.o.o.
za proizvodnju i trgovinu
Ivanić-Grad
Jalševčki odvojak 17
OIB:17436320396***

Projekt:

Građevinski projekt

t.d.

124/16

Zagreb, prosinac 2016.

Direktor:

Marko Gazzari, d.i.g.

Građevina:

**GOSPODARSKA GRAĐEVINA
SKLADIŠNA HALA ZA BAZENSKU TEHNIKU
Ivanić-Grad, Nova industrijska cesta
na k.č.br. 3629/1, 4066/2, k.o. Ivanić-Grad**

Investitor:

**AQUACHEM d.o.o.
za proizvodnju i trgovinu
Ivanić-Grad, Jalševečki odvojak 17
OIB:17436320396**

Projekt:

Građevinski projekt

t.d.

124/16

Projektant konstrukcije:

Marko Gazzari, dig

Na temelju Zakona o gradnji (NN RH 153/13) GRAĐEVINSKI PROJEKT daje

I Z J A V U

I ime ovlaštenog inženjera:

Marko Gazzari, dig; samostalni ovlašteni inženjer; r.b. 130

II oznaka projekta

**GOSPODARSKA GRAĐEVINA, SKLADIŠNA HALA ZA BAZENSKU
TEHNIKU, td: 124/16**

III ovaj projekt je usklađen s:

1. Zakonom o zaštiti na radu (NN RH 93/14)
2. Zakonom o zaštiti od požara (NN 92/10)
3. Zakonom o gradnji (NN 153/13)
4. Tehnički propis za betonske konstrukcije (NN RH br. 139/09, 14/10, 125/10, 136/12)
5. Tehnički propis za zidane konstrukcije (NN RH br. 01/07)
6. Tehnički propis za spregnute konstrukcije od čelika i betona (NN br. 119/09 i 125/10)
7. Tehnički propis za čelične konstrukcije (NN br. 112/08., 125/10., 73/12., 136/12.)

Projektirani vijek trajanja konstrukcije

Razred 3 – Konstrukcije zgrada i druge uobičajene konstrukcije –
prema HRN EN 1991-1

Zahtjevani proračunski uporabni vijek > 50 godina

Izloženost ovisno o uvjetima okoliša

(prema Eurokodu 2)

Razred 1 – Suhi okoliš - unutrašnjost stambenih ili uredskih zgrada

Razred 2 – Vlažni okoliš – s mrazom;

- vanjski elementi izloženi mrazu
- elementi u neškodljivom tlu, ili vodi izloženi mrazu
- unutrašnji elementi u velikoj vlazi izloženi mrazu

(prema HRN EN 206-1)

Razred oznake – 2 – Korozija uzrokovana karbonizacijom

XC1 – Beton unutar građevine s niskom vlagom zraka

XC2 – Vlažna, rjeđe suha – Površina betona izložena dugotrajnom dodiru sa vodom; temelji – temelji samci i trake

XC3 – Umjereno vlažna – Beton unutar građevina s umjerenom ili niskom vlažnosti zraka. Vanjski beton zaštićen od kiše

Najmanja debljina zaštitnog sloja za obični beton (Eurokod 2)

Pri upotrebi čelika za armiranje, te razreda izloženosti oznake

2 – najmanji zaštitni sloj – 25 mm

Projektiranje trajnosti podrazumijeva definiranje i izvedbu betonskih elemenata odgovarajuće otpornosti, a u osnovi se sastoji u ispunjavanju tri zahtjeva koji se odnose na:

- maksimalan vodocementni faktor
- minimalan sadržaj cementa
- minimalan razred čvrstoće betona

2. PROGRAM KONTROLE I OSIGURANJA KVALITETE

Svaki građevinski proizvod predviđen za određenu namjenu može biti uporabljiv ako posjeduje takva tehnička svojstva da građevina u koju se ugrađuje ispunji BITNE ZAHTEJEVE i druge uvjete propisane Zakonom o gradnji (NN RH br. 153/13) tehničkim propisima i drugim propisima donesenim na temelju zakona, lokacijskim uvjetima utvrđenim na temelju navedenog zakona, te drugim uvjetima propisanim posebnim propisima koji su od utjecaja na bitne zahtjeve za građevinu.

Potvrđivanje sukladnosti proizvoda i sustava propisano je:

- Zakonom o gradnji (NN RH br. 153/13)
- Pravilnikom o ocjenjivanju sukladnosti, ispravama o sukladnosti i označavanju građevinskih proizvoda (N.N. 1/2005)
- Pravilnikom o izmjeni i dopuni pravilnika o ocjenjivanju sukladnosti, ispravama o sukladnosti i označavanju građevinskih proizvoda (N.N. 7/2010)
- Tehničkom propisu za betonske konstrukcije (NN 139/09, 14/10, 125/10 i 136/12)

Specificirana svojstva, dokazivanje uporabljivosti, potvrđivanje sukladnosti te označavanje građevinskih proizvoda, ispitivanje građevinskih proizvoda, posebnosti pri projektiranju i građenju, te potrebni kontrolni postupci kao i drugi zahtjevi koje moraju ispunjavati građevni proizvodi određeni su u prilogima TPBK i to za:

- beton – u Prilogu „A“
- armatura, čelik za armiranje i čelik za prednapinjanje - u Prilogu „B“
- cement - u Prilogu „C“
- agregat - u Prilogu „D“
- dodatak betonu i dodatak mortu za injektiranje natega – u Prilogu „E“
- voda – u Prilogu „F“
- predgotovljeni betonski elementi – u Prilogu „G“
- proizvodi za zaštitu i popravak betonskih konstrukcija – u Prilogu „K“

Potvrđivanje sukladnosti obuhvaća radnje ocjenjivanja sukladnosti građevinskih proizvoda ovisno o propisanom sustavu ocjenjivanja sukladnosti i izdavanje certifikata unutarnje kontrole proizvodnje odnosno izdavanje certifikata sukladnosti građevinskih proizvoda.

Program kontrole definira osnovne uvjete projekta konstrukcije za osiguranje kvalitete betona.

BETON

Tehnička svojstva betona i materijala od kojih se beton proizvodi moraju biti specificirana prema TPBK i normi HRN EN 206-1, te normama specifikacijama za materijale.

Svojstva svježeg betona specificira izvođač betonskih radova ili su prema potrebi specificirana u projektu betonske konstrukcije. Svojstva očvrstlog betona specificiraju se u projektu betonske konstrukcije. Obavezno se specificira razred tlačne čvrstoće, te ostala svojstva prema potrebi (otpornost na cikluse smrzavanja i odmrzavanja, vodonepropusnost i dr.).

Proizvođač je odgovoran za proizvodnju i transport, a Izvođač za ugradnju, zbijanje i njegu svježeg betona. Postupak njege betona provodi se prema HRN ENV 13670-1. Svojstva svježeg betona moraju se kontrolirati na mjestu proizvodnje i pri preuzimanju na mjestu ugradnje.

Prema TPBK i normi HRN EN 206-1 zaštita armature od korozije u betonu postiže se izvedbom zahtijevanog zaštitnog sloja betona, izborom vrste cementa i ograničenjem maksimalne količine kloridnih iona u betonu. Jedna od glavnih mjera zaštite armature od korozije, ali i povećanja trajnosti, je ostvarivanje kvalitetnog betona u području zaštitnog sloja, te projektiranje i izvedba dovoljne debljine zaštitnog sloja. Minimalna debljina zaštitnog sloja betona utvrđuje se u ovisnosti o razredu izloženosti, te načinu armiranja elementa.

Potvrđivanje sukladnosti sastoji se od kontrole proizvodnje koju provodi proizvođač betona uz ovlašteno tijelo. Potvrđivanje sukladnosti je postupak kojim se potvrđuje da proizvedeni beton ima svojstva prema tehničkoj specifikaciji HRN EN 206-1, prema Prilogu „A“ TPBK što je potrebno dokumentirati.

Za betone i betonske proizvode proizvedene na gradilištu, a u skladu sa projektom betonske konstrukcije, potrebno je dokazati uporabljivost u skladu sa projektom betonske konstrukcije i TPBK.

Osim isprave o sukladnosti isporučeni građevinski proizvod mora pratiti otpremnica koja sadrži podatke propisane u prilogu „A“.

Uzimanje uzoraka, priprema ispitnih uzoraka i ispitivanje svojstava svježeg betona provodi se prema normama niza HRN EN 12350, a ispitivanje svojstava očvrstlog betona prema normama niza HRN EN 12390.

Kad se betonara nalazi na gradilištu, pri uzimanju uzoraka i potvrđivanju sukladnosti betona, u gradilišnoj dokumentaciji i ostaloj dokumentaciji ispitivanja, navodi se obavezno oznaka pojedinačnog elementa betonske konstrukcije i mjesta u elementu betonske konstrukcije na kojem je ugrađen beton iz kojeg je uzet uzorak.

Označavanje betona u projektnim specifikacijama, proizvođačevim izjavama i sličnim dokumentima treba provoditi prema uputama poglavlja 11 norme HRN EN 206-1 koje se svode na obavezno navođenje norme HRN EN 206-1 i skraćenica specificiranih svojstava (razred tlačne čvrstoće, granične vrijednosti prema razredima izloženosti, najveće količine klorida, najveće nazivne gornje veličine zrna agregata, gustoće, konzistencije i sl.) Izvođenje i održavanje betonskih konstrukcija obuhvaćeno je Prilogom „J“ TPBK-a.

Pri izvođenju betonske konstrukcije izvođač je dužan pridržavati se projekta betonske konstrukcije i tehničkih uputa za ugradnju i uporabu građevinskih proizvoda i odredaba TPBK-a.

Postignuta propisana svojstva i uporabljivost građevnog proizvoda izrađenog na gradilištu izvođač treba zapisivati sukladni posebnim propisima o vođenju građevinskog dnevnika.

Zabranjena je ugradnja građevinskog proizvoda koji je isporučen bez oznake s posebnim propisom, bez tehničke upute za ugradnju i uporabu i koji nema svojstva zahtijevana projektom ili mu je istekao rok uporabe, odnosno čiji podatci značajni za ugradnju, uporabu i utjecaj na svojstva i trajnost betonske konstrukcije nisu sukladni podacima određenim glavnim projektom.

Ugradnju građevnog proizvoda mora odobriti nadzorni inženjer što se zapisuje u skladu sa posebnim propisom o vođenju građevnog dnevnika.

ARMATURA I ČELIK ZA ARMIRANJE

Tehnička svojstva i drugi zahtjevi, te dokazivanje uporabljivosti armature provodi se prema projektu betonske konstrukcije.

Tehnička svojstva i drugi zahtjevi, te potvrđivanje sukladnosti armature proizvedene prema tehničkoj specifikaciji (norme ili tehničko dopuštenje) provodi se prema toj specifikaciji, prema normama iz Priloga „B” TPBK i normama na koje one upućuju, te u skladu sa odredbama posebnog propisa.

Tehnička svojstva armature moraju ispunjavati opće i posebne zahtjeve bitne za krajnju namjenu i ovisno o vrsti čelika moraju biti specificirana prema normama nizova HRN EN 10080, odnosno HRN EN 10138 i odredbama Priloga „B” TPBK. Armatura se izrađuje, odnosno proizvodi kao armatura za armiranje betonskih konstrukcija, od čelika za armiranje.

Tehnička svojstva armature, čelika za armiranje, specificiraju se u projektu betonske konstrukcije, odnosno u tehničkoj specifikaciji za taj proizvod.

Dokazivanje uporabljivosti armature izrađene prema projektu betonske konstrukcije provodi se prema tom projektu, te prema odredbama Priloga „B” TPBK i uključuje zahtjeve za:

- a) izvođačevom kontrolom izrade i ispitivanja armature
- b) nadzorom proizvodnog pogona i nadzorom izvođačeve kontrole izrade armature na način primjeren postizanju tehničkih svojstava betonske konstrukcije, a u skladu s TPBK

Potvrđivanje sukladnosti armature prema tehničkoj specifikaciji provodi se prema odredbama te specifikacije, te odredbama Priloga „B” TPBK i posebnog propisa. Potvrđivanje sukladnosti čelika za armiranje provodi se prema odredbama Dodataka za norme HRN EN 10080-1 i odredbama posebnog propisa.

Armatura proizvedena prema tehničkoj specifikaciji označava se na otpremnici i na oznaci prema odredbama te specifikacije. Oznaka mora obavezno sadržavati upućivanje na tu specifikaciju, a u skladu s posebnim propisom.

Čelik za armiranje označava se na otpremnici i na oznaci prema normama niza HRN EN 10080, a u skladu s HRN CR 10260, normama HRN EN 10027-1:1999, HRN EN 10027-2:1999 i HRN EN 10020:1999. Oznaka mora obavezno sadržavati upućivanje na tu normu, a u skladu s posebnim propisom.

Uzimanje uzoraka, priprema ispitnih uzoraka i ispitivanje svojstava za armiranje provodi se prema normama nizova HRN EN 10080, odnosno HRN EN 10138 i prema normama niza HRN EN ISO 15630 i prema normi HRN EN 10002-1. Ako je armatura sklop čelika za armiranje i drugog čeličnog proizvoda (čelični lim, čelični profil, čelična cijev i sl.) uzimanje uzoraka i priprema ispitnih uzoraka za mehanička ispitivanja tih čeličnih proizvoda provodi se prema normi HRN EN ISO 377 Prilog „B“ TPBK.

Pri ugradnji armature treba odgovarajuće primijeniti pravila određena Prilogom „J“ TPBK, te:

- pojedinosti koje se odnose na ugradnju armature
- pojedinosti koje se odnose na sastavne materijale od kojih se armature izrađuju, te norme kojima se potvrđuje sukladnost tih proizvoda
- pojedinosti koje se odnose na uporabu i održavanje dane projektom betonske konstrukcije i/ili tehničkom uputom za ugradnju i uporabu

Pri izradi ili proizvodnji armature treba poštovati pravila armiranja prema Prilogu „H“ TPBK, priznatim tehničkim pravilima na koji taj prilog upućuje, odnosno prema Prilogu „I“ TPBK.

Za ispitivanje postupaka zavarivanja i osposobljenosti zavarivača primjenjuje se norma EN ISO 17660 ili norma HRN EN 287-1.

Armatura od čelika za armiranje ima nastavke u obliku prijeklopa, zavora ili mehaničkog spoja. Oni se proizvode i potvrđuje im se sukladnost prema tehničkoj specifikaciji ili se izrađuju prema projektu betonske konstrukcije.

Armatura izrađena prema projektu betonske konstrukcije smije se ugraditi u betonsku konstrukciju ako je sukladnost čelika, zavora, mehaničkih spojeva, spojki potvrđena ili ispitana na način određen Prilogom „B“ TPBK i ako ispunjava zahtjeve projekta betonske konstrukcije.

Prije ugradnje armature provode se odgovarajuće nadzorne radnje određene normom HRN EN 13670-1, te druge kontrolne radnje određene Prilogom „J“ TPBK.

Sastavni materijali od kojih se beton proizvodi ili koji mu se pri proizvodnji dodaju, moraju ispunjavati zahtjeve normi na koje upućuje norma HRN EN 206-1 i zahtjeve prema Prilozima „C“ , „D“, „E“ i „F“ TPBK.

CEMENT

Tehnička svojstva i drugi zahtjevi te potvrđivanje sukladnosti cementa provodi se, ovicno o vrsti cementa, prema Tehničkom propisu za cimente za betonske konstrukcije (N.N. 64/05), odredaba TPBK, Prilog „C“, te u skladu s odredbama posebnog propisa.

Tehnička svojstva cementa specificiraju se u projektu betonske konstrukcije. Kontrola cementa provodi se u centralnoj betonari (tvornici betona) i u betonari na gradilištu prema normi HRN EN 206-1.

Kasnija ispitivanja u slučaju sumnje provode se odgovarajućom primjenom normi tehničkog propisa za cement za betonske konstrukcije.

AGREGAT

Odredbe Priloga „D“ TPBK primjenjuju se na agregat koji je sastavni dio betona iz Priloga „A“ TPBK.

Tehnička svojstva agregata za beton moraju ispunjavati, ovisno o porijeklu agregata, opće i posebne zahtjeve bitne za krajnju namjenu u betonu i moraju biti specificirana prema normi HRN EN 12620, te normama na koje ta norma upućuje i odredbama Priloga „D“ TPBK.

Potvrđivanje sukladnosti agregata za beton provodi se prema odredbama Dodatka za norme HRN EN 12620 i odredbama posebnog propisa ako Prilogom „D“ TPBK nije drugačije određeno.

Postignuti rezultati ispitivanja svakog svojstva agregata za beton svrstavaju se u razrede ili daju opisno prema normi HRN EN 12620. Uzorke za ispitivanje uzima proizvođač agregata za beton i ovlaštena pravna osoba na način određen Prilogom „D“ TPBK.

Agregat za beton označava se na otpremnici i na pakovini prema normi HRN EN 12620. Oznaka mora obavezno sadržavati upućivanje na tu normu, a u skladu s posebnim propisom.

Ispitivanje svojstva ovisno o vrsti agregata za beton i laganog agregata za beton provodi se prema normama niza HRN EN 932, HRN EN 933, HRN EN 1097, HRN EN 1367 i HRN EN 1744 i odredbama Priloga „D“ TPBK.

Kontrola agregata provodi se u centralnoj betonari (tvornici betona) i betonari na gradilištu. Provodi se prema normi HRN EN 206-1. Kontrola agregata provodi se odgovarajućom primjenom normi iz točke D.3.1. Priloga „D“ TPBK.

Proizvođač i distributer agregata, te proizvođač betona dužni su poduzeti odgovarajuće mjere u cilju održavanja svojstava agregata tijekom rukovanja, prijevoza, pretovara i skladištenja prema Dodatku „H“ norme HRN EN 12620, odnosno Dodatku „F“ norme HRN EN 13055-1.

VODA

Tehnička svojstva i drugi zahtjevi, te potvrđivanje prikladnosti vode određuju se prema normi HRN EN 1008:2002.

Tehnička svojstva vode za primjenu u betonu moraju ispunjavati opće i posebne zahtjeve bitne za svojstva betona, odnosno morta za injektiranje prednapetih natega i moraju se specificirati prema normi HRN EN 1008, normama na koje ta norma upućuje i odredbama Priloga „F“ TPBK.

Potvrđivanje prikladnosti provodi se u skladu s odredbama norme HRN EN 1008, i odredbama priloga „F“ TPBK. Za pitku vodu iz vodovoda nije potrebno provoditi potvrđivanje prikladnosti za pripremu betona. Morska i bočata voda nisu prikladne za pripremu betona za armirano-betonske konstrukcije. Ispitivanje sadržaja i granične količine štetnih tvari u vodi i utjecaja tih voda na svojstva svježeg i očvrslog betona provodi se prema normi HRN EN 1008, normama na koje ta norma upućuje i odredbama Priloga „F“ TPBK.

Ispitivanje uporabljivosti, prikladnosti vode provodi se prije prve uporabe, te u slučaju kada je došlo do promjene u koncentraciji štetnih tvari u vodi, u slučaju kada postoji sumnja da je došlo do promjene u njenom sastavu.

Kontrola vode provodi se u centralnoj betonari (tvornici betona) i betonari na gradilištu prije prve uporabe, te u slučaju kada postoji sumnja da je došlo do promjene njezinih svojstava.

Kontrola u slučaju kada postoji sumnja da je došlo do promjene svojstava vode provodi se odgovarajućom primjenom norme HRN EN 1008 i normama na koje ta norma upućuje.

DODACI BETONU

Dodatci betonu prema normi HRN EN 206-1 dijele se na mineralne i kemijske dodatke. Odredbe Priloga „E“ TPBK primjenjuju se na kemijske i mineralne dodatke betonu.

Tehnička svojstva kemijskog dodatka betonu pri niskim temperaturama moraju zadovoljiti opće zahtjeve iz norme HRN EN 934-2 i posebne zahtjeve za taj tip dodatka prema normi HRN U.M1.35.

Kontrola kemijskog i mineralnog dodatka betonu provodi se u centralnoj betonari (tvornici betona) i u betonari na gradilištu prema normi HRN EN 206-1. Preporučuje se uzimanje uzoraka i odlaganje za svaku vrstu isporuka.

ODRŽAVANJE KONSTRUKCIJE

Radnje u okviru održavanja konstrukcije treba provoditi prema odredbama **Priloga J. Tehničkog propisa za betonske konstrukcije (NN br. 101/05)** i normama na koje upućuje navedeni Prilog, te odgovarajućom primjenom odredaba važećih ostalih propisa.

Bitni dijelovi konstrukcije su:

- AB konstrukcija

a) Održavanje AB konstrukcije zgrade

Redoviti pregledi u svrhu održavanja betonske konstrukcije provode se ne rjeđe od 10 godina.

Pregled uključuje najmanje:

- vizualni pregled, u kojeg je uključeno utvrđivanje položaja i veličine napuklina i pukotina te drugih oštećenja bitnih za očuvanje mehaničke otpornosti i stabilnosti građevine,
- utvrđivanja stanja zaštitnog sloja armature,
- utvrđivanje veličine progiba glavnih nosivih elemenata ako se vizualanom kontrolom sumnja u ispunjavanje bitnog zahtijeva mehaničke otpornosti i stabilnosti,

U slučaju da su pukotine veće da narušavaju trajnost AB konstrukcije potrebno ih je sanirati prema provjerenim tehničkim sustavima koji su u skladu sa TPBK.

b) Čuvanje dokumentacije održavanja

Dokumentaciju pregleda te dokumentaciju o održavanju konstrukcije dužan je trajno čuvati vlasnik građevine.

Pregled konstrukcije zgrade moraju obavljati za to ovlaštene osobe i ako se uoče da su bitna svojstva građevine narušena potrebno konstrukciju sanirati.

7. ČELIČNA KONSTRUKCIJA**Opći uvjeti za izradu i montažu čelične konstrukcije**

Čelični dio konstrukcija obrađen u ovom projektu podliježe primjeni tehničkih propisa za nosive čelične konstrukcije. Popis svih primijenjenih propisa naveden je u izjavi projektanta o usklađenosti projekta. U tehničkoj dokumentaciji (statički proračun i radioničko – montažna dokumentacija) predviđena je vrsta i kvaliteta materijala od kojeg konstrukciju treba izraditi. Materijal druge vrste i kvalitete ne može se upotrijebiti bez suglasnosti i odobrenja projektanta. U istoj tehničkoj dokumentaciji definiran je oblik, kvaliteta i pozicije. Za svaku promjenu potrebno je prethodno ishoditi odobrenje projektanta.

Osnovni dokumenti za izvođenje

Prije početka izvođenja sukladno Zakonu o prostornom uređenju i gradnji (NN RH br.: 76/2007, 38/2009, 55/2011, 90/2011 i 50/2012) potrebno je sve radove izvoditi prema:

- glavnom projektu (građevna dozvola)
- izvedbenom projektu (usklađenom s glavnim projektom)
- radioničkim nacrtima
- tehnološkom projektu

Izvođač radova izrade i montaže mora imati zakonske potvrde podobnosti.

Prije početka radova izvoditelj izrađuje tehnološki projekt izrade i montaže. Podloga za izradu ovog projekt je revidirani glavni i izvedbeni projekt, radionički nacrti čelične konstrukcije, tehnički propisi i normativi, zakon zaštite na radu i drugi važeći zakoni.

Osnovni sastav projekta je:

- opći dokumenti pogona
- rješenje o postavljanju odgovorne osobe za izradu i montažu
- opis tehnologije po kojoj se izvodi i montira čelični dio konstrukcije
- tehnološki postupak zavarivanja
- plan kontrole i popis svih potrebnih atesta materijala
- mjere i sredstva zaštite na radu
- organizacija montaže usuglašena sa ukupnom organizacijom gradilišta
- terminski planovi izrade i montaže

Dokazi kvalitete prije početka izrade čelične konstrukcije

- rješenja za voditelja izrade i montaže čelične nosive konstrukcije
- atesti materijala od kojih će biti izrađena čelična konstrukcija
- atesti za spojni materijal (vijci, elektrode)
- svjedodžbe tehnologa zavarivanja i zavarivača koji će raditi na ovoj konstrukciji
- tehnologija izrade (tehnologija zavarivanja)
- tehnologija montaže
- plan kontrole

Ova dokumentacija ovjerena od strane nadzornog inženjera odnosno projektanta sastavni je dio dokumenata za tehnički pregled konstrukcije.

Ukoliko se materijal nabavlja tijekom rada, potrebno je ateste materijala prije početka izrade dostaviti nadzornom inženjeru na ovjeru.

Kvaliteta čelika se mora biti dokazana u skladu sa normama i propisima navedenima u IZJAVI, te kvalitetom i svojstvima definiranim u statičkom proračunu.

Spojna sredstva – vijci, moraju imati kvalitetu definiranu statičkim proračunom, a dokazanu u skladu sa normama i propisima navedenima u IZJAVI.

Varovi moraju biti kvalitete i debljine definirane statičkim proračunom, a kvaliteta se mora dokazati prema normama i propisima navedenima u IZJAVI.

Kontrola u toku izrade, transporta i montaže

Tijekom izrade konstrukcije u radionici i montaže izvoditelj je dužan voditi zakonom propisane dnevnike i provoditi svoju kontrolu u skladu s planom kontrole. Dužnost je nadzornog inženjera kontrolirati izvedbu u svim fazama izrade i montaže, tj. usklađenost s tehničkom dokumentacijom i važećim tehničkim normama i pravilima, ovjeravati navedene dokumente i ateste, te zapisnik o preuzimanju elemenata u radionici prije isporuke na montažu. Sve izmjene u dimenzijama ili načinu spajanja elemenata moraju biti ovjerene od projektanta konstrukcije.

Fazne kontrole (fazni tehnički pregled) koje se provode u toku

Izvedba čelične konstrukcije ima slijedeće faze:

- 3.1.1. izrada elemenata u radionici
- 3.1.2. transport od radionice na gradilište
- 3.1.3. montaža čelične konstrukcije na gradilištu na prethodno pripremljenu sidrenu konstrukciju (temelje ili dijelove zgrade)

U pravilu se svaka faza mora pregledati i utvrditi da je izvedena prema tehničkoj dokumentaciji i prema važećim tehničkim propisima. Izvršenje fazne kontrole potvrđuju putem zapisnika odgovorne osobe projektanta, stručnog nadzora i izvoditelja. Dok se ne uklone nedostaci utvrđeni u nekoj fazi, u pravilu ne može započeti iduća faza.

Fazni pregledi sa zapisnicima potpisanim od strane odgovornih imenovanih osoba su:

- kontrola dokaza kvalitete prije početka izrade konstrukcije
- prijem čelične konstrukcije po izradi u radionici
- prijem čelične konstrukcije po transportu na gradilište
- geodetska kontrola montirane čelične konstrukcije
- završni pregled čelične konstrukcije prije početka drugih radova na čeličnoj konstrukciji (pokrivanje, oblaganje, montaža instalacija ili opreme i drugo)

Prijem elementa obavlja se na temelju radioničkih crteža i specifikacija.

Kontrola i prijem čelične konstrukcije vrši se prema Pravilniku o tehničkim mjerama i uvjetima za montažu čeličnih konstrukcija. Sve daljnje aktivnosti prigodom transporta, skladištenja i montažnih radova moraju biti u skladu s navedenim Pravilnikom. Posebno se naglašava potreba pažljivog postupanja prigodom utovara, istovara i transporta dijelova konstrukcije.

Dijelovi konstrukcije ne smiju se odlagati neposredno na zemlju nego na drvene grede i sl.

Dijelovi konstrukcije se slažu u tako da se omogući lagano pronalaženje pozicija i pristup zbog dizanja i transporta.

Prigodom prijema u radionici izvoditelji radova na izradi čelične konstrukcije dužan je staviti na uvid potrebnu tehničku dokumentaciju.

- radioničke nacрте sa specifikacijama
- ateste osnovnog materijala
- ateste dodatnog materijala
- ateste zavarivača
- ateste priključnih elemenata
- dnevnik izrade materijala
- dnevnik zavarivanja
- podatke o tehnologiji zavarivanja
- izvješće interne tehničke kontrole
- uvjerenja o kvalifikacijama stručnih osoba koje sudjeluju u izradi konstrukcije

Završnom pregledu po montaži u pravilu sudjeluje i rukovoditelj ili koordinator izgradnje cjelokupne građevine.

Zaštita čelične konstrukcije od korozije

Zaštita čelične konstrukcije od korozije u svemu se provodi kako je definirano u tehničkom opisu.

3. Primjena zakona i tehničkih propisa

Primjenjeni su važeći zakoni i tehnički propisi u pogledu mehaničke otpornosti i stabilnosti predmetne građevine. Popis primjenjenih zakona i propisa:

10.1. Zakon o gradnji (NN RH br. 153/13) i prateći posebni propisi

10.3 Tehnički propis za betonske konstrukcije (NN RH br. 139/09, 14/10, 125/10, 136/12)

10.5 Tehnički propis o građevnim proizvodima (NN RH br. 33/10, 87/10, 146/10, 81/11, 100/11, 130/12, 81/13 i 136/14)

10.6 Zakon o zaštiti na radu (NN RH br. 93/14, 118/14 i 154/14)

10.7 Zakon o zaštiti od požara (NN RH br. 92/10)

10.8 Zakon o građevnim proizvodima (NN RH br. 76/13, 30/14)

10.9 Tehnički propis za zidane konstrukcije (NN RH br. 01/07)

10.10 Tehnički propis za drvene konstrukcije (NN RH br 121/07., 58/09., 125/10., 136/12.)

TEHNIČKI OPIS

Konstruktivni sistem karakteristike

Predmet ovog elaborata je dokaz nosivosti i stabilnosti konstrukcije objekta - poslovne građevine u Ivanić Gradu.

Građevina je poslovno-skladišne namjene, te će se izvesti kao okvirna, čelična konstrukcija. Objekt je u gabaritima 42,6 x 16,6m, a katnosti je prizemlje + 2 kata na dijelu od osi A - C , dok je na preostalom dijelu jednovolumenska visine 9,3 m.

Cijeli objekt sastoji se od okvira (stup-prečka presjeka) u rasteru 4,6 i 5,6 m, dok je krovna ploha dvostrešna čelična konstrukcija, sa blagim padom od 7%. Pokrov sandwich - toplinski paneli, predviđen je prema uputama proizvođača, za nalijeganje na sekundarno čeličnu konstrukciju na osnovom rasponu od 4,6 i 5,6 m . Međukatne konstrukcije predstavljaju spregnute armiranobetonske ploče.

Fasada objekta izvodi se također od TI panela, oslonjenih na podnu gredu i čeličnu potkonstrukciju - prečke između okvira.

Temeljenje objekta

Temeljenje građevine je predviđeno kao plitko na temeljnim stopama. Geotehničkim elaboratom izrađenim od GEO-LAB d.o.o. br. td. 095/2016 utvrđena je pogodnost tla za temeljenje, te su utvrđeni parametri tla. Podna ploča je debljine 20 cm, dilatirna u zasjecanjem u određenom rasteru.

Zaključno, utvrđuje se, da će objekt izgrađen sa predviđenim elementima, u potpunosti zadovoljavati svoje funkcije nosivosti, stabilnosti, otpornosti i uporabljivosti, a izgradnja istog neće utjecati na sigurnost susjednih objekata iz razloga njihove dilatiranosti sa 5 cm EPSa i temeljenja u istom nivou.

Stabilizacija objekta izvedena je uz pomoć krovnih spregova i vertikalnih spregova okomito na ravninu glavnih okvira.

Lokacija i opterećenja

Građevina se nalazi na lokaciji koja prema važećim propisima spada u I vjetrovnu zonu, te u I zoni snijega.

Osnovni materijal

Čelični vruće valjani profili i limovi prema normi HRN EN 10025 za opće konstrukcione čelike, oznake S235 J2.

Vijci

Vijčane spojeve izvesti u skladu s HRN EN 14399. Vijci su kvalitete materijala 10.9. Temeljna sidra projektirana su iz konstrukcijskog čelika S355 J2. Ovim projektom primjenjuju se vijci dimenzija prema statičkom računu.

Dodatni materijal (zavari)

Dodatni materijal za zavarivanje mora biti kvalitete minimalno kao osnovni materijal za konstrukcije. Zavarivanje nosivih konstrukcija treba izvesti u skladu s HRN EN 13479:2007 i pratećim normama.

Radionička izrada i montaža čelične konstrukcije

Radionička izrada i montaža čelične konstrukcije treba biti u skladu sa HRN EN 1090-1 i HRN EN 1090-2. Propisana klasa izvođenja je EXC2.

U skladu sa propisanom klasom izvođenja potrebno je provoditi sve radnje kod radioničke izrade i montaže čelične konstrukcije, te kontrolu izrade i montaže čelične konstrukcije.

Zaštita od korozije

Zaštitu čelične konstrukcije od korozije treba provesti u skladu sa Tehničkim propisom za čelične konstrukcije (NN RH br. 112/08, 125/10, 73/12 i 136/12).

Prema HRN ISO 12944, građevina se nalazi u sredini s atmosferskim uvjetima koji spadaju u C3 kategoriju.

Sustav zaštite od korozije (sukladno HRN EN ISO 12944-5: 2008):

Sustav zaštite od korozije	d.s.f. (μm)
1) 1x epoksi temelj, HEMPADUR QUATRO 17634 / HEMPADUR 17410	1 × 140 μm
2) 1x HEMPATANE HS 55610	1 × 60 μm
	Ukupno: 200 μm d.s.f.

Očekivani rok trajnosti sustava zaštite od korozije: *H* (> 15 god.)

Tehnologija izvođenja zaštite od korozije**Primarna priprema površine**

Priprema površine na kvalitetu Sa 2 ½ prema HRN EN ISO 8501-1: 2007 na automatskoj sačmarilici te automatska aplikacija shopprimera na bazi cinksilikata (kao što je na pr. HEMPEL'S SHOPPRIMER ZS 15890)

Sekundarna priprema površine na gradilištu na mjestima gradilišnih zavara (prije aplikacije cjelokupnog sustava) izvodi se na kvalitetu St 3 prema HRN EN ISO 8501-1: 2007.

Nanošenje premaza

Premaze nanositi sukladno preporuci proizvođača premaza te sukladno specifikaciji bojanja i to uvijek na isključivo čistu i suhu površinu koja ima temperaturu minimalno 3 stupnja iznad točke rosišta.

Popravci na premazu

Od velike je važnosti da se sva oštećena mjesta i nedostaci poprave bilo da se radi o većim ili manjim oštećenjima. Izvođač radova treba kontrolirati i pregledati vlastiti rad. U slučaju da otkrije nedostatke kao što su nedovoljna debljina filma, rupice, curenje, nebojane površine i ostale nepravilnosti, njegov je zadatak da površinu popravi na vlastitu inicijativu.

Kontrola izvođenja zaštite od korozije**Uvod**

Cilj kontrole je da se osigura u ime naručitelja radova, u korist izvođača radova te dobavljača premaza, realizacija dogovorene specifikacije u svrhu da se postigne željeno ponašanje antikorozivne zaštite.

Proizvođač premaza mora organizirati kontrolu praćenja radova na dogovorenim referentnim površinama s osposobljenim inspektorima koji imaju položen međunarodni certifikat o poznavanju antikorozivne zaštite FROSIO ili NACE.

Dogovorene referentne površine u skladu sa standardom HRN EN ISO 12944-7: 1999 mjerodavne su za kontrolu kvalitete izvođenja radova antikorozivne zaštite. Podaci se ispunjavaju u obrascima sukladno HRN EN ISO 12944-8: 1999.

Faze kontrole

Kontrola pripreme za pripremu površine

Vizualnom metodom ocijeniti će se kvaliteta čelične površine (ISO 8501 - 1.), kvaliteta i okolina zavarova, prisutnost ulja i masnoća, dostupnost i osvjetljenost površina.

Kontrola opreme i mogućnosti izvođenja radova

Kontrolira se da li je prisutna oprema u skladu s izvođenjem radova: oprema za abrazivno čišćenje i nanošenje premaza, da li je dostupnost moguća do svih površina, da li postoji adekvatno osvjetljenje, te da li je u radionici ili na gradilištu prisutna neophodna količina boje i razrjeđivača za obavljanje posla.

Kontrola pripreme površine

Kontrola se izvodi vizualnom pregledom pripremljene površine po standardu ISO 8501 - 1.

Profil hrapavosti odrediti će se usporednom metodom: Rugotest No. 3. Kao pomoćno sredstvo za određivanje Rz profila hrapavosti može se koristiti : Elcometer 123.

Kontrola uvjeta nanošenja premaza

Kontrolira se temp. zraka, temp. površine, "DewPoint" vizuelno termometrom i ili "Sling Psychrometer-om" i usporedbom s "Dewpoint kalkulatorom".

Kontrola pripreme premaza

Kontrolira se vizualno izgled premaza, miješanje, dodatak razrjeđivača.

Kontrola nanošenja premaza i kontrola debljine mokrog filma

Kontrolira se debljina mokrog filma pomoćnim sredstvom "WFT - Gauge" s rasponom mjerenja debljina mokrog filma od 25 μ m te vizualni izgled površine.

Kontrola debljine suhog filma i prijanjanja

Kontrolira se debljina suhog filma svakog sloja 24 sata nakon otvrdnjavanja premaza elektronskim mjeracima tipa Elcometer 456 ili sličnim uređajima. Debljina suhog filma mora biti u propisanim granicama. Kontrola ukupne debljine suhog filma prema pravilu: "DFT RULES"80 - 20"

"X - rez" ili " mrežni rez" (X - Cut ili Cross- Cut) nisu relevantni za debljine filma preko 250 μ m (ISO 2409). Prirodljivost premaza može se ocijeniti tek 2 mjeseca nakon aplikacije i to jedino Pull - Off test metodom (ISO 4624); Kriteriji prihvatljivosti (minimalna sila čupanja): 3,5 Mpa, minimalna debljina čelika 8 mm, temp. kod očitavanja: od 15 do 30 °C; oprema: pneumatski ili hidraulični uređaj (sve sukladno HEMPEL's Code of Practice COP 0006-1).

Svi parametri koji se kontroliraju moraju biti u skladu s radnom specifikacijom i tehničkim uputama dobavljača premaza.

Požarna otpornost konstrukcije

Elaboratom zaštite od požara Flamit d.o.o. br. 181116; utvrđeni su zahtjevi otpornosti konstrukcije.

Betonske ploče stubišta i međukatne konstrukcije posjeduju otpornost 90 min

Dijelovi čelične konstrukcije koji zahtijevaju otpornost R 90 i REI 90 štite se primjenjenim oblogama (tip kao Knauf, Promat, Fermacell ...).

Proračunom konstrukcije programskim paketom SCIA, na modelu je predviđena požarna otpornost R 30 svih elemenata konstrukcije, te se njihovim dimenzioniranjem zaključuje vatrootpornost na R30 bez dodatnih vatrootpornih premaza.

NAPOMENA

Za prostor skadišta (prizemlje i 2. kat) potrebno je zadovoljiti zahtjeve vezane samo za reakciju na požar (tablica 1 ove točke), a zahtjevi u odnosu na otpornost konstrukcije su definirani tablicom 2.

Za pomoćne prostore (prizemlje) i prostor spremišta (1. kat) potrebno je zadovoljiti zahtjeve vezane samo za reakciju na požar (tablica 1 ove točke), a zahtjevi u odnosu na otpornost konstrukcije su definirani tablicom 3.

Karakteristike građevinskih konstrukcija u odnosu na otpornost protiv požara i reakciju na požar bit će definirane sukladno Pravilniku o otpornosti na požar i drugim zahtjevima koje građevina mora zadovoljiti u slučaju požara (NN 29/13 i 87/15) i moraju biti slijedeće:

TABLICA 1.

Zgrade podskupine 3 (ZPS3) KONSTRUKCIJE I ELEMENTI ZGRADE MORAJU ZADOVOLJITI ZAHTJEVE ZA OTPORNOST NA POŽAR	
ZAHTJEVI OTPORNOSTI NA POŽAR SIGURNOSNIH STUBIŠTA	
Zidovi stubišta	
Prizemlje i katovi ⁽²⁾	REI 60 EI 60
⁽²⁾ Zahtjevi za otpornost na požar nisu potrebni kod vanjskih zidova stubišta izvedenih od građevinskih proizvoda koji se razvrstavaju prema reakciji na požar u najmanje A2 i koji u slučaju požara ne mogu biti ugroženi susjednim dijelovima građevine spojenim na te vanjske zidove.	
Strop iznad stubišta ⁽⁴⁾	REI 60 EI 60
⁽⁴⁾ Od zahtjeva se može odstupiti ako se prijenos požara sa susjednih elemenata građevine na stubište može spriječiti odgovarajućim mjerama.	
Vrata u zidovima stubišta bez zapornice	
za poslovne prostore i druge prostore koji izravno vode na stubište	EI ₂ 30 - C
Krakov i podest stubišta	
u stubištima bez predprostora	R 60
Sustav za automatsku dojavu požara u stubištima, bez zapornice	nije potrebno
Mehanička ventilacija u stubištima bez zapornice	nije potrebno
UREĐAJ ZA ODVODNJU DIMA ⁽⁵⁾	
⁽⁵⁾ Sustav za odvodnju dima nije potreban ukoliko je predviđen sustav nadtlaka.	
Lokacija	na vrhu stubišta
Veličina	područje slobodnog presjeka od 1,00 m ²
uređaji za otvaranje	Pokretanje preko autonomnog dojavnog uređaja ⁽⁷⁾

i dodatna opcija – ručno otvaranje na posljednjem podestu i prizemlju odnosno katu na koji mogu pristupiti vatrogasci.

Otvaranje mora biti neovisno o općem napajanju električnom energijom.

Da bi se osigurao prirodni uzgon odvođenja dima iz stubišta nužno je osigurati dovod vanjskog zraka i to kanalom ili prozorom dovoljnog poprečnog presjeka sa stalnim otvorom ili vratima povezanim sa vanjskim prostorom opremljena uređajem za fiksiranje u stalno otvorenom položaju. Otvori za dovod vanjskog zraka moraju se nalaziti ispod jedne polovice srednje konstrukcijske visine stubišta.

(7) Autonomni dojavni uređaj koristi se u sigurnosnom stubištu kod zgrada u kojima nije predviđen stabilni sustav za automatsku dojavu požara, a sastoji se od centrale, rezervnog izvora napajanja, javljača dima u najvišem dijelu stubišta, te tipkala za ručno aktiviranje u najnižem i najvišem dijelu stubišta.

GRAĐEVNI PROIZVODI KOJI SE UGRAĐUJU U GRAĐEVINU TREBAJU ZADOVOLJITI ZAHTJEVE U POGLEDU REAKCIJE NA POŽAR

PROČELJA			
Ovješeni ventilirani elementi pročelja			
Klasificirani sustav	D-d1		
ili			
izvedba sa slijedećim klasificiranim komponentama			
Vanjski sloj	D		
Podkonstrukcija			
- Stapasta	D		
- Točkasta	A2		
Izolacija	D		
Toplinski kontaktni sustav pročelja			
Klasificirani sustav	D-d1		
ili			
sustav slojeva sa slijedećim klasificiranim komponentama			
- Pokrovni sloj	D		
- Izolacijski sloj	C		
Unutarnje zidne obloge i završni slojevi			
Unutarnje zidne obloge, izuzimajući evakuacijske putove			
Klasificirani sustav	D		
ili			
izvedba sa slijedećim klasificiranim komponentama			
- Obloga	D	ili	B
- Izolacija	C		D
Unutarnje zidne obloge, u evakuacijskim putovima			
Klasificirani sustav	C		
ili			
izvedba sa slijedećim klasificiranim komponentama			
- Obloga	C	ili	A2
- Podkonstrukcija	A2		A2
- Izolacija	B		D

Unutarnji završni slojevi zida unutar evakuacijskih putova			
- Stubište	C-s1,d0		
Građevni proizvodi za podove i stropove			
Podne podloge na evakuacijskim putovima			
- Stubište	Cfl-s1		
Podne podloge u neizgrađenim dijelovima potkrovlja			
Dfl			
Podne konstrukcije			
Klasificirani sustav	D		
ili			
izvedba sa slijedećim klasificiranim komponentama			
Nosivi dio	C	ili	C
Izolacijski sloj	C		D
Konstrukcije ispod neobrađene stropne ploče uključujući i pričvršćenja izuzev stropne obloge			
Klasificirani sustav	D-d0		
Ili			
izvedba sa slijedećim klasificiranim komponentama			
Podkonstrukcija	A2	ili	A2
Izolacijski sloj	C-d0		D
Obloga ili spuštenu strop	D-d0		B-d0
Stropne obloge na evakuacijskim putovima			
- Stubište	C-s1,d0		
KROVOVI			
Ravni krovovi			
- Izolacija	BKROV (t1)		
- Toplinska izolacija*	E		
Kanali za dovod zraka, kanali i ventilacijski kanali			
Kanali	C		
Izolacija	C	ili	D
Obloge	D		B
Materijali za ispunu sljubnica			
Materijal za ispunjavanje sljubnica	A2		
Ispune ograda			
u građevini (u prolazima kroz evakuacijske putove)	C		
Dupli i supltji podovi			
Dupli podovi			
- Nosivi sloj	D		
- Stupovi	D		
Supltji podovi			
- Estrih	A2		
- Oplata	D		

TABLICA 2.

Karakteristike građevinskih konstrukcija skladišta (prizemlje i 2. kat) u odnosu na otpornost protiv požara u slučaju požara moraju biti slijedeće:

GRAĐEVINSKI ELEMENT	VATROOTPORNOST	PRIMJENJENI PROPISI
nosiva konstrukcija	R 90 R 30	HRN DIN 4102 dio 4 HRN EN 1365 - 1, 3, 4 HRN EN 13501 - 2
međukatna konstrukcija	REI 90	HRN DIN 4102 dio 4 HRN EN 1365 - 2 HRN EN 13501 - 2
zidovi- granica požarnog odjeljka	REI 90 (nosivi zidovi) EI 90 (nenosivi zidovi)	HRN DIN 4102 dio 4 HRN EN 1365 - 1 HRN EN 1364 - 1 HRN EN 13501 - 2
vatrootporna vrata	EI ₂ 90-C	HRN DIN 4102 dio 5 HRN EN 1634 - 1, 2 HRN EN 13501 - 2
zaštita prolaza električnih kablova na granici požarnih odjeljaka	EI 90	HRN DIN 4102 dio 9 HRN EN 1366 - 3, 4 HRN EN 13501 - 2
zaštita prolaza ventilacijskih kanala na granici požarnih odjeljaka (PP zaklopke)	EI 90	HRN DIN 4102 dio 11 HRN EN 1366 - 2 HRN EN 13501 - 3

TABLICA 3.

Karakteristike građevinskih konstrukcija za pomoćne prostore (prizemlje) i prostor spremišta (1. kat) u odnosu na otpornost protiv požara u slučaju požara moraju biti slijedeće:

GRAĐEVINSKI ELEMENT	VATROOTPORNOST	PRIMJENJENI PROPISI
nosiva konstrukcija	R 90	HRN DIN 4102 dio 4 HRN EN 1365 - 1, 3, 4 HRN EN 13501 - 2
međukatna konstrukcija	REI 90	HRN DIN 4102 dio 4 HRN EN 1365 - 2 HRN EN 13501 - 2
zidovi- granica požarnog odjeljka	REI 90 (nosivi zidovi) EI 90 (nenosivi zidovi)	HRN DIN 4102 dio 4 HRN EN 1365 - 1 HRN EN 1364 - 1 HRN EN 13501 - 2
vatrootporna vrata	EI ₂ 30-C	HRN DIN 4102 dio 5 HRN EN 1634 - 1, 2 HRN EN 13501 - 2
zaštita prolaza električnih kablova na granici požarnih odjeljaka	EI 90	HRN DIN 4102 dio 9 HRN EN 1366 - 3, 4 HRN EN 13501 - 2
zaštita prolaza ventilacijskih kanala na granici požarnih odjeljaka (PP zaklopke)	EI 90	HRN DIN 4102 dio 11 HRN EN 1366 - 2 HRN EN 13501 - 3



WAZELAS POPEY-VISUCCI

ANALIZA OPTEREĆENJA: (vlastite težine ploča su uključene u model)

1. krovna konstrukcija - neprohodni krov

- izolacijski krovni panel	= 0,25 kN/m ²
- podgled (zavješnja)	= <u>0,2 kN/m²</u>
stalno g	= 0,45 kN/m ²
korisno (snijeg) 120 m.n.v. 0,8x1,2	= 1,0 kN/m ²

2. međukatna konstrukcija - spregnuta ploča na 5,78

- protuprašni premaz	= 0,05 kN/m ²
- podgled	= <u>0,2 kN/m²</u>
dodatno stalno g	= 0,25 kN/m ²
korisno (priručno skladište) p	= 3,0 kN/m ²

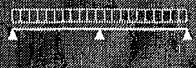
3. međukatna konstrukcija - spregnuta ploča na 2,98

- keramičke pločice u ljepilu	= 0,25 kN/m ²
- podgled	= <u>0,2 kN/m²</u>
dodatno stalno g	= 0,45 kN/m ²
korisno (uredi) p	= 1,5 kN/m ²

Pokrov - izolacijski krovni paneli kao Kingspan X-dek postavljaju se na čeličnu konstrukciju na osne razmake 4,6 i 5,6 metara, a sukladno uputama proizvođača.

Odabrano: - panel XD 100; debljine donjeg lima 0,9mm

TABLICE RASPODJELE OPTEREĆENJA za XM, XG, XB

Jednopoljni raspon Statička shema	Debljina donjeg lima [mm]	Debljina izolacije	Slučaj opterećenja	Tip opterećenja	Jednoliko raspodijeljeno opterećenje u kN/m ² pri danom rasponu				
					4.00m	4.50m	5.00m	5.50m	6.00m
	0.9	XD 80	ULS	Prema dolje	4.704	3.561	2.758	2.279	1.915
				Prema gore	3.329	2.417	1.785	1.529	1.329
			SLS	Prema dolje	2.622	1.757	1.220	0.929	0.725
				Prema gore	2.500	1.728	1.240	0.946	0.740
		XD 100	ULS	Prema dolje	4.704	3.639	2.885	2.378	1.993
				Prema gore	3.433	2.559	1.949	1.487	1.145
			SLS	Prema dolje	2.619	1.777	1.251	0.963	0.760
				Prema gore	2.350	1.717	1.300	0.969	0.740
Dvopoljni raspon Statička shema	Debljina donjeg lima [mm]	Debljina izolacije	Slučaj opterećenja	Tip opterećenja	Jednoliko raspodijeljeno opterećenje u kN/m ² pri danom rasponu				
					4.00m	4.50m	5.00m	5.50m	6.00m
	0.9	XD 80	ULS	Prema dolje	2.853	2.316	1.927	1.561	1.285
				Prema gore	3.095	2.388	1.886	1.702	1.551
			SLS	Prema dolje	5.466	4.107	3.189	2.506	2.015
				Prema gore	4.981	3.816	3.014	2.352	1.879
		XD 100	ULS	Prema dolje	3.187	2.448	1.927	1.553	1.271
				Prema gore	3.108	2.746	2.460	2.161	1.924
			SLS	Prema dolje	5.466	3.911	2.903	2.398	2.015
				Prema gore	4.662	3.704	3.014	2.379	1.920
Jednopoljni raspon Statička shema	Debljina donjeg lima [mm]	Debljina izolacije	Slučaj opterećenja	Tip opterećenja	Jednoliko raspodijeljeno opterećenje u kN/m ² pri danom rasponu				
					4.50m	5.00m	5.50m	6.00m	6.50m
	1.1	XD 80	ULS	Prema dolje	4.367	3.475	2.844	2.337	1.968
				Prema gore	3.848	3.146	2.565	2.228	1.917
			SLS	Prema dolje	1.990	1.391	1.017	0.728	0.535
				Prema gore	2.200	1.670	1.300	1.040	0.860
		XD 100	ULS	Prema dolje	4.452	3.543	2.900	2.383	2.006
				Prema gore	3.848	3.146	2.565	2.228	1.917
			SLS	Prema dolje	2.027	1.417	1.036	0.741	0.545
				Prema gore	2.200	1.670	1.300	1.040	0.860
Dvopoljni raspon Statička shema	Debljina donjeg lima [mm]	Debljina izolacije	Slučaj opterećenja	Tip opterećenja	Jednoliko raspodijeljeno opterećenje u kN/m ² pri danom rasponu				
					4.50m	5.00m	5.50m	6.00m	6.50m
	1.1	XD 80	ULS	Prema dolje	3.450	2.805	2.310	1.920	1.620
				Prema gore	4.185	3.416	2.862	2.417	2.093
			SLS	Prema dolje	2.530	2.057	1.694	1.408	1.205
				Prema gore	3.410	2.783	2.332	1.969	1.669
		XD 100	ULS	Prema dolje	3.450	2.805	2.310	1.920	1.620
				Prema gore	4.185	3.416	2.862	2.417	2.093
			SLS	Prema dolje	2.645	2.151	1.771	1.472	1.242
				Prema gore	3.565	2.910	2.438	2.059	1.783

NAPOMENE:

ULS – krajnje granično stanje – dana opterećenja bi trebala biti uspoređena s graničnim (konstrukcijskim) opterećenjima
SLS – granično stanje uporabivosti – dana opterećenja bi trebala biti uspoređena s karakterističnim (nereguliranim) opterećenjima
Maksimalni dopuštena progib (SLS): $L / 200$
Minimalna širina nosača: na krajevima – 40mm, u sredini – 120mm
Vlastito opterećenje panela je uključeno u gornje iznose

PRORACUN KONSTRUKCIJE

ANALIZA OPTEREĆENJA

Sva opterećenja na konstrukciju uzeta su prema važećim propisima, tj. hrvatska norma HRN EN-1991.

- HRN EN 1991-1-1: 2012 – Eurokod 1: Djelovanja na konstrukcije -- Dio 1-1: Opća djelovanja -- Obujamske težine, vlastite težine i uporabna opterećenja zgrada (EN 1991-1-1:2002+AC:2009)
- HRN EN 1991-1-1:2012/NA: 2012 - Eurokod 1: Djelovanja na konstrukcije - Dio 1-1: Opća djelovanja - Obujamske težine, vlastite težine i uporabna opterećenja za zgrade - Nacionalni dodatak
- HRN EN 1991-1-3: 2012 - Eurokod 1: Djelovanja na konstrukcije -- Dio 1-3: Opća djelovanja - Opterećenja snijegom (EN 1991-1-3:2003+AC:2009)
- HRN EN 1991-1-3:2012/NA: 2016 - Eurokod 1: Djelovanja na konstrukcije -- Dio 1-3: Opća djelovanja - Opterećenja snijegom - Nacionalni dodatak
- HRN EN 1991-1-4: 2012 - Eurokod 1: Djelovanja na konstrukcije - Dio 1-4: Opća djelovanja - Djelovanja vjetra (EN 1991-1-4:2005+AC:2010+A1:2010)
- HRN EN 1991-1-4:2012/NA: 2012 - Eurokod 1: Djelovanja na konstrukcije - Dio 1-4: Opća djelovanja - Djelovanja vjetra - Nacionalni dodatak

Vlastita težina

Vlastita težina smatra se stalnim djelovanjem, tj. za vlastitu težinu se smatra da će djelovati na konstrukciju tijekom cijelog vijeka trajanja konstrukcije. U statičkom proračunu automatski se proračunava od strane programskog paketa u kojem se analizira promatrana konstrukcija. Može se prikazati pomoću jedne karakteristične vrijednosti (G_k), a proračunava se na osnovu prostornih težina i nazivnih dimenzija.

Dodatno stalno opterećenje Krovn konstrukcija

	d (cm)	γ (kN/m ³)	d × γ
Krovni panel			0.25
Podgled+instalacije			0.20

Ukupno dodatno stalno opterećenje: $g_0 = 0,45 \text{ (kN/m}^2\text{)}$

Međukatna konstrukcija:

	d (cm)	γ (kN/m ³)	d × γ
Armirano betonska konstrukcija	15	25.0	3.75
Kamena vuna	10		0.15
Podgled			0.20

Ukupno dodatno stalno opterećenje: $g_0 = 0,76 \text{ (kN/m}^2\text{)}$

Opterećenje vjetrom:

Predmetna građevina se nalazi u općini Ivanić grad, na poziciji gdje je uglavnom nezaštićena od djelovanja vjetra. Prema navedenim normama, predmetna lokacija ima slijedeće karakteristike opterećenja vjetra:

Predmetna građevina:

Visina objekta (od kote terena):	9.3 m
Širina objekta (x-smjer) (m):	16.9 m
Dužina objekta (y-smjer) (m):	42.7 m

Osnovna brzina vjetra za predmetnu lokaciju:

osnovna brzina vjetra:	$V_{b,0} = 20.00$ m/s
koeficijenti:	$C_{DIR} = 1.00$
	$C_{season} = 1.00$
	$V_b = 20.00$ m/s

Srednja brzina vjetra

Kategorija terena	II.
Najmanja visina z_{min} :	= 2 m
Duljina hrapavosti z_0 :	= 0.050 m
Najveća visina z_{max} :	= 100 m
Faktor vertikalne razvedenosti $c_0(z)$:	= 1.00
Faktor terena $k_r = 0.19(z_0/z_{0,II})^{0.07}$:	= 0.190
Faktor hrapavosti: $c_r(z)$:	= 0.99

$$v_m(z) = v_b \cdot c_r(z) \cdot c_0(z) = 19.86 \text{ m/s}$$

Tlak pri osnovnoj brzini vjetra:

$$q_b = \frac{1}{2} \cdot \rho \cdot v_b^2 = 0.25 \text{ kN/m}^2$$

Faktor izloženosti

$$c_e(z) = 2.20$$

(iz dijagrama):

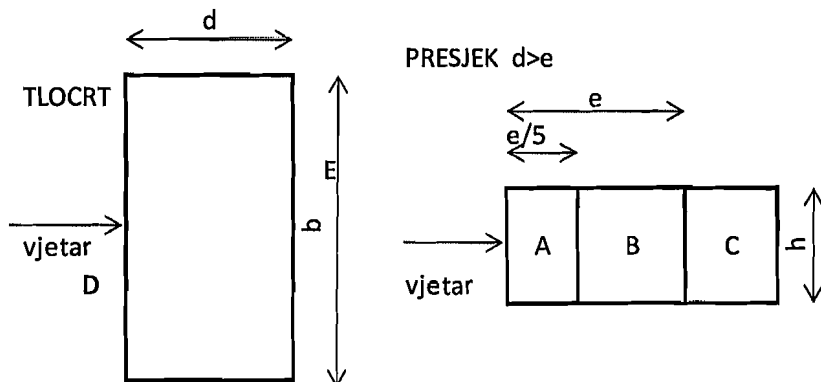
Tlak pri vršnoj brzini:

$$q_p(z) = 0.55 \text{ kN/m}^2$$

Tlak vjetra na površine $w_e = q_p(z_e) \cdot c_{pe}$

Opterećenje vertikalnih ploha x-smjer

$$z_e = 9.3 \text{ m} \quad e = 16.9 \text{ m}$$



$$e/5 = 3.38 \text{ m}$$

$$h/d = 0.55$$

zona	A	B	C	D	E
h/d	$C_{pe,10}$	$C_{pe,10}$	$C_{pe,10}$	$C_{pe,10}$	$C_{pe,10}$
5	-1.4	-0.8	-0.5	0.8	-0.5
1	-1.2	-0.8	-0.5	0.8	-0.5
<0,25	-1.2	-0.8	-0.5	0.8	-0.3

Površine:

$$A = 31.43 \text{ m}^2$$

$$B = 125.74 \text{ m}^2$$

$$w_e^A = -0.66 \text{ kN/m}^2$$

$$w_e^B = -0.44 \text{ kN/m}^2$$

$$C = 0.00 \text{ m}^2$$

$$D = 397.11 \text{ m}^2$$

$$E = 397.11 \text{ m}^2$$

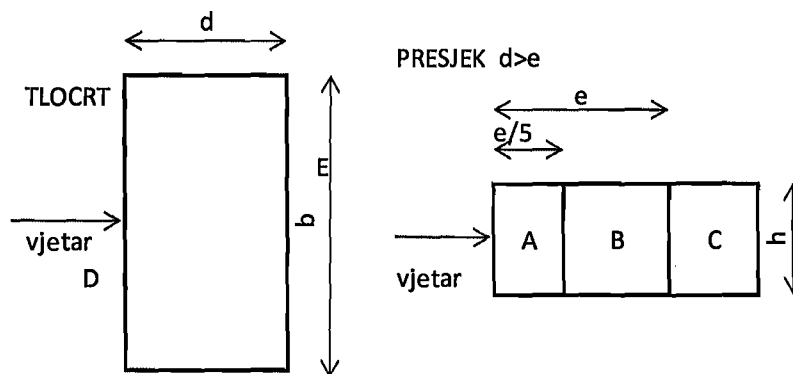
$$w_e^C = -0.28 \text{ kN/m}^2$$

$$w_e^D = 0.44 \text{ kN/m}^2$$

$$w_e^E = -0.28 \text{ kN/m}^2$$

Opterećenje vertikalnih ploha y smjer

$z_e = 9.3 \text{ m}$ $e = 18.6 \text{ m}$



$e/5 = 3.72 \text{ m}$
 $h/d = 0.22$

zona	A	B	C	D	E
h/d	$C_{pe,10}$	$C_{pe,10}$	$C_{pe,10}$	$C_{pe,10}$	$C_{pe,10}$
5	-1.4	-0.8	-0.5	0.8	-0.5
1	-1.2	-0.8	-0.5	0.8	-0.5
<0,25	-1.2	-0.8	-0.5	0.8	-0.3

Površine:

A= 34.60 m²

B= 138.38 m²

$w_e^A = -0.77 \text{ kN/m}^2$

$w_e^B = -0.44 \text{ kN/m}^2$

C= 224.13 m²

D= 157.17 m²

E= 157.17 m²

$w_e^C = -0.28 \text{ kN/m}^2$

$w_e^D = 0.44 \text{ kN/m}^2$

$w_e^E = -0.28 \text{ kN/m}^2$

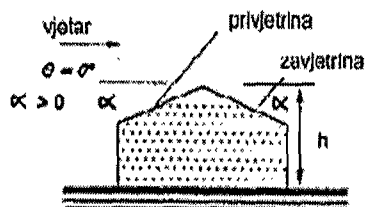
Opterećenje dvostrešnog krova

b= 16.9 m

a= 42.7 m

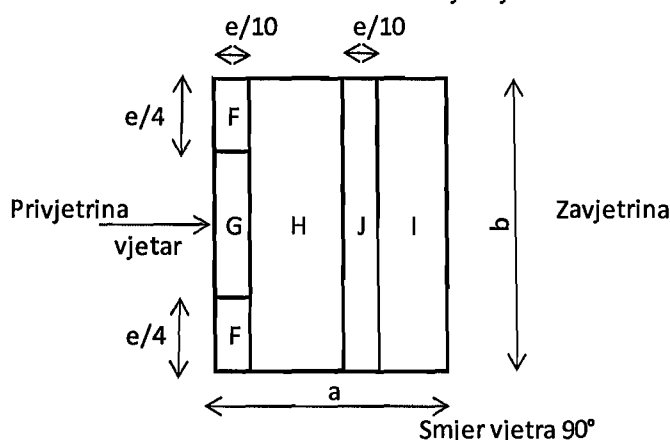
$z_e = h = 9.3 \text{ m}$

Nagib: 10.00°

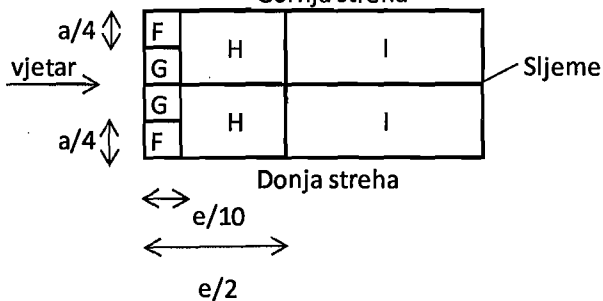


pozitivan nagib krova

Smjer vjetra 0°



Gornja streha



Smjer vjetra $\Theta = 0^\circ$

e= 16.9 m	nagib	F	G	H	I	J
		$C_{pe,10}$	$C_{pe,10}$	$C_{pe,10}$	$C_{pe,10}$	$C_{pe,10}$
	10	-1.30	-1.00	-0.45	-0.20	-0.65

Površine:

F= 7.14 m ²	$w_e^F = -0.72$ kN/m ²	H= 332.25 m ²	$w_e^H = -0.25$ kN/m ²
G= 14.28 m ²	$w_e^G = -0.55$ kN/m ²	I= 332.25 m ²	$w_e^I = -0.11$ kN/m ²
		J= 28.56 m ²	$w_e^J = -0.36$ kN/m ²

Smjer vjetra $\Theta = 90^\circ$

e= 18.6 m	nagib	F	G	H	I
		$C_{pe,10}$	$C_{pe,10}$	$C_{pe,10}$	$C_{pe,10}$
	10	-1.45	-1.30	-0.65	-0.55

Površine:

F= 19.86 m ²	$w_e^F = -0.80$ kN/m ²	H= 158.84 m ²	$w_e^H = -0.36$ kN/m ²
G= 19.86 m ²	$w_e^G = -0.72$ kN/m ²	I= 162.26 m ²	$w_e^I = -0.30$ kN/m ²

Pritisak vjetra na unutarnje površine

Unutrašnji koeficijent pritiska: $c_{pi} = \pm 0.3$

$w_i = \pm 0.17$ kN/m²

Rezultirajuće djelovanje vjetra smjer-x:

$w_k = w_e - w_i$ (kN/m²)

Vjetar W1 - pozitivni unutarnji pritisak $c_{pi} = + 0.3$)

PODRUČJE	A	B	C	D	E	F	G	H	I	I
w_e (kN/m ²)	-0.66	-0.44	-0.28	0.44	-0.28	-0.72	-0.55	-0.25	-0.11	-0.36
w_i (kN/m ²)	0.17	0.17	0.17	0.17	0.17	0.17	0.17	0.17	0.17	0.17
w_k (kN/m ²)	-0.83	-0.61	-0.44	0.28	-0.44	-0.88	-0.72	-0.41	-0.28	-0.52

Vjetar W2 - negativni unutarnji pritisak $c_{pi} = - 0.3$)

PODRUČJE	A	B	C	D	E	F	G	H	I	I
w_e (kN/m ²)	-0.66	-0.44	-0.28	0.44	-0.28	-0.72	-0.55	-0.25	-0.11	-0.36
w_i (kN/m ²)	-0.17	-0.17	-0.17	-0.17	-0.17	-0.17	-0.17	-0.17	-0.17	-0.17
w_k (kN/m ²)	-0.50	-0.28	-0.11	0.61	-0.11	-0.55	-0.39	-0.08	0.06	-0.19

Rezultirajuće djelovanje vjetra smjer-y:

$w_k = w_e - w_i$ (kN/m²)

Vjetar W1 - pozitivni unutarnji pritisak $c_{pi} = + 0.3$)

PODRUČJE	A	B	C	D	E	F	G	H	I
w_e (kN/m ²)	-0.77	-0.44	-0.28	0.44	-0.28	-0.80	-0.72	-0.36	-0.30
w_i (kN/m ²)	0.17	0.17	0.17	0.17	0.17	0.17	0.17	0.17	0.17
w_k (kN/m ²)	-0.94	-0.61	-0.44	0.28	-0.44	-0.96	-0.88	-0.52	-0.47

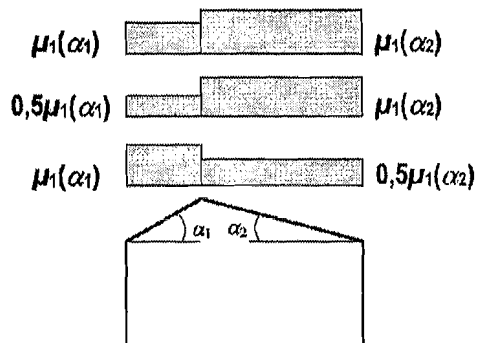
Vjetar W2 - negativni unutarnji pritisak $c_{pi} = - 0.3$)

PODRUČJE	A	B	C	D	E	F	G	H	I
w_e (kN/m ²)	-0.77	-0.44	-0.28	0.44	-0.28	-0.80	-0.72	-0.36	-0.30
w_i (kN/m ²)	-0.17	-0.17	-0.17	-0.17	-0.17	-0.17	-0.17	-0.17	-0.17
w_k (kN/m ²)	-0.61	-0.28	-0.11	0.61	-0.11	-0.63	-0.55	-0.19	-0.14

Opterećenje snijegom

Lokacija objekta: IVANIĆ GRAD
Nadmorska visina: 110 m.n.m
Područje opterećenja snijegom: I
 $s_k = 1.15$

Opterećenje snijegom dvostrešnih krovova

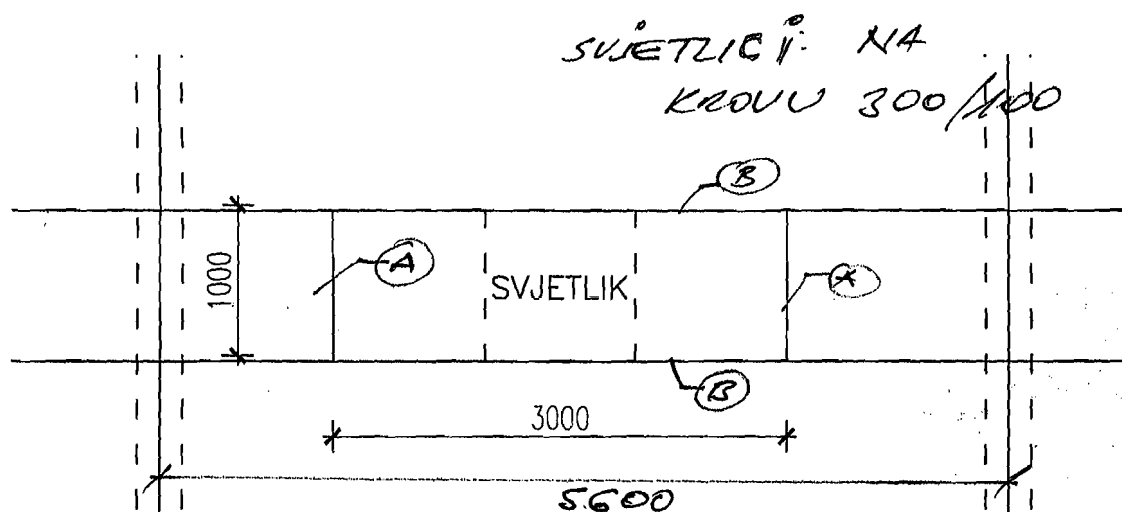


Koeficijent oblika za opterećenje snijegom - dvostrešni krovovi

$\alpha_1 = 3.00^\circ$ $\alpha_2 = 3.00^\circ$
 $\mu_1 = 0.80$ $\mu_2 = 0.80$

Opterećenje snijegom:

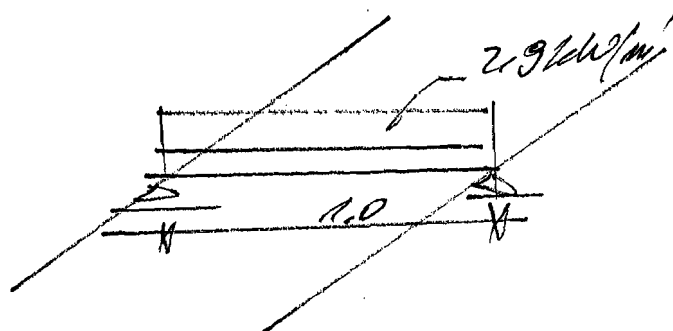
Kombinacija opterećenja 1: $S_{1(\alpha_1)} = 0.92 \text{ kN/m}^2$ $S_{1(\alpha_2)} = 0.92 \text{ kN/m}^2$
 $0.5 S_{1(\alpha_1)} = 0.46 \text{ kN/m}^2$ $0.5 S_{1(\alpha_2)} = 0.46 \text{ kN/m}^2$



OJHAČANJA OKO SVJETLIKA

- opterećenja na (A)
- od T.I. PANELE - $0,6 \times 1,5 = 0,9 \text{ kN/m}^2$
- od svjetlika $1, \times 2,0 = \frac{2,0 \text{ kN/m}^2}{2,9 \text{ kN/m}^2}$

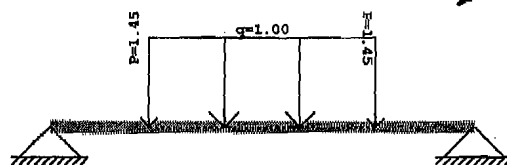
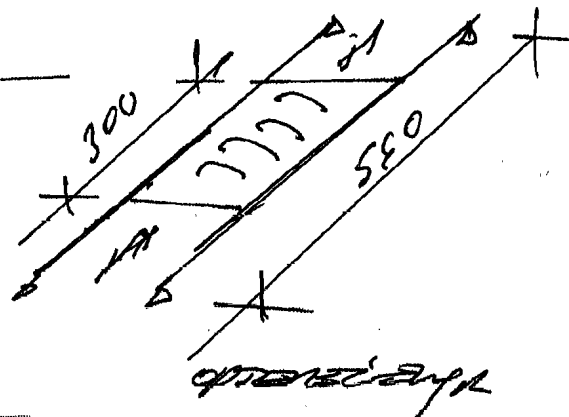
STAT. SHETA



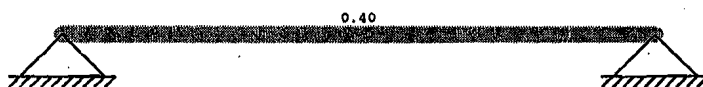
OPREĆENJA NA (B)

- od (A) $\downarrow = 2,9 \times 1,0 \times 0,5 = 1,45 \text{ kN}$
- od svjetlika $= 2 \times 1 \times 0,5 = 1,0 \text{ kN/m}^2$
- + v.h. t.

GREDA (B) - NPU 140

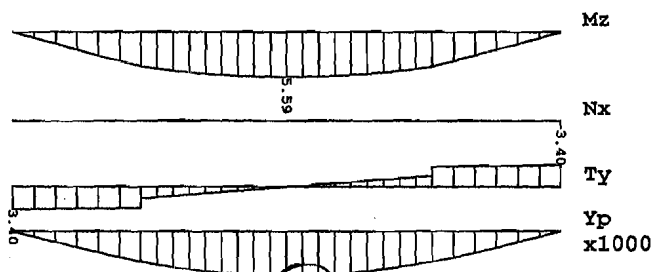


Opt. 1: sve (g) - Ulazni podaci



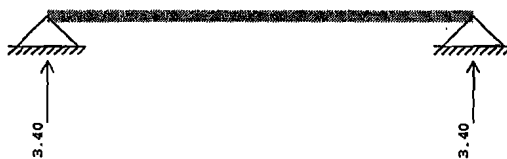
Kontrola stabilnosti (čelik)

Stabilnost
OK



Opt. 1: sve (g) Utjecaji u gredi: 1 - 4

Profil 14 mm
OK



Opt. 1: sve (g) - Reakcije ležajeva (kN-kNm)

Reakcije na
HEA profile

GREDA (A) = GREDA (B)

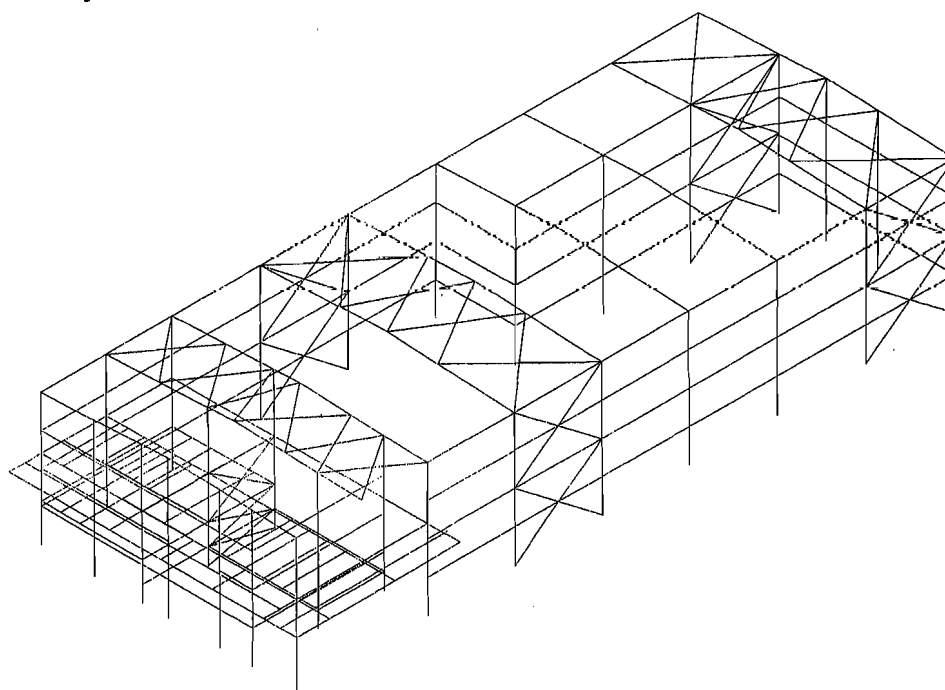
NPU 140 - Zadržavanje

NOSIVA ČELIČNA KONSTRUKCIJA

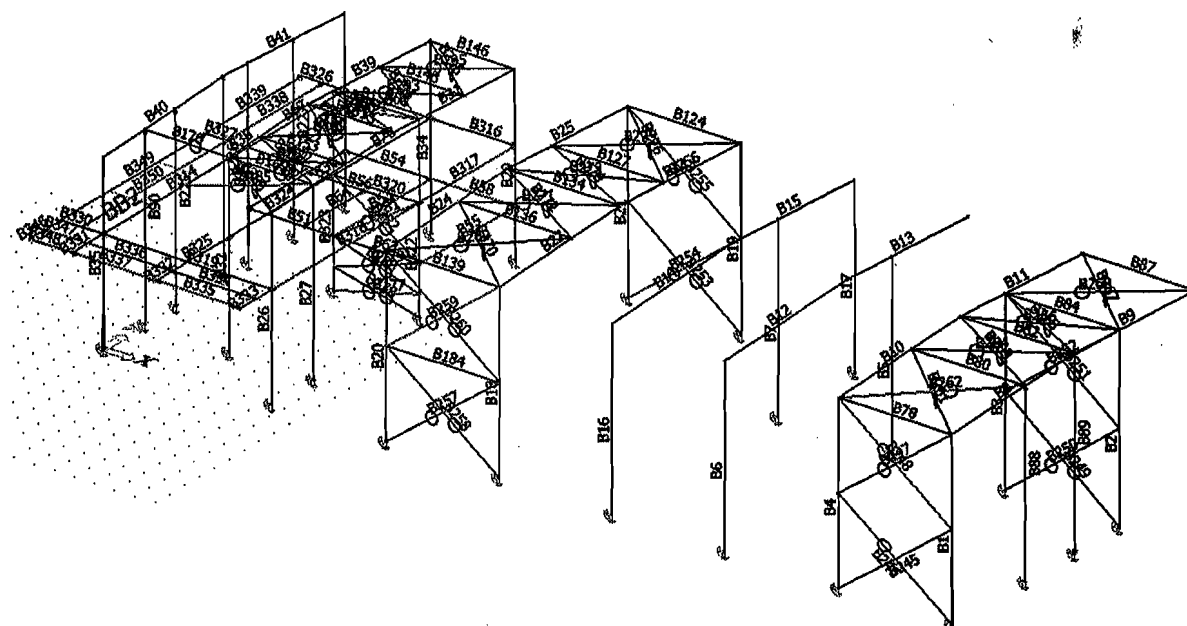
Proračun reznih sila napravljen je u programskom paketu SCIA ENGINEER. Analizirane su sve kombinacije opterećenja za granično stanje nosivosti i granično stanje uporabljivosti s odgovarajućim parcijalnim koeficijentima sigurnosti za pojedino granično stanje. U nastavku su dani samo utjecaji od najnepovoljnije kombinacije djelovanja za pojedino granično stanje.

Globalni model konstrukcije

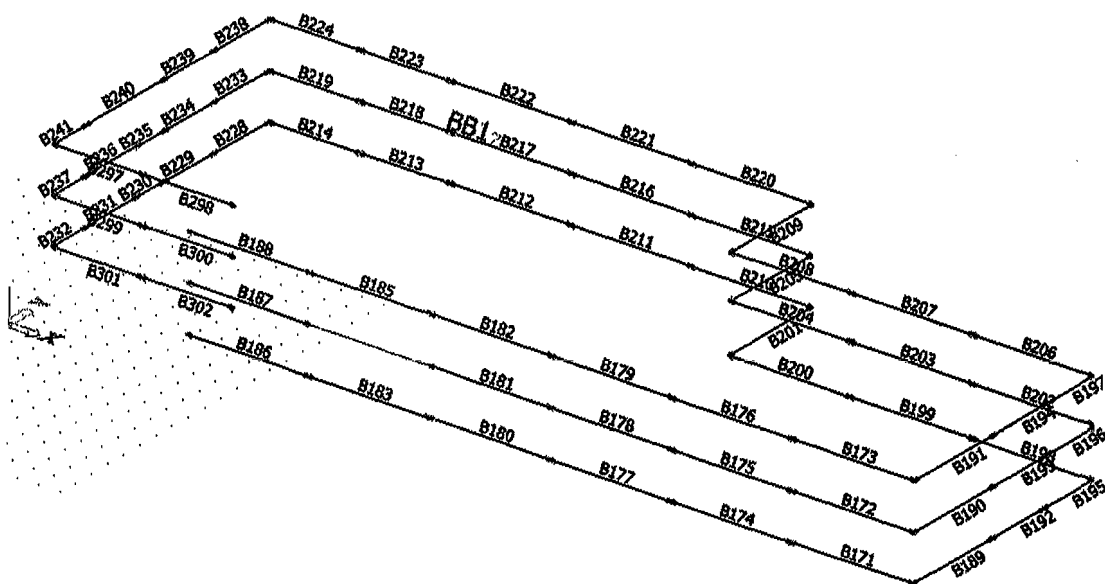
Prikaz geometrije modela:



Prikaz elemenata glavne nosive konstrukcije:

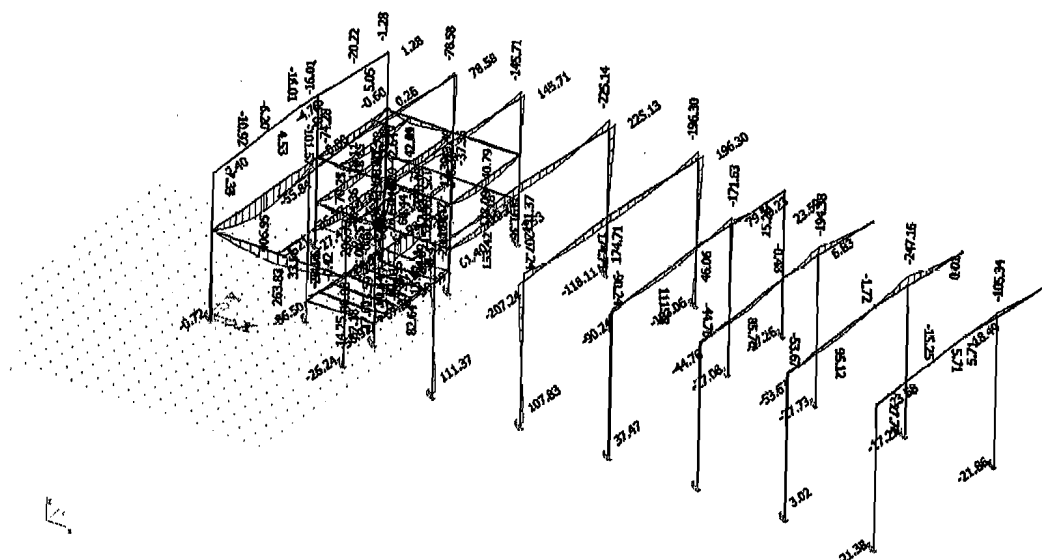


Prikaz elemenata sekundarne konstrukcije:

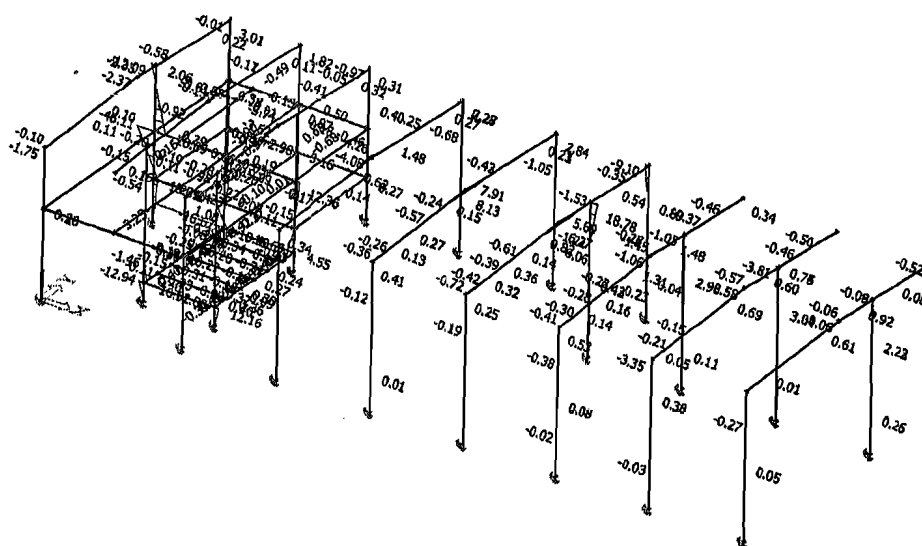


Glavna konstrukcija

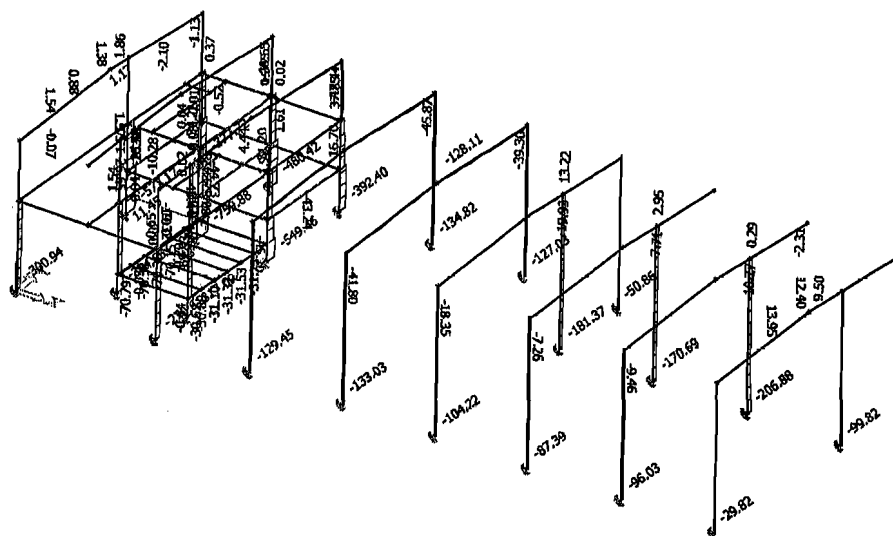
Rezne sile i pomaci (progibi) STALNO + KORISNO + SNIJEG



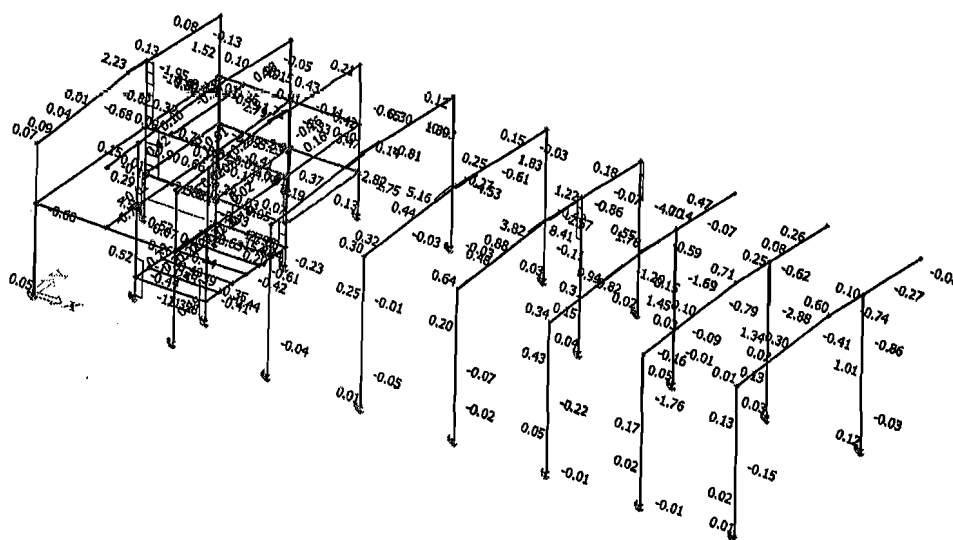
Dijagram momenata savijanja M_y [kNm]



Dijagram momenata savijanja M_z [kNm]



Dijagram uzdužnih sila N [kN]



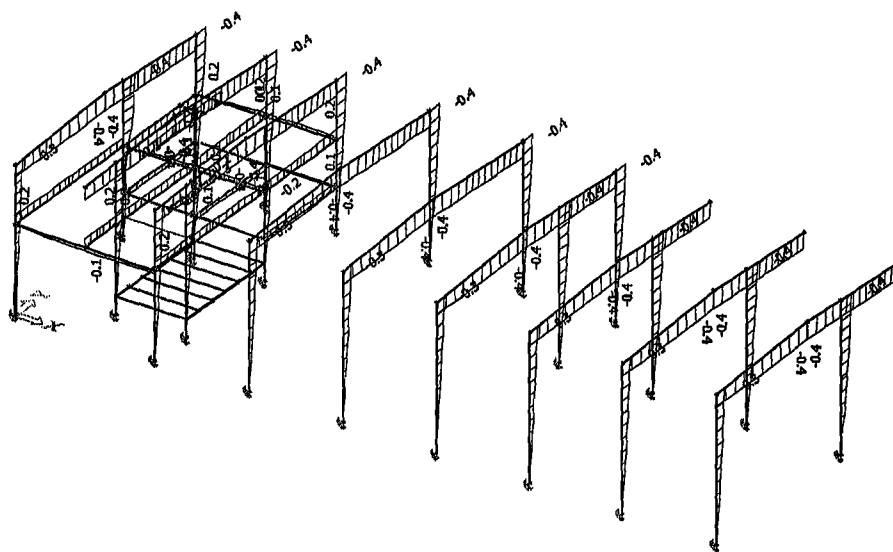
Dijagram poprečnih sila Ty [kN]



Dijagram poprečnih sila T_z [kN]

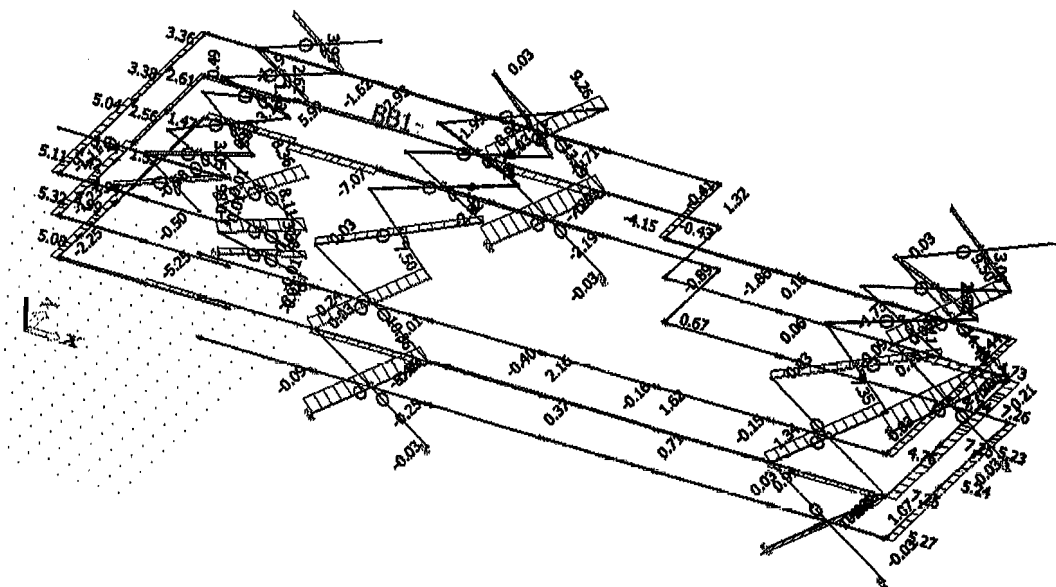


Dijagram progiba z [cm]

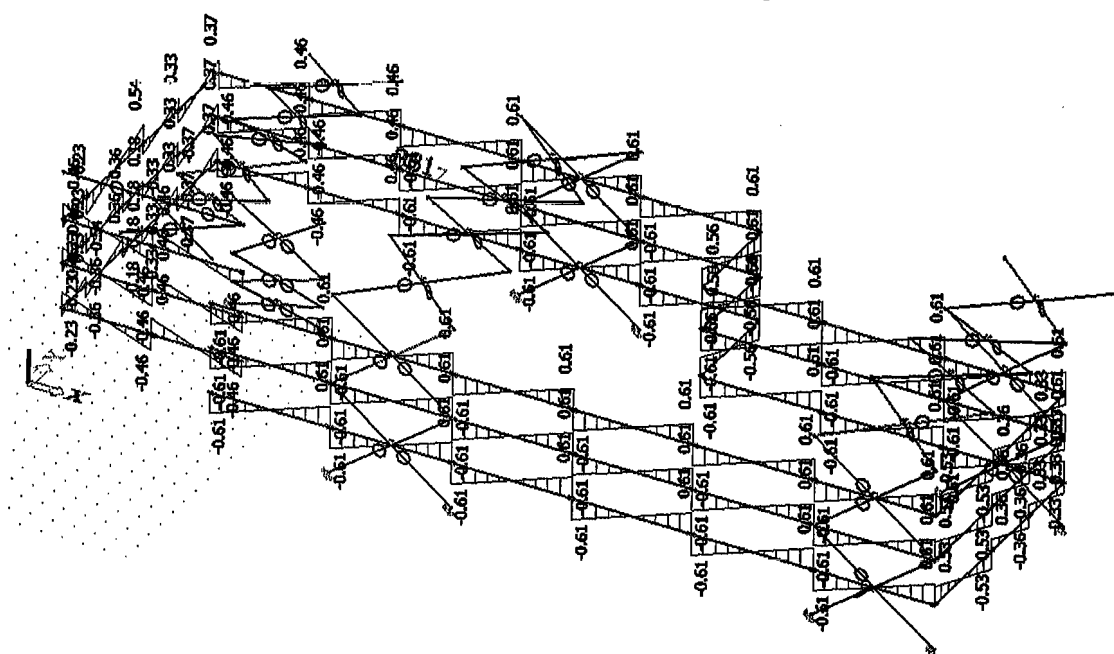


Dijagram progiba x [cm]

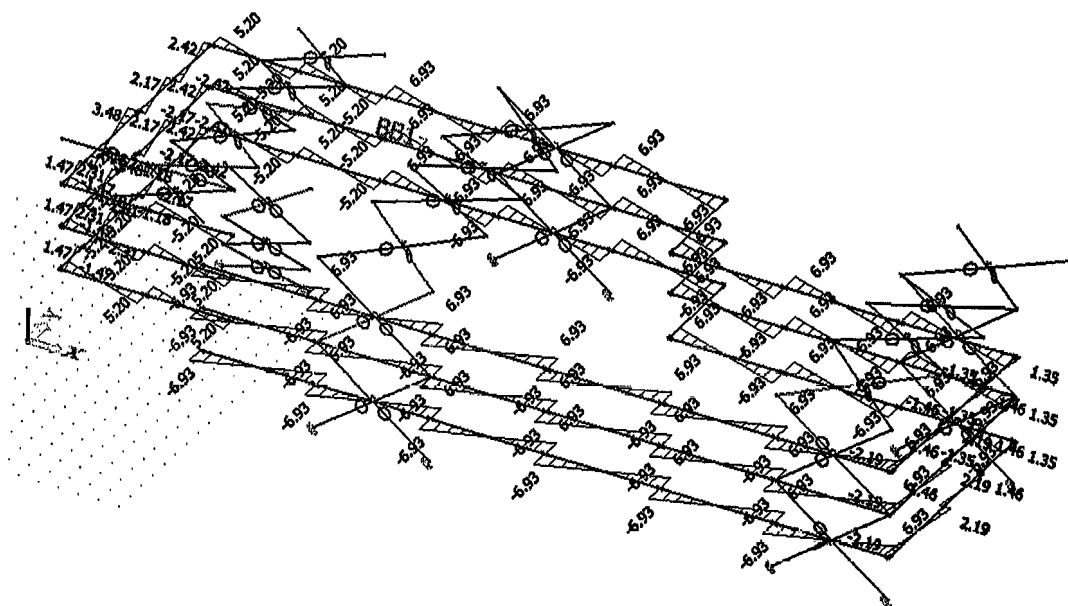
Dijagram momenata savijanja M_z [kNm]



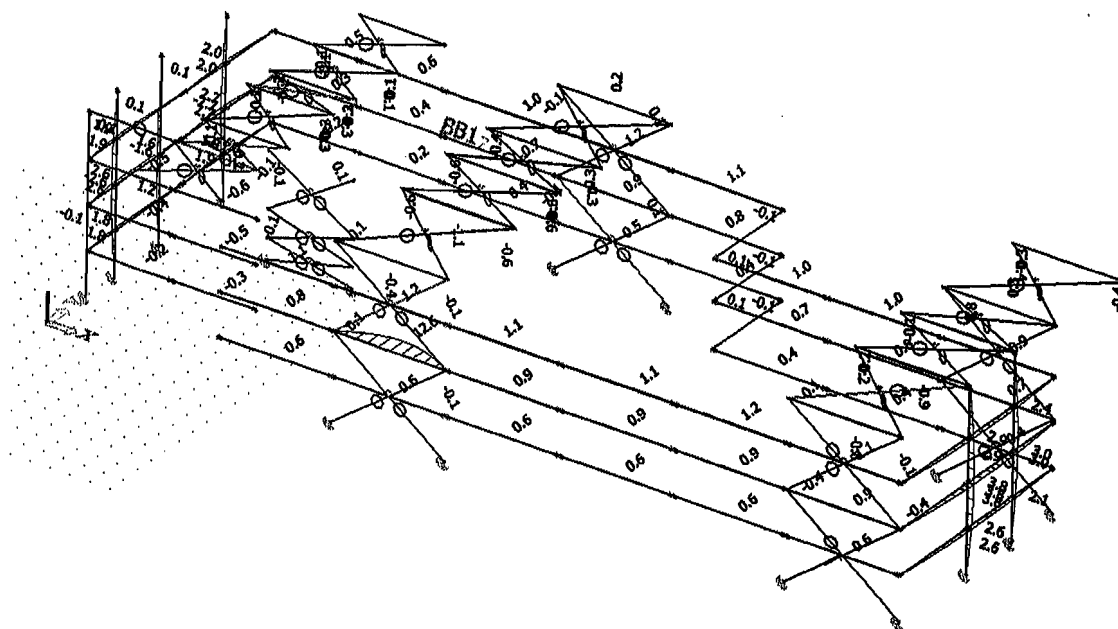
Dijagram uzdužnih sila N [kN]



Dijagram poprečnih sila Ty [kN]



Dijagram poprečnih sila T_z [kN]



Dijagram progiba y [cm]

DIMENZIONIRANJE ELEMENATA

U nastavku je dan prikaz dimenzioniranja elemenata. Detaljno je ispisan postupak dimenzioniranja najkritičnijih elemenata, a prikaz dimenzioniranja ostalih dan je tablično s konačnim stupnjom iskoristivosti pojedinog elementa obzirom na KGS (GSN).

Glavna konstrukcija

Check of steel – stupovi poz. 1 i poz 2

Nonlinear calculation, Extreme : Member

Selection : B29, B20

Nonlinear combinations : ST+KO+SN KGS

EN 1993-1-1 Code Check

National annex: Standard EN

Member B29	8.600 m	HEA320	S 235	ST+KO+SN KGS	0.73 -
------------	---------	--------	-------	--------------	--------

Partial safety factors	
Gamma M0 for resistance of cross-sections	1.00
Gamma M1 for resistance to instability	1.00
Gamma M2 for resistance of net sections	1.25

Material		
Yield strength fy	235.0	MPa
Ultimate strength fu	360.0	MPa
Fabrication	Rolled	

....SECTION CHECK:....

The critical check is on position 0.000 m

Internal forces	Calculated	Unit
N,Ed	-392.40	kN
Vy,Ed	0.13	kN
Vz,Ed	-10.75	kN
T,Ed	0.00	kNm
My,Ed	7.53	kNm
Mz,Ed	0.00	kNm

Classification for cross-section design

According to EN 1993-1-1 article 5.5.2

Classification of Internal Compression parts

According to EN 1993-1-1 Table 5.2 Sheet 1

Maximum width-to-thickness ratio	25.00
Class 1 Limit	33.00
Class 2 Limit	38.00
Class 3 Limit	45.11

=> Internal Compression parts Class 1

Classification of Outstand Flanges

According to EN 1993-1-1 Table 5.2 Sheet 2

Maximum width-to-thickness ratio	7.65
Class 1 Limit	9.00
Class 2 Limit	10.00
Class 3 Limit	13.77

=> Outstand Flanges Class 1

=> Section classified as Class 1 for cross-section design

Compression check

According to EN 1993-1-1 article 6.2.4 and formula (6.9)

A	1.2400e-02	m ²
N _{c,Rd}	2914.00	kN
Unity check	0.13	-

Bending moment check for M_y

According to EN 1993-1-1 article 6.2.5 and formula (6.12),(6.13)

W _{pl,y}	1.6292e-03	m ³
M _{pl,y,Rd}	382.85	kNm
Unity check	0.02	-

Shear check for V_y

According to EN 1993-1-1 article 6.2.6 and formula (6.17)

E _t	1.20	
A _v	9.6240e-03	m ²
V _{pl,y,Rd}	1305.76	kN
Unity check	0.00	-

Shear check for V_z

According to EN 1993-1-1 article 6.2.6 and formula (6.17)

E _t	1.20	
A _v	4.0765e-03	m ²
V _{pl,z,Rd}	553.09	kN
Unity check	0.02	-

Torsion check

According to EN 1993-1-1 article 6.2.7 and formula (6.23)

Tau _{t,Ed}	0.0	MPa
Tau _{Rd}	135.7	MPa
Unity check	0.00	-

Combined bending, axial force and shear force check

According to EN 1993-1-1 article 6.2.9.1 and formula (6.31)

MN _{y,Rd}	378.63	kNm
Unity check	0.02	-

The member satisfies the section check.

....**STABILITY CHECK**:....

Classification for member buckling design

Decisive position for stability classification: 0.000 m

Classification of Internal Compression parts

According to EN 1993-1-1 Table 5.2 Sheet 1

Maximum width-to-thickness ratio	25.00
Class 1 Limit	33.00
Class 2 Limit	38.00
Class 3 Limit	45.11

=> Internal Compression parts Class 1

Classification of Outstand Flanges

According to EN 1993-1-1 Table 5.2 Sheet 2

Maximum width-to-thickness ratio	7.65
Class 1 Limit	9.00
Class 2 Limit	10.00
Class 3 Limit	13.77

=> Outstand Flanges Class 1

=> Section classified as Class 1 for member buckling design

Flexural Buckling Check

According to EN 1993-1-1 article 6.3.1.1 and formula (6.46)

Buckling parameters	yy	zz	
Sway type	sway	non-sway	
System length L	8.500	8.500	m
Buckling factor k	2.00	1.00	
Buckling length L _{cr}	17.000	8.500	m
Critical Euler load N _{cr}	1642.32	2005.20	kN
Slenderness Lambda	125.10	113.21	
Relative slenderness Lambda _{rel}	1.33	1.21	
Limit slenderness Lambda _{rel,0}	0.20	0.20	
Buckling curve	b	c	
Imperfection Alpha	0.34	0.49	
Reduction factor Chi	0.41	0.43	
Buckling resistance N _b ,R _d	1199.85	1256.39	kN

Flexural Buckling verification		
Cross-section area A	1.2400e-02	m ²
Buckling resistance N _b ,R _d	1199.85	kN
Unity check	0.33	-

Torsional (-Flexural) Buckling check

According to article EN 1993-1-1 : 6.3.1.1. and formula (6.46)

Table of values		
Torsional Buckling length	8.500	m
N _{cr,T}	5418.53	kN

Ncr,TF	1642.32	kN
Relative slenderness Lambda,T	1.33	
Limit slenderness Lambda,0	0.20	
Buckling curve	c	
Imperfection Alpha	0.49	
A	1.2400e-02	m^2
Reduction factor Chi	0.38	
Buckling resistance Nb,Rd	1094.40	kN
Unity check	0.36	-

Lateral Torsional Buckling Check

According to article EN 1993-1-1 : 6.3.2.1. and formula (6.54)

LTB Parameters		
Method for LTB curve	Art. 6.3.2.2.	
Wy	1.6292e-03	m^3
Elastic critical moment Mcr	1640.97	kNm
Relative slenderness Lambda,LT	0.48	
Limit slenderness Lambda,LT,0	0.40	

Mcr Parameters		
LTB length	8.500	m
k	1.00	
kw	1.00	
C1	3.21	
C2	0.61	
C3	0.41	

The slenderness or bending moment is such that Lateral Torsional Buckling effects may be ignored according to EN 1993-1-1 article 6.3.2.2(4)

Compression and bending check

According to article EN 1993-1-1 : 6.3.3. and formula (6.61), (6.62)

Interaction Method 1

Table of values		
ky	1.053	
kz	0.714	
kzy	0.647	
kzz	0.850	
Delta My	0.00	kNm
Delta Mz	0.00	kNm
A	1.2400e-02	m^2
Wy	1.6292e-03	m^3
Wz	7.0833e-04	m^3
NRk	2914.00	kN
My,Rk	382.85	kNm
Mz,Rk	166.46	kNm
My,Ed	145.71	kNm

Mz,Ed	-0.97	kNm
Interaction Method 1		
Mcr0	511.77	kNm
reduced slenderness 0	0.86	
Psi y	0.052	
Psi z	0.000	
Cmy,0	0.794	
Cmz,0	0.767	
Cmy	0.925	
Cmz	0.767	
CmLT	1.000	
muy	0.844	
muz	0.878	
wy	1.101	
wz	1.500	
npl	0.135	
aLT	0.995	
bLT	0.001	
cLT	0.431	
dLT	0.002	
eLT	0.272	
Cyy	0.975	
Cyz	0.790	
Czy	0.848	
Czz	0.985	

Unity check (6.61) = $0.33 + 0.40 + 0.00 = 0.73$

Unity check (6.62) = $0.36 + 0.25 + 0.00 = 0.61$

Shear buckling check

in buckling field 1

According to article EN 1993-1-5 : 5. & 7.1. and formula (5.10) & (7.1)

Table of values	
hw/t	31.000

The web slenderness is such that the Shear Buckling Check is not required.

The member satisfies the stability check.

EN 1993-1-1 Code Check

National annex: Standard EN

Member B20	8.500 m	HEA360	S 235	ST+KO+SN KGS	0.54 -
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Partial safety factors	
Gamma M0 for resistance of cross-sections	1.00
Gamma M1 for resistance to instability	1.00
Gamma M2 for resistance of net sections	1.25

Material		
Yield strength f_y	235.0	MPa
Ultimate strength f_u	360.0	MPa
Fabrication	Rolled	

.....SECTION CHECK:.....

The critical check is on position 0.000 m

Internal forces	Calculated	Unit
N,Ed	-129.45	kN
Vy,Ed	0.00	kN
Vz,Ed	-38.27	kN
T,Ed	0.00	kNm
My,Ed	111.37	kNm
Mz,Ed	0.00	kNm

Classification for cross-section design

According to EN 1993-1-1 article 5.5.2

Classification of Internal Compression parts

According to EN 1993-1-1 Table 5.2 Sheet 1

Maximum width-to-thickness ratio	26.10
Class 1 Limit	57.63
Class 2 Limit	66.36
Class 3 Limit	92.75

=> Internal Compression parts Class 1

Classification of Outstand Flanges

According to EN 1993-1-1 Table 5.2 Sheet 2

Maximum width-to-thickness ratio	6.74
Class 1 Limit	9.00
Class 2 Limit	10.00
Class 3 Limit	13.77

=> Outstand Flanges Class 1

=> Section classified as Class 1 for cross-section design

Compression check

According to EN 1993-1-1 article 6.2.4 and formula (6.9)

A	1.4300e-02	m ²
Nc,Rd	3360.50	kN
Unity check	0.04	-

Bending moment check for My

According to EN 1993-1-1 article 6.2.5 and formula (6.12),(6.13)

Wpl,y	2.0875e-03	m ³
Mpl,y,Rd	490.56	kNm
Unity check	0.23	-

Shear check for Vy

According to EN 1993-1-1 article 6.2.6 and formula (6.17)

Eta	1.20	
Av	1.0870e-02	m ²
V _{pl,y} , Rd	1474.81	kN
Unity check	0.00	-

Shear check for Vz

According to EN 1993-1-1 article 6.2.6 and formula (6.17)

Eta	1.20	
Av	4.9200e-03	m ²
V _{pl,z} , Rd	667.53	kN
Unity check	0.06	-

Torsion check

According to EN 1993-1-1 article 6.2.7 and formula (6.23)

Tau, t, Ed	0.0	MPa
Tau, Rd	135.7	MPa
Unity check	0.00	-

Combined bending, axial force and shear force check

According to EN 1993-1-1 article 6.2.9.1 and formula (6.31)

M _{pl,y} , Rd	490.56	kNm
Unity check	0.23	-

Note: Since the shear forces are less than half the plastic shear resistances their effect on the moment

resistances is neglected.

Note: Since the axial force satisfies both criteria (6.33) and (6.34) of EN 1993-1-1 article 6.2.9.1(4)
its effect on the moment resistance about the y-y axis is neglected.

The member satisfies the section check.

....:STABILITY CHECK:....

Classification for member buckling design

Decisive position for stability classification: 0.000 m

Classification of Internal Compression parts

According to EN 1993-1-1 Table 5.2 Sheet 1

Maximum width-to-thickness ratio	26.10
Class 1 Limit	57.63
Class 2 Limit	66.36
Class 3 Limit	92.75

=> Internal Compression parts Class 1

Classification of Outstand Flanges

According to EN 1993-1-1 Table 5.2 Sheet 2

Maximum width-to-thickness ratio	6.74
Class 1 Limit	9.00
Class 2 Limit	10.00

Class 3 Limit	13.77
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=> Outstand Flanges Class 1

=> Section classified as Class 1 for member buckling design

Flexural Buckling Check

According to EN 1993-1-1 article 6.3.1.1 and formula (6.46)

Buckling parameters	yy	zz	
Sway type	sway	non-sway	
System length L	8.500	8.500	m
Buckling factor k	2.00	1.00	
Buckling length Lcr	17.000	8.500	m
Critical Euler load Ncr	2373.83	2263.38	kN
Slenderness Lambda	111.74	114.43	
Relative slenderness Lambda,rel	1.19	1.22	
Limit slenderness Lambda,rel,0	0.20	0.20	
Buckling curve	b	c	
Imperfection Alpha	0.34	0.49	
Reduction factor Chi	0.48	0.43	
Buckling resistance Nb,Rd	1625.43	1428.35	kN

Flexural Buckling verification		
Cross-section area A	1.4300e-02	m^2
Buckling resistance Nb,Rd	1428.35	kN
Unity check	0.09	-

Lateral Torsional Buckling Check

According to article EN 1993-1-1 : 6.3.2.1. and formula (6.54)

LTB Parameters		
Method for LTB curve	Art. 6.3.2.2.	
Wy	2.0875e-03	m^3
Elastic critical moment Mcr	1513.59	kNm
Relative slenderness Lambda,LT	0.57	
Limit slenderness Lambda,LT,0	0.40	

Mcr Parameters		
LTB length	8.500	m
k	1.00	
kw	1.00	
C1	2.35	
C2	0.00	
C3	1.00	

The slenderness or bending moment is such that Lateral Torsional Buckling effects may be ignored according to EN 1993-1-1 article 6.3.2.2(4)

Compression and bending check

According to article EN 1993-1-1 : 6.3.3. and formula (6.61), (6.62)

Interaction Method 1

Table of values		
kyy	1.047	
kyz	0.904	
kzy	0.556	
kzz	0.994	
Delta My	0.00	kNm
Delta Mz	0.00	kNm
A	1.4300e-02	m ²
Wy	2.0875e-03	m ³
Wz	8.0417e-04	m ³
NRk	3360.50	kN
My,Rk	490.56	kNm
Mz,Rk	188.98	kNm
My,Ed	-213.89	kNm
Mz,Ed	0.34	kNm
Interaction Method 1		
Mcr0	643.20	kNm
reduced slenderness 0	0.87	
Psi y	-0.521	
Psi z	0.000	
Cmy,0	0.964	
Cmz,0	0.953	
Cmy	0.992	
Cmz	0.953	
CmLT	1.020	
muy	0.971	
muz	0.966	
wy	1.104	
wz	1.500	
npl	0.039	
aLT	0.995	
bLT	0.000	
cLT	0.463	
dLT	0.001	
eLT	0.282	
Cyy	0.993	
Cyz	0.759	
Czy	0.958	
Czz	0.983	

Unity check (6.61) = 0.08 + 0.46 + 0.00 = 0.54

Unity check (6.62) = 0.09 + 0.24 + 0.00 = 0.33

Shear buckling check

in buckling field 1

According to article EN 1993-1-5 : 5. & 7.1. and formula (5.10) & (7.1)

Table of values	
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hw/t	31.500
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The web slenderness is such that the Shear Buckling Check is not required.
The member satisfies the stability check.

Check of steel – grede: poz. 4

Nonlinear calculation, Extreme : Member
Selection : B22, B23
Nonlinear combinations : ST+KO+SN KGS

EN 1993-1-1 Code Check

National annex: Standard EN

Member B22	8.110 m	HEA360	S 235	ST+KO+SN KGS	0.54 -
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Partial safety factors	
Gamma M0 for resistance of cross-sections	1.00
Gamma M1 for resistance to instability	1.00
Gamma M2 for resistance of net sections	1.25

Material		
Yield strength fy	235.0	MPa
Ultimate strength fu	360.0	MPa
Fabrication	Rolled	

....SECTION CHECK:....

The critical check is on position 0.000 m

Internal forces	Calculated	Unit
N,Ed	-41.80	kN
Vy,Ed	0.30	kN
Vz,Ed	92.43	kN
T,Ed	0.09	kNm
My,Ed	-207.24	kNm
Mz,Ed	-0.36	kNm

Classification for cross-section design

According to EN 1993-1-1 article 5.5.2

Classification of Internal Compression parts

According to EN 1993-1-1 Table 5.2 Sheet 1

Maximum width-to-thickness ratio	26.10
Class 1 Limit	66.63
Class 2 Limit	76.73
Class 3 Limit	115.77

=> Internal Compression parts Class 1
Classification of Outstand Flanges

According to EN 1993-1-1 Table 5.2 Sheet 2

Maximum width-to-thickness ratio	6.74
Class 1 Limit	9.00
Class 2 Limit	10.00
Class 3 Limit	13.78

=> Outstand Flanges Class 1

=> Section classified as Class 1 for cross-section design

Compression check

According to EN 1993-1-1 article 6.2.4 and formula (6.9)

A	1.4300e-02	m ²
N _{c,Rd}	3360.50	kN
Unity check	0.01	-

Bending moment check for M_y

According to EN 1993-1-1 article 6.2.5 and formula (6.12),(6.13)

W _{pl,y}	2.0875e-03	m ³
M _{pl,y,Rd}	490.56	kNm
Unity check	0.42	-

Bending moment check for M_z

According to EN 1993-1-1 article 6.2.5 and formula (6.12),(6.13)

W _{pl,z}	8.0417e-04	m ³
M _{pl,z,Rd}	188.98	kNm
Unity check	0.00	-

Shear check for V_y

According to EN 1993-1-1 article 6.2.6 and formula (6.17)

Eta	1.20	
A _v	1.0870e-02	m ²
V _{pl,y,Rd}	1474.81	kN
Unity check	0.00	-

Shear check for V_z

According to EN 1993-1-1 article 6.2.6 and formula (6.17)

Eta	1.20	
A _v	4.9200e-03	m ²
V _{pl,z,Rd}	667.53	kN
Unity check	0.14	-

Torsion check

According to EN 1993-1-1 article 6.2.7 and formula (6.23)

Tau _{t,Ed}	1.1	MPa
Tau _{Rd}	135.7	MPa
Unity check	0.01	-

Combined bending, axial force and shear force check

According to EN 1993-1-1 article 6.2.9.1 and formula (6.41)

Mpl,y,Rd	490.56	kNm
Alpha	2.00	
Mpl,z,Rd	188.98	kNm
Beta	1.00	

Unity check (6.41) = 0.18 + 0.00 = 0.18 -

Note: Since the shear forces are less than half the plastic shear resistances their effect on the moment

resistances is neglected.

Note: Since the axial force satisfies both criteria (6.33) and (6.34) of EN 1993-1-1 article 6.2.9.1(4) its effect on the moment resistance about the y-y axis is neglected.

Note: Since the axial force satisfies criteria (6.35) of EN 1993-1-1 article 6.2.9.1(4) its effect on the moment

resistance about the z-z axis is neglected.

The member satisfies the section check.

.....STABILITY CHECK:.....

Classification for member buckling design

Decisive position for stability classification: 0.000 m

Classification of Internal Compression parts

According to EN 1993-1-1 Table 5.2 Sheet 1

Maximum width-to-thickness ratio	26.10
Class 1 Limit	66.63
Class 2 Limit	76.73
Class 3 Limit	115.77

=> Internal Compression parts Class 1

Classification of Outstand Flanges

According to EN 1993-1-1 Table 5.2 Sheet 2

Maximum width-to-thickness ratio	6.74
Class 1 Limit	9.00
Class 2 Limit	10.00
Class 3 Limit	13.78

=> Outstand Flanges Class 1

=> Section classified as Class 1 for member buckling design

Flexural Buckling Check

According to EN 1993-1-1 article 6.3.1.1 and formula (6.46)

Buckling parameters	yy	zz	
Sway type	sway	non-sway	
System length L	8.110	8.110	m
Buckling factor k	2.21	0.86	
Buckling length Lcr	17.905	6.987	m
Critical Euler load Ncr	2139.99	3349.54	kN
Slenderness Lambda	117.69	94.07	
Relative slenderness Lambda,rel	1.25	1.00	

Limit slenderness $\Lambda_{rel,0}$	0.20	0.20	
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Note: The slenderness or compression force is such that Flexural Buckling effects may be ignored

according to EN 1993-1-1 article 6.3.1.2(4).

Lateral Torsional Buckling Check

According to article EN 1993-1-1 : 6.3.2.1. and formula (6.54)

LTB Parameters		
Method for LTB curve	Art. 6.3.2.2.	
Wy	2.0875e-03	m ³
Elastic critical moment M _{cr}	1257.32	kNm
Relative slenderness Λ_{LT}	0.62	
Limit slenderness $\Lambda_{LT,0}$	0.40	
LTB curve	a	
Imperfection α_{LT}	0.21	
Reduction factor χ_{LT}	0.88	
Buckling resistance M _{b,Rd}	431.90	kNm
Unity check	0.48	-

Mcr Parameters		
LTB length	8.110	m
k	1.00	
k _w	1.00	
C ₁	1.83	
C ₂	0.34	
C ₃	1.00	

Note: C Parameters according to ECCS 119 2006 / Galea 2002

load in center of gravity

Compression and bending check

According to article EN 1993-1-1 : 6.3.3. and formula (6.61), (6.62)

Interaction Method 1

Table of values		
k _{yy}	1.024	
k _{yz}	0.770	
k _{zy}	0.536	
k _{zz}	0.791	
Delta M _y	0.00	kNm
Delta M _z	0.00	kNm
A	1.4300e-02	m ²
Wy	2.0875e-03	m ³
Wz	8.0417e-04	m ³
NR _k	3360.50	kN
M _{y,Rk}	490.56	kNm
M _{z,Rk}	188.98	kNm
M _{y,Ed}	-207.24	kNm
M _{z,Ed}	8.13	kNm
Interaction Method 1		

Mcr0	685.38	kNm
reduced slenderness 0	0.85	
Psi y	-0.843	
Psi z	-0.044	
Cmy,0	0.990	
Cmz,0	0.779	
Cmy	0.999	
Cmz	0.779	
CmLT	1.002	
muy	1.000	
muz	1.000	
wy	1.104	
wz	1.500	
npl	0.012	
aLT	0.995	
bLT	0.007	
cLT	0.570	
dLT	0.040	
eLT	0.622	
Cyy	0.997	
Cyz	0.717	
Czy	0.981	
Czz	0.997	

Unity check (6.61) = $0.01 + 0.49 + 0.03 = 0.54$

Unity check (6.62) = $0.01 + 0.26 + 0.03 = 0.30$

Shear buckling check

in buckling field 1

According to article EN 1993-1-5 : 5. & 7.1. and formula (5.10) & (7.1)

Table of values	
hw/t	31.500

The web slenderness is such that the Shear Buckling Check is not required.

The member satisfies the stability check.

EN 1993-1-1 Code Check

National annex: Standard EN

Member B23	8.110 m	HEA360	S 235	ST+KO+SN KGS	0.46 -
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Partial safety factors	
Gamma M0 for resistance of cross-sections	1.00
Gamma M1 for resistance to instability	1.00
Gamma M2 for resistance of net sections	1.25

Material		
Yield strength fy	235.0	MPa

Ultimate strength f_u	360.0	MPa
Fabrication	Rolled	

.....SECTION CHECK:.....

The critical check is on position 8.110 m

Internal forces	Calculated	Unit
N_{Ed}	-39.30	kN
$V_{y,Ed}$	-0.03	kN
$V_{z,Ed}$	-82.29	kN
T_{Ed}	-0.17	kNm
$M_{y,Ed}$	-196.30	kNm
$M_{z,Ed}$	0.15	kNm

Classification for cross-section design

According to EN 1993-1-1 article 5.5.2

Classification of Internal Compression parts

According to EN 1993-1-1 Table 5.2 Sheet 1

Maximum width-to-thickness ratio	26.10
Class 1 Limit	66.93
Class 2 Limit	77.07
Class 3 Limit	115.82

=> Internal Compression parts Class 1

Classification of Outstand Flanges

According to EN 1993-1-1 Table 5.2 Sheet 2

Maximum width-to-thickness ratio	6.74
Class 1 Limit	9.00
Class 2 Limit	10.00
Class 3 Limit	13.77

=> Outstand Flanges Class 1

=> Section classified as Class 1 for cross-section design

Compression check

According to EN 1993-1-1 article 6.2.4 and formula (6.9)

A	1.4300e-02	m ²
$N_{c,Rd}$	3360.50	kN
Unity check	0.01	-

Bending moment check for M_y

According to EN 1993-1-1 article 6.2.5 and formula (6.12),(6.13)

$W_{pl,y}$	2.0875e-03	m ³
$M_{pl,y,Rd}$	490.56	kNm
Unity check	0.40	-

Bending moment check for M_z

According to EN 1993-1-1 article 6.2.5 and formula (6.12),(6.13)

$W_{pl,z}$	8.0417e-04	m ³
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Mpl,z,Rd	188.98	kNm
Unity check	0.00	-

Shear check for Vy

According to EN 1993-1-1 article 6.2.6 and formula (6.17)

Eta	1.20	
Av	1.0870e-02	m^2
Vpl,y,Rd	1474.81	kN
Unity check	0.00	-

Shear check for Vz

According to EN 1993-1-1 article 6.2.6 and formula (6.17)

Eta	1.20	
Av	4.9200e-03	m^2
Vpl,z,Rd	667.53	kN
Unity check	0.12	-

Torsion check

According to EN 1993-1-1 article 6.2.7 and formula (6.23)

Tau,t,Ed	2.0	MPa
Tau,Rd	135.7	MPa
Unity check	0.01	-

Combined bending, axial force and shear force check

According to EN 1993-1-1 article 6.2.9.1 and formula (6.41)

Mpl,y,Rd	490.56	kNm
Alpha	2.00	
Mpl,z,Rd	188.98	kNm
Beta	1.00	

Unity check (6.41) = 0.16 + 0.00 = 0.16 -

Note: Since the shear forces are less than half the plastic shear resistances their effect on the moment

resistances is neglected.

Note: Since the axial force satisfies both criteria (6.33) and (6.34) of EN 1993-1-1 article 6.2.9.1(4) its effect on the moment resistance about the y-y axis is neglected.

Note: Since the axial force satisfies criteria (6.35) of EN 1993-1-1 article 6.2.9.1(4) its effect on the moment

resistance about the z-z axis is neglected.

The member satisfies the section check.

.....STABILITY CHECK:.....

Classification for member buckling design

Decisive position for stability classification: 0.000 m

Classification of Internal Compression parts

According to EN 1993-1-1 Table 5.2 Sheet 1

Maximum width-to-thickness ratio	26.10
Class 1 Limit	67.55
Class 2 Limit	77.78
Class 3 Limit	115.98

=> Internal Compression parts Class 1

Classification of Outstand Flanges

According to EN 1993-1-1 Table 5.2 Sheet 2

Maximum width-to-thickness ratio	6.74
Class 1 Limit	9.00
Class 2 Limit	10.00
Class 3 Limit	13.95

=> Outstand Flanges Class 1

=> Section classified as Class 1 for member buckling design

Flexural Buckling Check

According to EN 1993-1-1 article 6.3.1.1 and formula (6.46)

Buckling parameters	yy	zz	
Sway type	sway	non-sway	
System length L	8.110	8.110	m
Buckling factor k	2.10	0.85	
Buckling length Lcr	17.000	6.915	m
Critical Euler load Ncr	2373.82	3420.35	kN
Slenderness Lambda	111.74	93.09	
Relative slenderness Lambda,rel	1.19	0.99	
Limit slenderness Lambda,rel,0	0.20	0.20	

Note: The slenderness or compression force is such that Flexural Buckling effects may be ignored

according to EN 1993-1-1 article 6.3.1.2(4).

Lateral Torsional Buckling Check

According to article EN 1993-1-1 : 6.3.2.1. and formula (6.54)

LTB Parameters		
Method for LTB curve	Art. 6.3.2.2.	
Wy	2.0875e-03	m ³
Elastic critical moment Mcr	1319.99	kNm
Relative slenderness Lambda,LT	0.61	
Limit slenderness Lambda,LT,0	0.40	

Mcr Parameters		
LTB length	8.110	m
k	1.00	
kw	1.00	
C1	1.93	
C2	0.30	
C3	1.00	

The slenderness or bending moment is such that Lateral Torsional Buckling effects may be ignored according to EN 1993-1-1 article 6.3.2.2(4)

Compression and bending check

According to article EN 1993-1-1 : 6.3.3. and formula (6.61), (6.62)

Interaction Method 1

Table of values		
kyy	1.020	
kyz	0.924	
kzy	0.532	
kzz	1.008	
Delta My	0.00	kNm
Delta Mz	0.00	kNm
A	1.4300e-02	m ²
Wy	2.0875e-03	m ³
Wz	8.0417e-04	m ³
NRk	3360.50	kN
My,Rk	490.56	kNm
Mz,Rk	188.98	kNm
My,Ed	-196.30	kNm
Mz,Ed	7.91	kNm
Interaction Method 1		
Mcr0	685.38	kNm
reduced slenderness 0	0.85	
Psi y	-0.890	
Psi z	0.019	
Cmy,0	0.991	
Cmz,0	0.989	
Cmy	0.999	
Cmz	0.989	
CmLT	1.002	
muy	1.000	
muz	1.000	
wy	1.104	
wz	1.500	
npl	0.012	
aLT	0.995	
bLT	0.006	
cLT	0.479	
dLT	0.027	
eLT	0.538	
Cyy	0.997	
Cyz	0.757	
Czy	0.985	

Czz	0.993	
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Unity check (6.61) = $0.01 + 0.41 + 0.04 = 0.46$

Unity check (6.62) = $0.01 + 0.21 + 0.04 = 0.27$

Shear buckling check

in buckling field 1

According to article EN 1993-1-5 : 5. & 7.1. and formula (5.10) & (7.1)

Table of values	
hw/t	31.500

The web slenderness is such that the Shear Buckling Check is not required.

The member satisfies the stability check.

Check of steel – Glavna konstrukcija

Nonlinear calculation, Extreme : Member

Selection : All

Nonlinear combinations : ST+KO+SN KGS

Case	Member	css	mat	dx [m]	un.che ck [-]	sec.che ck [-]	stab.che ck [-]
ST+KO+SN KGS	B1	STUP 1 - HEA360	S 235	0.000	0.06	0.04	0.06
ST+KO+SN KGS	B2	STUP 1 - HEA360	S 235	0.000	0.12	0.04	0.12
ST+KO+SN KGS	B3	STUP 1 - HEA360	S 235	0.000	0.19	0.06	0.19
ST+KO+SN KGS	B4	STUP 1 - HEA360	S 235	0.000	0.18	0.03	0.18
ST+KO+SN KGS	B5	STUP 1 - HEA360	S 235	0.000	0.18	0.06	0.18
ST+KO+SN KGS	B6	STUP 1 - HEA360	S 235	0.000	0.12	0.03	0.12
ST+KO+SN KGS	B7	STUP 1 - HEA360	S 235	0.000	0.36	0.06	0.36
ST+KO+SN KGS	B8	GREDA 1 - HEA360	S 235	1.622	0.06	0.06	0.00
ST+KO+SN KGS	B9	GREDA 1 - HEA360	S 235	3.004	0.21	0.21	0.00
ST+KO+SN KGS	B10	GREDA 1 - HEA360	S 235	0.000	0.20	0.11	0.20
ST+KO+SN KGS	B11	GREDA 1 - HEA360	S 235	3.004	0.61	0.48	0.61
ST+KO+SN KGS	B12	GREDA 1 - HEA360	S 235	0.000	0.18	0.09	0.18
ST+KO+SN KGS	B13	GREDA 1 - HEA360	S 235	3.004	0.49	0.40	0.49
ST+KO+SN KGS	B14	GREDA 1 - HEA360	S 235	0.000	0.25	0.18	0.25
ST+KO+SN KGS	B15	GREDA 1 - HEA360	S 235	3.004	0.45	0.35	0.45
ST+KO+SN KGS	B16	STUP 1 - HEA360	S 235	0.000	0.24	0.08	0.24
ST+KO+SN KGS	B17	STUP 1 - HEA360	S 235	0.000	0.11	0.06	0.11
ST+KO+SN KGS	B18	STUP 1 - HEA360	S 235	0.000	0.53	0.22	0.53
ST+KO+SN KGS	B19	STUP 1 - HEA360	S 235	0.000	0.51	0.21	0.51
ST+KO+SN KGS	B20	STUP 1 - HEA360	S 235	0.000	0.54	0.23	0.54
ST+KO+SN KGS	B21	STUP 1 - HEA360	S 235	0.000	0.57	0.24	0.57
ST+KO+SN KGS	B22	GREDA 1 - HEA360	S 235	0.000	0.54	0.42	0.54
ST+KO+SN KGS	B23	GREDA 1 - HEA360	S 235	8.110	0.46	0.40	0.46
ST+KO+SN KGS	B24	GREDA 1 - HEA360	S 235	0.000	0.53	0.44	0.53

ST+KO+SN KGS	B25	GREDA 1 - HEA360	S 235	8.110	0.57	0.46	0.57
ST+KO+SN KGS	B26	STUP3 - HEA320	S 235	0.000	0.34	0.09	0.34
ST+KO+SN KGS	B27	STUP3 - HEA320	S 235	0.000	0.52	0.11	0.52
ST+KO+SN KGS	B29	STUP3 - HEA320	S 235	0.000	0.73	0.13	0.73
ST+KO+SN KGS	B30	GREDA 1 - HEA360	S 235	2.803	0.59	0.46	0.59
ST+KO+SN KGS	B31	GREDA 1 - HEA360	S 235	8.110	0.34	0.30	0.34
ST+KO+SN KGS	B32	STUP3 - HEA320	S 235	0.000	0.62	0.13	0.62
ST+KO+SN KGS	B34	STUP3 - HEA320	S 235	0.000	0.63	0.16	0.63
ST+KO+SN KGS	B35	STUP3 - HEA320	S 235	0.000	0.29	0.10	0.29
ST+KO+SN KGS	B36	STUP3 - HEA320	S 235	0.000	0.79	0.20	0.79
ST+KO+SN KGS	B37	STUP3 - HEA320	S 235	0.000	0.36	0.13	0.36
ST+KO+SN KGS	B38	GREDA 1 - HEA360	S 235	2.803	0.55	0.39	0.55
ST+KO+SN KGS	B39	GREDA 1 - HEA360	S 235	8.110	0.17	0.16	0.17
ST+KO+SN KGS	B40	GREDA 1 - HEA360	S 235	3.244	0.04	0.02	0.04
ST+KO+SN KGS	B41	GREDA 1 - HEA360	S 235	4.706	0.05	0.04	0.05
ST+KO+SN KGS	B323	STUP3 - HEA320	S 235	0.000	0.72	0.26	0.72
ST+KO+SN KGS	B324	GREDA 2 - HEA360	S 235	3.475	0.42	0.42	0.00
ST+KO+SN KGS	B325	GREDA 2 - HEA360	S 235	0.000	0.07	0.07	0.00

Međukatna konstrukcija i konstrukcija nadstrešnice

Check of steel – grede poz. 7 i poz. 10

Nonlinear calculation, Extreme : Member

Selection : B319

Nonlinear combinations : ST+KO+SN KGS

EN 1993-1-1 Code Check

National annex: Standard EN

Member B319	8.400 m	HEA360	S 235	ST+KO+SN KGS	0.94 -
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Partial safety factors	
Gamma M0 for resistance of cross-sections	1.00
Gamma M1 for resistance to instability	1.00
Gamma M2 for resistance of net sections	1.25

Material		
Yield strength fy	235.0	MPa
Ultimate strength fu	360.0	MPa
Fabrication	Rolled	

....SECTION CHECK:....

The critical check is on position 4.200 m

Internal forces	Calculated	Unit
N,Ed	-15.57	kN
Vy,Ed	1.53	kN
Vz,Ed	67.08	kN
T,Ed	0.42	kNm
My,Ed	444.89	kNm
Mz,Ed	6.41	kNm

Classification for cross-section design

According to EN 1993-1-1 article 5.5.2

Classification of Internal Compression parts

According to EN 1993-1-1 Table 5.2 Sheet 1

Maximum width-to-thickness ratio	26.10
Class 1 Limit	69.90
Class 2 Limit	80.49
Class 3 Limit	122.07

=> Internal Compression parts Class 1

Classification of Outstand Flanges

According to EN 1993-1-1 Table 5.2 Sheet 2

Maximum width-to-thickness ratio	6.74
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Class 1 Limit	9.00
Class 2 Limit	10.00
Class 3 Limit	13.83

=> Outstand Flanges Class 1

=> Section classified as Class 1 for cross-section design

Compression check

According to EN 1993-1-1 article 6.2.4 and formula (6.9)

A	1.4300e-02	m ²
N _{c,Rd}	3360.50	kN
Unity check	0.00	-

Bending moment check for M_y

According to EN 1993-1-1 article 6.2.5 and formula (6.12),(6.13)

W _{pl,y}	2.0875e-03	m ³
M _{pl,y,Rd}	490.56	kNm
Unity check	0.91	-

Bending moment check for M_z

According to EN 1993-1-1 article 6.2.5 and formula (6.12),(6.13)

W _{pl,z}	8.0417e-04	m ³
M _{pl,z,Rd}	188.98	kNm
Unity check	0.03	-

Shear check for V_y

According to EN 1993-1-1 article 6.2.6 and formula (6.17)

E _t	1.20	
A _v	1.0870e-02	m ²
V _{pl,y,Rd}	1474.81	kN
Unity check	0.00	-

Shear check for V_z

According to EN 1993-1-1 article 6.2.6 and formula (6.17)

E _t	1.20	
A _v	4.9200e-03	m ²
V _{pl,z,Rd}	667.53	kN
Unity check	0.10	-

Torsion check

According to EN 1993-1-1 article 6.2.7 and formula (6.23)

Tau _{t,Ed}	4.9	MPa
Tau _{Rd}	135.7	MPa
Unity check	0.04	-

Note: The unity check for torsion is lower than the limit value of 0.05. Therefore torsion is considered as

insignificant and is ignored in the combined checks.

Combined bending, axial force and shear force check

According to EN 1993-1-1 article 6.2.9.1 and formula (6.41)

Mpl,y,Rd	490.56	kNm
Alpha	2.00	
Mpl,z,Rd	188.98	kNm
Beta	1.00	

Unity check (6.41) = 0.82 + 0.03 = 0.86 -

Note: Since the shear forces are less than half the plastic shear resistances their effect on the moment

resistances is neglected.

The member satisfies the section check.

.....STABILITY CHECK:.....

Classification for member buckling design

Decisive position for stability classification: 0.000 m

Classification of Internal Compression parts

According to EN 1993-1-1 Table 5.2 Sheet 1

Maximum width-to-thickness ratio	26.10
Class 1 Limit	33.00
Class 2 Limit	38.00
Class 3 Limit	42.00

=> Internal Compression parts Class 1

Classification of Outstand Flanges

According to EN 1993-1-1 Table 5.2 Sheet 2

Maximum width-to-thickness ratio	6.74
Class 1 Limit	9.00
Class 2 Limit	10.00
Class 3 Limit	14.00

=> Outstand Flanges Class 1

=> Section classified as Class 1 for member buckling design

Flexural Buckling Check

According to EN 1993-1-1 article 6.3.1.1 and formula (6.46)

Buckling parameters	yy	zz	
Sway type	sway	non-sway	
System length L	8.400	8.400	m
Buckling factor k	1.00	1.00	
Buckling length Lcr	8.400	8.400	m
Critical Euler load Ncr	9722.74	2317.71	kN
Slenderness Lambda	55.21	113.08	
Relative slenderness Lambda,rel	0.59	1.20	
Limit slenderness Lambda,rel,0	0.20	0.20	

Note: The slenderness or compression force is such that Flexural Buckling effects may be ignored

according to EN 1993-1-1 article 6.3.1.2(4).

Lateral Torsional Buckling Check

According to article EN 1993-1-1 : 6.3.2.1. and formula (6.54)

LTB Parameters		
Method for LTB curve	Art. 6.3.2.2.	
Wy	2.0875e-03	m ³
Elastic critical moment M _{cr}	52375.15	kNm
Relative slenderness Lambda,LT	0.10	
Limit slenderness Lambda,LT,0	0.40	

Mcr Parameters		
LTB length	0.840	m
k	1.00	
kw	1.00	
C1	1.35	
C2	0.63	
C3	0.41	

The slenderness or bending moment is such that Lateral Torsional Buckling effects may be ignored according to EN 1993-1-1 article 6.3.2.2(4)

Compression and bending check

According to article EN 1993-1-1 : 6.3.3. and formula (6.61), (6.62)

Interaction Method 1

Table of values		
k _{yy}	1.003	
k _{yz}	0.710	
k _{zy}	0.518	
k _{zz}	1.008	
Delta M _y	0.00	kNm
Delta M _z	0.00	kNm
A	1.4300e-02	m ²
Wy	2.0875e-03	m ³
Wz	8.0417e-04	m ³
NR _k	3360.50	kN
M _y ,R _k	490.56	kNm
M _z ,R _k	188.98	kNm
M _y ,E _d	444.91	kNm
M _z ,E _d	6.41	kNm
Interaction Method 1		
M _{cr0}	38853.97	kNm
reduced slenderness 0	0.11	
Psi _y	1.000	

Psi z	1.000	
Cmy,0	1.000	
Cmz,0	0.999	
Cmy	1.000	
Cmz	0.999	
CmLT	1.000	
muy	1.000	
muz	1.000	
wy	1.104	
wz	1.500	
npl	0.005	
aLT	0.995	
bLT	0.000	
cLT	0.016	
dLT	0.003	
eLT	0.078	
Cyy	0.999	
Cyz	0.990	
Czy	0.995	
Czz	0.998	

Unity check (6.61) = 0.00 + 0.91 + 0.02 = 0.94

Unity check (6.62) = 0.00 + 0.47 + 0.03 = 0.51

Shear buckling check

in buckling field 1

According to article EN 1993-1-5 : 5. & 7.1. and formula (5.10) & (7.1)

Table of values	
hw/t	31.500

The web slenderness is such that the Shear Buckling Check is not required.

The member satisfies the stability check.

Check of steel – grede poz. 8, 9 i 11

Nonlinear calculation, Extreme : Member

Selection : B350

Nonlinear combinations : ST+KO+SN KGS

EN 1993-1-1 Code Check

National annex: Standard EN

Member B350	9.750 m	HEA200	S 235	ST+KO+SN KGS	0.96 -
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Partial safety factors	
Gamma M0 for resistance of cross-sections	1.00
Gamma M1 for resistance to instability	1.00
Gamma M2 for resistance of net sections	1.25

Material		
Yield strength f_y	235.0	MPa
Ultimate strength f_u	360.0	MPa
Fabrication	Rolled	

.....SECTION CHECK:....

The critical check is on position 4.875 m

Internal forces	Calculated	Unit
N,Ed	-3.05	kN
Vy,Ed	0.06	kN
Vz,Ed	0.51	kN
T,Ed	-0.01	kNm
My,Ed	84.39	kNm
Mz,Ed	-0.31	kNm

Classification for cross-section design

According to EN 1993-1-1 article 5.5.2

Classification of Internal Compression parts

According to EN 1993-1-1 Table 5.2 Sheet 1

Maximum width-to-thickness ratio	20.62
Class 1 Limit	70.75
Class 2 Limit	81.47
Class 3 Limit	122.65

=> Internal Compression parts Class 1

Classification of Outstand Flanges

According to EN 1993-1-1 Table 5.2 Sheet 2

Maximum width-to-thickness ratio	7.88
Class 1 Limit	9.00
Class 2 Limit	10.00
Class 3 Limit	13.78

=> Outstand Flanges Class 1

=> Section classified as Class 1 for cross-section design

Compression check

According to EN 1993-1-1 article 6.2.4 and formula (6.9)

A	5.3800e-03	m ²
Nc,Rd	1264.30	kN
Unity check	0.00	-

Bending moment check for My

According to EN 1993-1-1 article 6.2.5 and formula (6.12),(6.13)

Wpl,y	4.2917e-04	m ³
Mpl,y,Rd	100.85	kNm
Unity check	0.84	-

Bending moment check for M_z

According to EN 1993-1-1 article 6.2.5 and formula (6.12),(6.13)

$W_{pl,z}$	2.0375e-04	m^3
$M_{pl,z,Rd}$	47.88	kNm
Unity check	0.01	-

Shear check for V_y

According to EN 1993-1-1 article 6.2.6 and formula (6.17)

η	1.20	
A_v	4.1592e-03	m^2
$V_{pl,y,Rd}$	564.32	kN
Unity check	0.00	-

Shear check for V_z

According to EN 1993-1-1 article 6.2.6 and formula (6.17)

η	1.20	
A_v	1.8050e-03	m^2
$V_{pl,z,Rd}$	244.90	kN
Unity check	0.00	-

Torsion check

According to EN 1993-1-1 article 6.2.7 and formula (6.23)

$\tau_{t,Ed}$	0.5	MPa
$\tau_{t,Rd}$	135.7	MPa
Unity check	0.00	-

Note: The unity check for torsion is lower than the limit value of 0.05. Therefore torsion is considered as

insignificant and is ignored in the combined checks.

Combined bending, axial force and shear force check

According to EN 1993-1-1 article 6.2.9.1 and formula (6.41)

$M_{pl,y,Rd}$	100.85	kNm
α	2.00	
$M_{pl,z,Rd}$	47.88	kNm
β	1.00	

$$\text{Unity check (6.41)} = 0.70 + 0.01 = 0.71 -$$

Note: Since the shear forces are less than half the plastic shear resistances their effect on the moment

resistances is neglected.

Note: Since the axial force satisfies both criteria (6.33) and (6.34) of EN 1993-1-1 article 6.2.9.1(4) its effect on the moment resistance about the y-y axis is neglected.

Note: Since the axial force satisfies criteria (6.35) of EN 1993-1-1 article 6.2.9.1(4) its effect on the moment

resistance about the z-z axis is neglected.

The member satisfies the section check.

....:STABILITY CHECK:....

Classification for member buckling design

Decisive position for stability classification: 0.000 m

Classification of Internal Compression parts

According to EN 1993-1-1 Table 5.2 Sheet 1

Maximum width-to-thickness ratio	20.62
Class 1 Limit	70.75
Class 2 Limit	81.47
Class 3 Limit	110.74

=> Internal Compression parts Class 1

Classification of Outstand Flanges

According to EN 1993-1-1 Table 5.2 Sheet 2

Maximum width-to-thickness ratio	7.88
Class 1 Limit	9.00
Class 2 Limit	10.00
Class 3 Limit	14.16

=> Outstand Flanges Class 1

=> Section classified as Class 1 for member buckling design

Flexural Buckling Check

According to EN 1993-1-1 article 6.3.1.1 and formula (6.46)

Buckling parameters	yy	zz	
Sway type	sway	non-sway	
System length L	9.750	9.750	m
Buckling factor k	2.69	0.82	
Buckling length L _{cr}	26.219	7.994	m
Critical Euler load N _{cr}	111.25	434.56	kN
Slenderness Lambda	316.59	160.19	
Relative slenderness Lambda _{rel}	3.37	1.71	
Limit slenderness Lambda _{rel,0}	0.20	0.20	

Note: The slenderness or compression force is such that Flexural Buckling effects may be ignored

according to EN 1993-1-1 article 6.3.1.2(4).

Lateral Torsional Buckling Check

According to article EN 1993-1-1 : 6.3.2.1. and formula (6.54)

LTB Parameters		
Method for LTB curve	Art. 6.3.2.2.	
W _y	4.2917e-04	m ³
Elastic critical moment M _{cr}	356.97	kNm
Relative slenderness Lambda _{LT}	0.53	
Limit slenderness Lambda _{LT,0}	0.40	
LTB curve	a	
Imperfection Alpha _{LT}	0.21	
Reduction factor Chi _{LT}	0.91	

Buckling resistance Mb.Rd	92.20	kNm
Unity check	0.92	-

Mcr Parameters		
LTB length	3.250	m
k	1.00	
kw	1.00	
C1	1.13	
C2	0.45	
C3	0.53	

Note: C Parameters according to ECCS 119 2006 / Galea 2002

load in center of gravity

Compression and bending check

According to article EN 1993-1-1 : 6.3.3. and formula (6.61), (6.62)

Interaction Method 1

Table of values		
kyy	1.033	
kyz	0.633	
kzy	0.542	
kzz	0.802	
Delta My	0.00	kNm
Delta Mz	0.00	kNm
A	5.3800e-03	m ²
Wy	4.2917e-04	m ³
Wz	2.0375e-04	m ³
NRk	1264.30	kN
My,Rk	100.85	kNm
Mz,Rk	47.88	kNm
My,Ed	84.39	kNm
Mz,Ed	-0.62	kNm
Interaction Method 1		
Mcr0	316.74	kNm
reduced slenderness 0	0.56	
Psi y	0.000	
Psi z	0.000	
Cmy,0	1.001	
Cmz,0	0.789	
Cmy	1.000	
Cmz	0.789	
CmLT	1.000	

muy	1.000	
muz	1.000	
wy	1.103	
wz	1.500	
npl	0.002	
aLT	0.994	
bLT	0.002	
cLT	0.215	
dLT	0.002	
eLT	0.102	
Cyy	0.995	
Cyz	0.879	
Czy	0.976	
Czz	0.990	

Unity check (6.61) = 0.00 + 0.95 + 0.01 = 0.96

Unity check (6.62) = 0.00 + 0.50 + 0.01 = 0.51

Shear buckling check

in buckling field 1

According to article EN 1993-1-5 : 5. & 7.1. and formula (5.10) & (7.1)

Table of values	
hw/t	26.154

The web slenderness is such that the Shear Buckling Check is not required.

The member satisfies the stability check.

Check of steel

Nonlinear calculation, Extreme : Member

Selection : All

Nonlinear combinations : ST+KO+SN KGS

Case	Member	css	mat	dx [m]	un.che ck [-]	sec.che ck [-]	stab.che ck [-]
ST+KO+SN KGS	B51	GREDA 2 - HEA360	S 235	2.750	0.21	0.20	0.21
ST+KO+SN KGS	B52	GREDA 2 - HEA360	S 235	3.429	0.44	0.38	0.44
ST+KO+SN KGS	B53	GREDA 2 - HEA360	S 235	0.000	0.31	0.31	0.00
ST+KO+SN KGS	B54	GREDA 2 - HEA360	S 235	0.000	0.08	0.08	0.08
ST+KO+SN KGS	B55	GREDA 2 - HEA360	S 235	2.932	0.31	0.31	0.00
ST+KO+SN KGS	B56	GREDA 2 - HEA360	S 235	2.750	0.14	0.14	0.00
ST+KO+SN KGS	B58	GREDA 3 - HEA200	S 235	0.000	0.40	0.38	0.40
ST+KO+SN KGS	B63	GREDA 3 - HEA200	S 235	0.000	0.37	0.34	0.37
ST+KO+SN KGS	B64	GREDA 3 - HEA200	S 235	2.000	0.75	0.66	0.75

ST+KO+SN KGS	B67	GEDA 2 - HEA360	S 235	3.367	0.42	0.36	0.42
ST+KO+SN KGS	B68	GEDA 2 - HEA360	S 235	1.450	0.29	0.29	0.00
ST+KO+SN KGS	B70	GEDA 2 - HEA360	S 235	3.225	0.42	0.36	0.42
ST+KO+SN KGS	B71	GEDA 2 - HEA360	S 235	3.083	0.42	0.36	0.42
ST+KO+SN KGS	B72	GEDA 2 - HEA360	S 235	0.000	0.34	0.34	0.00
ST+KO+SN KGS	B314	GEDA 2 - HEA360	S 235	4.875	0.83	0.83	0.83
ST+KO+SN KGS	B316	GEDA 2 - HEA360	S 235	2.100	0.08	0.08	0.00
ST+KO+SN KGS	B317	GEDA 2 - HEA360	S 235	3.225	0.42	0.36	0.42
ST+KO+SN KGS	B318	GEDA 2 - HEA360	S 235	4.875	0.83	0.83	0.00
ST+KO+SN KGS	B319	GEDA 2 - HEA360	S 235	4.200	0.94	0.91	0.94
ST+KO+SN KGS	B320	GEDA 3 - HEA200	S 235	0.000	0.35	0.32	0.35
ST+KO+SN KGS	B321	GEDA 3 - HEA200	S 235	0.000	0.00	0.00	0.00
ST+KO+SN KGS	B326	GEDA 2 - HEA360	S 235	0.000	0.16	0.16	0.16
ST+KO+SN KGS	B327	GEDA 2 - HEA360	S 235	0.000	0.38	0.38	0.00
ST+KO+SN KGS	B330	GEDA 2 - HEA360	S 235	0.000	0.36	0.36	0.00
ST+KO+SN KGS	B331	GEDA 2 - HEA360	S 235	0.000	0.21	0.21	0.00
ST+KO+SN KGS	B332	GEDA 2 - HEA360	S 235	1.050	0.20	0.07	0.20
ST+KO+SN KGS	B333	GEDA 2 - HEA360	S 235	0.000	0.11	0.11	0.11
ST+KO+SN KGS	B334	GEDA 3 - HEA200	S 235	2.100	0.16	0.16	0.00
ST+KO+SN KGS	B335	GEDA 3 - HEA200	S 235	2.100	0.16	0.16	0.16
ST+KO+SN KGS	B336	GEDA 3 - HEA200	S 235	4.200	0.53	0.41	0.53
ST+KO+SN KGS	B337	GEDA 3 - HEA200	S 235	2.100	0.16	0.16	0.00
ST+KO+SN KGS	B338	GEDA 3 - HEA200	S 235	3.225	0.49	0.38	0.49
ST+KO+SN KGS	B339	GEDA 3 - HEA200	S 235	3.225	0.50	0.38	0.50
ST+KO+SN KGS	B346	GEDA 3 - HEA200	S 235	0.000	0.77	0.21	0.77
ST+KO+SN KGS	B347	GEDA 3 - HEA200	S 235	0.000	0.53	0.41	0.53
ST+KO+SN KGS	B348	GEDA 3 - HEA200	S 235	1.050	0.04	0.04	0.00
ST+KO+SN KGS	B349	GEDA 3 - HEA200	S 235	4.588	0.91	0.76	0.91
ST+KO+SN KGS	B350	GEDA 3 - HEA200	S 235	4.875	0.96	0.84	0.96

Sekundarna konstrukcija - zidna

Check of steel

Nonlinear calculation, Extreme : Member

Selection : B202

Nonlinear combinations : ST+VJ2 KGS

EN 1993-1-1 Code Check

National annex: Standard EN

Member B202	5.600 m	IPE200	S 235	ST+VJ2 KGS	0.55 -
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Partial safety factors	
Gamma M0 for resistance of cross-sections	1.00
Gamma M1 for resistance to instability	1.00
Gamma M2 for resistance of net sections	1.25

Material		
Yield strength fy	235.0	MPa
Ultimate strength fu	360.0	MPa
Fabrication	Rolled	

.....SECTION CHECK:.....

The critical check is on position 2.800 m

Internal forces	Calculated	Unit
N,Ed	-7.75	kN
Vy,Ed	0.00	kN
Vz,Ed	0.00	kN
T,Ed	0.00	kNm
My,Ed	-9.70	kNm
Mz,Ed	0.86	kNm

Classification for cross-section design

According to EN 1993-1-1 article 5.5.2

Classification of Internal Compression parts

According to EN 1993-1-1 Table 5.2 Sheet 1

Maximum width-to-thickness ratio	28.39
Class 1 Limit	68.98
Class 2 Limit	79.43
Class 3 Limit	109.86

=> Internal Compression parts Class 1

Classification of Outstand Flanges

According to EN 1993-1-1 Table 5.2 Sheet 2

Maximum width-to-thickness ratio	4.14
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Class 1 Limit	9.00
Class 2 Limit	10.00
Class 3 Limit	14.26

=> Outstand Flanges Class 1

=> Section classified as Class 1 for cross-section design

Compression check

According to EN 1993-1-1 article 6.2.4 and formula (6.9)

A	2.8500e-03	m ²
Nc,Rd	669.75	kN
Unity check	0.01	-

Bending moment check for My

According to EN 1993-1-1 article 6.2.5 and formula (6.12),(6.13)

Wpl,y	2.2100e-04	m ³
Mpl,y,Rd	51.94	kNm
Unity check	0.19	-

Bending moment check for Mz

According to EN 1993-1-1 article 6.2.5 and formula (6.12),(6.13)

Wpl,z	4.4600e-05	m ³
Mpl,z,Rd	10.48	kNm
Unity check	0.08	-

Torsion check

According to EN 1993-1-1 article 6.2.7 and formula (6.23)

Tau,t,Ed	0.0	MPa
Tau,Rd	135.7	MPa
Unity check	0.00	-

Note: The unity check for torsion is lower than the limit value of 0.05. Therefore torsion is considered as

insignificant and is ignored in the combined checks.

Combined bending, axial force and shear force check

According to EN 1993-1-1 article 6.2.9.1 and formula (6.41)

Mpl,y,Rd	51.94	kNm
Alpha	2.00	
Mpl,z,Rd	10.48	kNm
Beta	1.00	

$$\text{Unity check (6.41)} = 0.03 + 0.08 = 0.12 -$$

Note: Since the axial force satisfies both criteria (6.33) and (6.34) of EN 1993-1-1 article 6.2.9.1(4) its effect on the moment resistance about the y-y axis is neglected.

Note: Since the axial force satisfies criteria (6.35) of EN 1993-1-1 article 6.2.9.1(4) its effect on the moment

resistance about the z-z axis is neglected.

The member satisfies the section check.

....:STABILITY CHECK:....

Classification for member buckling design

Decisive position for stability classification: 0.000 m

Classification of Internal Compression parts

According to EN 1993-1-1 Table 5.2 Sheet 1

Maximum width-to-thickness ratio	28.39
Class 1 Limit	33.00
Class 2 Limit	38.00
Class 3 Limit	42.00

=> Internal Compression parts Class 1

Classification of Outstand Flanges

According to EN 1993-1-1 Table 5.2 Sheet 2

Maximum width-to-thickness ratio	4.14
Class 1 Limit	9.00
Class 2 Limit	10.00
Class 3 Limit	14.00

=> Outstand Flanges Class 1

=> Section classified as Class 1 for member buckling design

Flexural Buckling Check

According to EN 1993-1-1 article 6.3.1.1 and formula (6.46)

Buckling parameters	yy	zz	
Sway type	sway	non-sway	
System length L	5.600	5.600	m
Buckling factor k	1.00	1.00	
Buckling length L _{cr}	5.600	5.600	m
Critical Euler load N _{cr}	1284.15	93.85	kN
Slenderness Lambda	67.82	250.87	
Relative slenderness Lambda _{rel}	0.72	2.67	
Limit slenderness Lambda _{rel,0}	0.20	0.20	
Buckling curve	a	b	
Imperfection Alpha	0.21	0.34	
Reduction factor Chi	0.84	0.12	
Buckling resistance N _{b,Rd}	560.63	82.74	kN

Warning: Slenderness 250.87 is larger than the limit value of 200.00.

Flexural Buckling verification		
Cross-section area A	2.8500e-03	m ²
Buckling resistance N _{b,Rd}	82.74	kN
Unity check	0.09	-

Lateral Torsional Buckling Check

According to article EN 1993-1-1 : 6.3.2.1. and formula (6.54)

LTB Parameters		
Method for LTB curve	Art. 6.3.2.2.	
Wy	2.2100e-04	m ³
Elastic critical moment M _{cr}	27.83	kNm
Relative slenderness Lambda,LT	1.37	
Limit slenderness Lambda,LT,0	0.40	
LTB curve	a	
Imperfection Alpha,LT	0.21	
Reduction factor Chi,LT	0.43	
Buckling resistance Mb,Rd	22.58	kNm
Unity check	0.43	-

Mcr Parameters		
LTB length	5.600	m
k	1.00	
kw	1.00	
C1	1.13	
C2	0.45	
C3	0.53	

Note: C Parameters according to ECCS 119 2006 / Galea 2002

load in center of gravity

Compression and bending check

According to article EN 1993-1-1 : 6.3.3. and formula (6.61), (6.62)

Interaction Method 1

Table of values		
kyy	1.076	
kyz	0.879	
kzy	0.554	
kzz	1.065	
Delta My	0.00	kNm
Delta Mz	0.00	kNm
A	2.8500e-03	m ²
Wy	2.2100e-04	m ³
Wz	4.4600e-05	m ³
NRk	669.75	kN
My,Rk	51.94	kNm
Mz,Rk	10.48	kNm
My,Ed	-9.70	kNm
Mz,Ed	0.86	kNm
Interaction Method 1		
Mcr0	24.69	kNm
reduced slenderness 0	1.45	

Psi y	1.000	
Psi z	1.000	
Cmy,0	1.000	
Cmz,0	1.002	
Cmy	1.000	
Cmz	1.002	
CmLT	1.045	
muy	0.999	
muz	0.927	
wy	1.139	
wz	1.500	
npl	0.012	
aLT	0.996	
bLT	0.037	
cLT	0.161	
dLT	0.002	
eLT	0.021	
Cyy	0.976	
Cyz	0.855	
Czy	0.919	
Czz	0.951	

Unity check (6.61) = $0.01 + 0.46 + 0.07 = 0.55$

Unity check (6.62) = $0.09 + 0.24 + 0.09 = 0.42$

The member satisfies the stability check.

Check of steel – sekundarna zidna

Nonlinear calculation, Extreme : Member

Selection : All

Nonlinear combinations : ST+VJ2 KGS

Case	Member	css	mat	dx [m]	un.che ck [-]	sec.che ck [-]	stab.che ck [-]
ST+VJ2 KGS	B171	sekund zidna - IPE200	S 235	2.800	0.43	0.19	0.43
ST+VJ2 KGS	B172	sekund zidna - IPE200	S 235	2.800	0.53	0.19	0.53
ST+VJ2 KGS	B173	sekund zidna - IPE200	S 235	2.800	0.43	0.19	0.43
ST+VJ2 KGS	B174	sekund zidna - IPE200	S 235	2.800	0.43	0.19	0.43
ST+VJ2 KGS	B175	sekund zidna - IPE200	S 235	2.800	0.43	0.19	0.43
ST+VJ2 KGS	B176	sekund zidna - IPE200	S 235	2.800	0.49	0.19	0.49
ST+VJ2 KGS	B177	sekund zidna - IPE200	S 235	2.800	0.43	0.19	0.43
ST+VJ2 KGS	B178	sekund zidna - IPE200	S 235	2.800	0.43	0.19	0.43
ST+VJ2 KGS	B179	sekund zidna - IPE200	S 235	2.800	0.49	0.19	0.49

ST+VJ2 KGS	B180	sekund zidna - IPE200	S 235	2.800	0.43	0.19	0.43
ST+VJ2 KGS	B181	sekund zidna - IPE200	S 235	2.800	0.43	0.19	0.43
ST+VJ2 KGS	B182	sekund zidna - IPE200	S 235	2.800	0.49	0.19	0.49
ST+VJ2 KGS	B183	sekund zidna - IPE200	S 235	2.800	0.49	0.19	0.49
ST+VJ2 KGS	B184	sekund zidna - IPE200	S 235	2.800	0.53	0.19	0.53
ST+VJ2 KGS	B185	sekund zidna - IPE200	S 235	2.800	0.50	0.19	0.50
ST+VJ2 KGS	B186	sekund zidna - IPE200	S 235	2.800	0.49	0.19	0.49
ST+VJ2 KGS	B187	sekund zidna - IPE200	S 235	2.800	0.43	0.19	0.43
ST+VJ2 KGS	B188	sekund zidna - IPE200	S 235	2.800	0.50	0.19	0.50
ST+VJ2 KGS	B189	sekund zidna - IPE200	S 235	2.430	0.06	0.06	0.00
ST+VJ2 KGS	B190	sekund zidna - IPE200	S 235	2.430	0.06	0.06	0.00
ST+VJ2 KGS	B191	sekund zidna - IPE200	S 235	2.430	0.06	0.06	0.00
ST+VJ2 KGS	B192	sekund zidna - IPE200	S 235	1.620	0.03	0.03	0.00
ST+VJ2 KGS	B193	sekund zidna - IPE200	S 235	1.620	0.03	0.03	0.00
ST+VJ2 KGS	B194	sekund zidna - IPE200	S 235	1.620	0.03	0.03	0.00
ST+VJ2 KGS	B195	sekund zidna - IPE200	S 235	1.500	0.02	0.02	0.00
ST+VJ2 KGS	B196	sekund zidna - IPE200	S 235	1.500	0.02	0.02	0.00
ST+VJ2 KGS	B197	sekund zidna - IPE200	S 235	1.500	0.02	0.02	0.00
ST+VJ2 KGS	B198	sekund zidna - IPE200	S 235	2.800	0.43	0.19	0.43
ST+VJ2 KGS	B199	sekund zidna - IPE200	S 235	2.800	0.43	0.19	0.43
ST+VJ2 KGS	B200	sekund zidna - IPE200	S 235	2.800	0.43	0.19	0.43
ST+VJ2 KGS	B201	sekund zidna - IPE200	S 235	2.550	0.07	0.07	0.00
ST+VJ2 KGS	B202	sekund zidna - IPE200	S 235	2.800	0.55	0.19	0.55
ST+VJ2 KGS	B203	sekund zidna - IPE200	S 235	2.800	0.49	0.19	0.49
ST+VJ2 KGS	B204	sekund zidna - IPE200	S 235	2.800	0.43	0.19	0.43
ST+VJ2 KGS	B205	sekund zidna - IPE200	S 235	5.102	0.07	0.00	0.07
ST+VJ2 KGS	B206	sekund zidna - IPE200	S 235	2.800	0.50	0.19	0.50
ST+VJ2 KGS	B207	sekund zidna - IPE200	S 235	2.800	0.50	0.19	0.50
ST+VJ2 KGS	B208	sekund zidna - IPE200	S 235	2.800	0.50	0.19	0.50
ST+VJ2 KGS	B209	sekund zidna - IPE200	S 235	2.550	0.08	0.07	0.08
ST+VJ2 KGS	B210	sekund zidna - IPE200	S 235	2.800	0.50	0.19	0.50
ST+VJ2 KGS	B211	sekund zidna - IPE200	S 235	2.800	0.51	0.19	0.51
ST+VJ2 KGS	B212	sekund zidna - IPE200	S 235	2.800	0.51	0.19	0.51
ST+VJ2 KGS	B213	sekund zidna - IPE200	S 235	2.100	0.16	0.11	0.16
ST+VJ2 KGS	B214	sekund zidna - IPE200	S 235	2.100	0.11	0.11	0.00
ST+VJ2 KGS	B215	sekund zidna - IPE200	S 235	2.800	0.43	0.19	0.43
ST+VJ2 KGS	B216	sekund zidna - IPE200	S 235	2.800	0.54	0.19	0.54
ST+VJ2 KGS	B217	sekund zidna - IPE200	S 235	2.800	0.43	0.19	0.43
ST+VJ2 KGS	B218	sekund zidna - IPE200	S 235	2.100	0.11	0.11	0.00
ST+VJ2 KGS	B219	sekund zidna - IPE200	S 235	2.100	0.15	0.11	0.15
ST+VJ2 KGS	B220	sekund zidna - IPE200	S 235	2.800	0.49	0.19	0.49
ST+VJ2 KGS	B221	sekund zidna - IPE200	S 235	2.800	0.50	0.19	0.50
ST+VJ2 KGS	B222	sekund zidna - IPE200	S 235	2.800	0.51	0.19	0.51
ST+VJ2 KGS	B223	sekund zidna - IPE200	S 235	2.100	0.14	0.11	0.14
ST+VJ2 KGS	B224	sekund zidna - IPE200	S 235	2.100	0.14	0.11	0.14

ST+VJ2 KGS	B228	sekund zidna - IPE200	S 235	1.700	0.04	0.04	0.00
ST+VJ2 KGS	B229	sekund zidna - IPE200	S 235	1.525	0.03	0.03	0.00
ST+VJ2 KGS	B230	sekund zidna - IPE200	S 235	0.000	0.01	0.01	0.00
ST+VJ2 KGS	B231	sekund zidna - IPE200	S 235	1.620	0.04	0.04	0.00
ST+VJ2 KGS	B232	sekund zidna - IPE200	S 235	1.030	0.01	0.01	0.00
ST+VJ2 KGS	B233	sekund zidna - IPE200	S 235	1.700	0.04	0.04	0.00
ST+VJ2 KGS	B234	sekund zidna - IPE200	S 235	1.525	0.03	0.03	0.00
ST+VJ2 KGS	B235	sekund zidna - IPE200	S 235	0.825	0.01	0.01	0.00
ST+VJ2 KGS	B236	sekund zidna - IPE200	S 235	1.620	0.04	0.04	0.00
ST+VJ2 KGS	B237	sekund zidna - IPE200	S 235	1.030	0.01	0.01	0.00
ST+VJ2 KGS	B238	sekund zidna - IPE200	S 235	1.700	0.04	0.04	0.00
ST+VJ2 KGS	B239	sekund zidna - IPE200	S 235	1.525	0.03	0.03	0.00
ST+VJ2 KGS	B240	sekund zidna - IPE200	S 235	2.445	0.08	0.08	0.00
ST+VJ2 KGS	B241	sekund zidna - IPE200	S 235	1.030	0.01	0.01	0.00
ST+VJ2 KGS	B297	sekund zidna - IPE200	S 235	2.100	0.14	0.11	0.14
ST+VJ2 KGS	B298	sekund zidna - IPE200	S 235	2.100	0.14	0.11	0.14
ST+VJ2 KGS	B299	sekund zidna - IPE200	S 235	2.100	0.14	0.11	0.14
ST+VJ2 KGS	B300	sekund zidna - IPE200	S 235	2.100	0.14	0.11	0.14
ST+VJ2 KGS	B301	sekund zidna - IPE200	S 235	2.100	0.14	0.11	0.14
ST+VJ2 KGS	B302	sekund zidna - IPE200	S 235	2.100	0.15	0.11	0.15

Vjetrovni spregovi

Check of steel – vertikalne sprega: poz. 5

Nonlinear calculation, Extreme : Member

Selection : B78

Nonlinear combinations : ST+VJ2 KGS

EN 1993-1-1 Code Check

National annex: Standard EN

Member B78	5.600 m	CHS76.1/5.0	S 235	ST+VJ2 KGS	0.96 -
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Partial safety factors	
Gamma M0 for resistance of cross-sections	1.00
Gamma M1 for resistance to instability	1.00
Gamma M2 for resistance of net sections	1.25

Material		
Yield strength f_y	235.0	MPa
Ultimate strength f_u	360.0	MPa
Fabrication	Rolled	

.....SECTION CHECK:.....

The critical check is on position 5.600 m

Internal forces	Calculated	Unit
N,Ed	-4.44	kN
Vy,Ed	0.14	kN
Vz,Ed	-2.92	kN
T,Ed	-0.02	kNm
My,Ed	-3.24	kNm
Mz,Ed	0.14	kNm

Classification for cross-section design

According to EN 1993-1-1 article 5.5.2

Classification for Tubular Sections

According to EN 1993-1-1 Table 5.2 Sheet 3

Maximum width-to-thickness ratio	15.22
Class 1 Limit	50.00
Class 2 Limit	70.00
Class 3 Limit	90.00

=> Section classified as Class 1 for cross-section design

Compression check

According to EN 1993-1-1 article 6.2.4 and formula (6.9)

A	1.1200e-03	m ²
Nc,Rd	263.20	kN
Unity check	0.02	-

Bending moment check for My

According to EN 1993-1-1 article 6.2.5 and formula (6.12),(6.13)

Wpl,y	2.4918e-05	m ³
Mpl,y,Rd	5.86	kNm
Unity check	0.55	-

Bending moment check for Mz

According to EN 1993-1-1 article 6.2.5 and formula (6.12),(6.13)

Wpl,z	2.4918e-05	m ³
Mpl,z,Rd	5.86	kNm
Unity check	0.02	-

Shear check for Vy

According to EN 1993-1-1 article 6.2.6 and formula (6.17)

Eta	1.20	
Av	7.1301e-04	m ²
Vpl,y,Rd	96.74	kN
Unity check	0.00	-

Shear check for Vz

According to EN 1993-1-1 article 6.2.6 and formula (6.17)

Eta	1.20	
Av	7.1301e-04	m ²
Vpl,z,Rd	96.74	kN
Unity check	0.03	-

Torsion check

According to EN 1993-1-1 article 6.2.7 and formula (6.23)

Tau,t,Ed	0.6	MPa
Tau,Rd	135.7	MPa
Unity check	0.00	-

Combined bending, axial force and shear force check

According to EN 1993-1-1 article 6.2.9.1 and formula (6.31)

M,resultant	3.24	kNm
V,resultant	2.92	kN
MN,Rd	5.85	kNm
Unity check	0.55	-

Note: The resultant internal forces are used for CHS sections.

Note: Since the shear forces are less than half the plastic shear resistances their effect on the moment

resistances is neglected.

The member satisfies the section check.

.....STABILITY CHECK:.....

Classification for member buckling design

Decisive position for stability classification: 0.000 m

Classification for Tubular Sections

According to EN 1993-1-1 Table 5.2 Sheet 3

Maximum width-to-thickness ratio	15.22
Class 1 Limit	50.00
Class 2 Limit	70.00
Class 3 Limit	90.00

=> Section classified as Class 1 for member buckling design

Flexural Buckling Check

According to EN 1993-1-1 article 6.3.1.1 and formula (6.46)

Buckling parameters	yy	zz	
Sway type	sway	non-sway	
System length L	5.600	5.600	m
Buckling factor k	2.21	0.84	
Buckling length L _{cr}	12.392	4.718	m
Critical Euler load N _{cr}	9.57	66.01	kN
Slenderness Lambda	492.52	187.53	
Relative slenderness Lambda _{rel}	5.24	2.00	
Limit slenderness Lambda _{rel,0}	0.20	0.20	
Buckling curve	a	a	
Imperfection Alpha	0.21	0.21	
Reduction factor Chi	0.03	0.22	
Buckling resistance N _{b,Rd}	9.20	58.84	kN

Flexural Buckling verification		
Cross-section area A	1.1200e-03	m ²
Buckling resistance N _{b,Rd}	9.20	kN
Unity check	0.48	-

Lateral Torsional Buckling Check

Note: The cross-section concerns a CHS section which is not susceptible to Lateral Torsional Buckling.

Compression and bending check

According to article EN 1993-1-1 : 6.3.3. and formula (6.61), (6.62)

Interaction Method 1

Table of values		
k _{yy}	0.845	
k _{yz}	0.645	
k _{zy}	1.069	
k _{zz}	1.234	
Delta M _y	0.00	kNm
Delta M _z	0.00	kNm
A	1.1200e-03	m ²
W _y	2.4918e-05	m ³
W _z	2.4918e-05	m ³

NRk	263.20	kN
My,Rk	5.86	kNm
Mz,Rk	5.86	kNm
My,Ed	-3.24	kNm
Mz,Ed	0.14	kNm
Interaction Method 1		
Mcr0	73.31	kNm
reduced slenderness 0	0.28	
Psi y	0.000	
Psi z	0.000	
Cmy,0	0.739	
Cmz,0	0.972	
Cmy	0.739	
Cmz	0.972	
CmLT	1.000	
muy	0.545	
muz	0.947	
wy	1.340	
wz	1.340	
npl	0.017	
aLT	0.000	
bLT	0.000	
cLT	0.000	
dLT	0.000	
eLT	0.000	
Cyy	0.889	
Cyz	0.529	
Czy	0.732	
Czz	0.800	

Unity check (6.61) = $0.48 + 0.47 + 0.02 = 0.96$

Unity check (6.62) = $0.08 + 0.59 + 0.03 = 0.70$

The member satisfies the stability check.

Check of steel – dijagonala sprega: poz. 6

Nonlinear calculation, Extreme : Member

Selection : B250

Nonlinear combinations : ST+VJ2 KGS

EN 1993-1-1 Code Check

National annex: Standard EN

Member B250	7.107 m	RD16	S 235	ST+VJ2 KGS	0.30 -
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Partial safety factors	
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Gamma M0 for resistance of cross-sections	1.00
Gamma M1 for resistance to instability	1.00
Gamma M2 for resistance of net sections	1.25

Material		
Yield strength f_y	235.0	MPa
Ultimate strength f_u	360.0	MPa
Fabrication	Rolled	

Warning: Strength reduction in function of the thickness is not supported for this type of cross-section.

....:SECTION CHECK:....

The critical check is on position 7.107 m

Internal forces	Calculated	Unit
N,Ed	14.32	kN
Vy,Ed	0.00	kN
Vz,Ed	0.00	kN
T,Ed	0.00	kNm
My,Ed	0.00	kNm
Mz,Ed	0.00	kNm

Classification for cross-section design

According to EN 1993-1-1 article 5.5.2

Warning: Classification is not supported for this type of cross-section.

The section is checked as elastic, class 3.

Tension check

According to EN 1993-1-1 article 6.2.3 and formula (6.5)

A	2.0096e-04	m ²
Npl,Rd	47.23	kN
Nu,Rd	52.09	kN
Nt,Rd	47.23	kN
Unity check	0.30	-

The member satisfies the section check.

....:STABILITY CHECK:....

The member satisfies the stability check.

Check of steel – vjetrovni spregovi

Nonlinear calculation, Extreme : Member

Selection : All

Nonlinear combinations : ST+VJ2 KGS

Case	Member	css	mat	dx [m]	un.che ck [-]	sec.che ck [-]	stab.che ck [-]
------	--------	-----	-----	--------	---------------	----------------	-----------------

ST+VJ2 KGS	B78	vert sprega - CHS76.1/5.0	S 235	5.600	0.96	0.55	0.96
ST+VJ2 KGS	B80	vert sprega - CHS76.1/5.0	S 235	5.600	0.53	0.53	0.00
ST+VJ2 KGS	B82	vert sprega - CHS76.1/5.0	S 235	5.600	0.54	0.54	0.00
ST+VJ2 KGS	B84	vert sprega - CHS76.1/5.0	S 235	0.000	1.00	0.55	1.00
ST+VJ2 KGS	B87	vert sprega - CHS76.1/5.0	S 235	0.000	0.51	0.51	0.00
ST+VJ2 KGS	B124	vert sprega - CHS76.1/5.0	S 235	0.000	0.45	0.38	0.45
ST+VJ2 KGS	B127	vert sprega - CHS76.1/5.0	S 235	5.600	0.41	0.37	0.41
ST+VJ2 KGS	B134	vert sprega - CHS76.1/5.0	S 235	5.600	0.46	0.36	0.46
ST+VJ2 KGS	B136	vert sprega - CHS76.1/5.0	S 235	0.000	0.39	0.36	0.39
ST+VJ2 KGS	B139	vert sprega - CHS76.1/5.0	S 235	0.000	0.81	0.38	0.81
ST+VJ2 KGS	B146	vert sprega - CHS76.1/5.0	S 235	0.000	0.25	0.22	0.25
ST+VJ2 KGS	B148	vert sprega - CHS76.1/5.0	S 235	0.000	0.26	0.22	0.26
ST+VJ2 KGS	B150	vert sprega - CHS76.1/5.0	S 235	0.000	0.32	0.24	0.32
ST+VJ2 KGS	B151	vert sprega - CHS76.1/5.0	S 235	0.000	0.30	0.23	0.30
ST+VJ2 KGS	B153	vert sprega - CHS76.1/5.0	S 235	0.000	0.28	0.23	0.28
ST+VJ2 KGS	B169	vert sprega - CHS76.1/5.0	S 235	0.000	0.00	0.00	0.00
ST+VJ2 KGS	B170	vert sprega - CHS76.1/5.0	S 235	0.000	0.00	0.00	0.00
ST+VJ2 KGS	B172	vert sprega - CHS76.1/5.0	S 235	2.800	0.16	0.06	0.16
ST+VJ2 KGS	B184	vert sprega - CHS76.1/5.0	S 235	2.800	0.17	0.06	0.17
ST+VJ2 KGS	B202	vert sprega - CHS76.1/5.0	S 235	2.800	0.22	0.06	0.22
ST+VJ2 KGS	B216	vert sprega - CHS76.1/5.0	S 235	2.800	0.20	0.06	0.20
ST+VJ2 KGS	B245	dijagonala - RD16	S 235	7.030	0.23	0.23	0.00
ST+VJ2 KGS	B246	dijagonala - RD16	S 235	7.030	0.26	0.00	0.26
ST+VJ2 KGS	B247	dijagonala - RD16	S 235	7.030	0.16	0.16	0.00
ST+VJ2 KGS	B248	dijagonala - RD16	S 235	7.030	0.26	0.00	0.26
ST+VJ2 KGS	B249	dijagonala - RD16	S 235	0.000	0.27	0.00	0.27
ST+VJ2 KGS	B250	dijagonala - RD16	S 235	7.107	0.30	0.30	0.00
ST+VJ2 KGS	B251	dijagonala - RD16	S 235	0.000	0.27	0.00	0.27
ST+VJ2 KGS	B252	dijagonala - RD16	S 235	7.107	0.21	0.21	0.00
ST+VJ2 KGS	B253	dijagonala - RD16	S 235	0.000	0.26	0.00	0.26
ST+VJ2 KGS	B254	dijagonala - RD16	S 235	7.030	0.28	0.28	0.00
ST+VJ2 KGS	B255	dijagonala - RD16	S 235	0.000	0.26	0.00	0.26
ST+VJ2 KGS	B256	dijagonala - RD16	S 235	7.030	0.21	0.21	0.00
ST+VJ2 KGS	B257	dijagonala - RD16	S 235	7.030	0.23	0.23	0.00
ST+VJ2 KGS	B258	dijagonala - RD16	S 235	0.000	0.26	0.00	0.26
ST+VJ2 KGS	B259	dijagonala - RD16	S 235	7.030	0.16	0.16	0.00
ST+VJ2 KGS	B260	dijagonala - RD16	S 235	0.000	0.26	0.00	0.26
ST+VJ2 KGS	B261	dijagonala - RD16	S 235	0.000	0.00	0.00	0.00
ST+VJ2 KGS	B262	dijagonala - RD16	S 235	0.000	0.11	0.11	0.00
ST+VJ2 KGS	B263	dijagonala - RD16	S 235	0.000	0.00	0.00	0.00
ST+VJ2 KGS	B264	dijagonala - RD16	S 235	6.472	0.02	0.02	0.00
ST+VJ2 KGS	B265	dijagonala - RD16	S 235	0.000	0.06	0.06	0.00
ST+VJ2 KGS	B266	dijagonala - RD16	S 235	6.355	0.00	0.00	0.00
ST+VJ2 KGS	B267	dijagonala - RD16	S 235	0.000	0.07	0.07	0.00
ST+VJ2 KGS	B268	dijagonala - RD16	S 235	7.579	0.00	0.00	0.00

ST+VJ2 KGS	B269	dijagonala - RD16	S 235	7.419	0.09	0.09	0.00
ST+VJ2 KGS	B270	dijagonala - RD16	S 235	0.000	0.00	0.00	0.00
ST+VJ2 KGS	B271	dijagonala - RD16	S 235	6.472	0.01	0.01	0.00
ST+VJ2 KGS	B272	dijagonala - RD16	S 235	0.000	0.00	0.00	0.00
ST+VJ2 KGS	B273	dijagonala - RD16	S 235	6.355	0.00	0.00	0.00
ST+VJ2 KGS	B274	dijagonala - RD16	S 235	0.000	0.03	0.03	0.00
ST+VJ2 KGS	B275	dijagonala - RD16	S 235	7.579	0.00	0.00	0.00
ST+VJ2 KGS	B276	dijagonala - RD16	S 235	0.000	0.06	0.06	0.00
ST+VJ2 KGS	B277	dijagonala - RD16	S 235	4.329	0.08	0.08	0.00
ST+VJ2 KGS	B278	dijagonala - RD16	S 235	0.000	0.00	0.00	0.00
ST+VJ2 KGS	B279	dijagonala - RD16	S 235	4.412	0.17	0.17	0.00
ST+VJ2 KGS	B280	dijagonala - RD16	S 235	0.000	0.00	0.00	0.00
ST+VJ2 KGS	B281	dijagonala - RD16	S 235	4.993	0.18	0.18	0.00
ST+VJ2 KGS	B282	dijagonala - RD16	S 235	0.000	0.08	0.00	0.08
ST+VJ2 KGS	B285	dijagonala - RD16	S 235	5.050	0.06	0.06	0.00
ST+VJ2 KGS	B286	dijagonala - RD16	S 235	0.000	0.00	0.00	0.00
ST+VJ2 KGS	B287	dijagonala - RD16	S 235	4.679	0.04	0.04	0.00
ST+VJ2 KGS	B288	dijagonala - RD16	S 235	0.000	0.00	0.00	0.00
ST+VJ2 KGS	B289	dijagonala - RD16	S 235	0.000	0.00	0.00	0.00
ST+VJ2 KGS	B290	dijagonala - RD16	S 235	5.307	0.00	0.00	0.00
ST+VJ2 KGS	B291	dijagonala - RD16	S 235	4.513	0.00	0.00	0.00
ST+VJ2 KGS	B292	dijagonala - RD16	S 235	0.000	0.03	0.03	0.00
ST+VJ2 KGS	B293	dijagonala - RD16	S 235	5.193	0.00	0.00	0.00
ST+VJ2 KGS	B294	dijagonala - RD16	S 235	0.000	0.06	0.06	0.00
ST+VJ2 KGS	B295	dijagonala - RD16	S 235	5.406	0.00	0.00	0.00
ST+VJ2 KGS	B296	dijagonala - RD16	S 235	0.000	0.09	0.09	0.00

PRORAČUN SPOJEVA

SPOJ STUP - TEMELJ

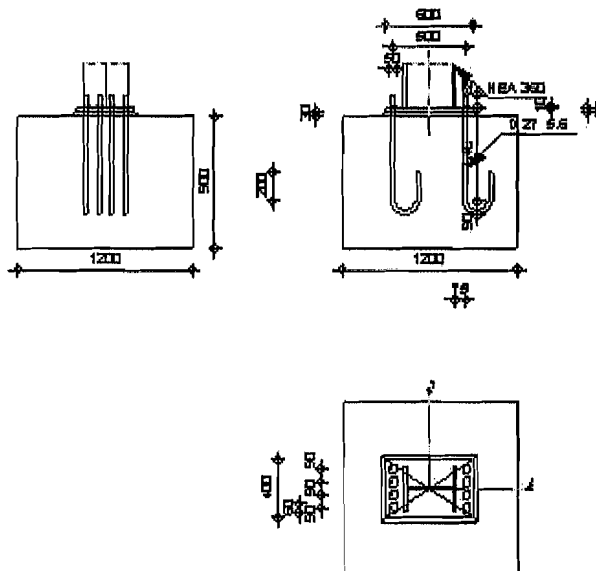


Autodesk Robot Structural Analysis Professional 2016

Fixed column base design

Eurocode 3: EN 1993-1-8:2005/AC:2009 + CEB Design Guide: Design of fastenings in concrete


Ratio
0.73



General

Connection no.:

1

Connection name:

Fixed column base

Geometry

Column

Section:

HEA 360

$L_c =$	8.50	[m]	Column length
$\alpha =$	0.0	[Deg]	Inclination angle
$h_c =$	350	[mm]	Height of column section
$b_{fc} =$	300	[mm]	Width of column section
$t_{wc} =$	10	[mm]	Thickness of the web of column section
$t_{fc} =$	18	[mm]	Thickness of the flange of column section
$r_c =$	27	[mm]	Radius of column section fillet
$A_c =$	14300	[mm ²]	Cross-sectional area of a column
$I_{yc} =$	330900000	[mm ⁴]	Moment of inertia of the column section

Material: STEEL

$f_{yc} =$	265.00	[MPa]	Resistance
$f_{uc} =$	430.00	[MPa]	Yield strength of a material

Column base

$l_{pd} =$	600	[mm]	Length
$b_{pd} =$	400	[mm]	Width
$t_{pd} =$	32	[mm]	Thickness

Material: STEEL

$f_{ypd} = 265.00$

[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 = 5.6

Anchor class

 $f_{yb} = 300.00$

[MPa]

Yield strength of the anchor material

 $f_{ub} = 500.00$

[MPa]

Tensile strength of the anchor material

 $d = 27$

[mm]

Bolt diameter

 $A_s = 459$ [mm²]

Effective section area of a bolt

 $A_v = 573$ [mm²]

Area of bolt section

 $n_H = 2$

Number of bolt columns

 $n_V = 4$

Number of bolt rows

Horizontal spacing $e_{HI} = 500$ [mm]Vertical spacing $e_{VI} = 90; 90$ [mm]**Anchor dimensions** $L_1 = 85$ [mm] $L_2 = 640$ [mm] $L_3 = 180$ [mm] $L_4 = 200$ [mm]**Washer** $l_{wd} = 50$ [mm] Length $b_{wd} = 60$ [mm] Width $t_{wd} = 10$ [mm] Thickness**Material factors** $\gamma_{MO} = 1.00$ Partial safety factor $\gamma_{M2} = 1.25$ Partial safety factor $\gamma_C = 1.50$ Partial safety factor**Spread footing** $L = 1200$ [mm] Spread footing length $B = 1200$ [mm] Spread footing width $H = 900$ [mm] Spread footing height**Concrete**

Class

C30

 $f_{ck} = 30.00$ [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** $a_p = 6$ [mm] Footing plate of the column base**Loads**

Case: Manual calculations.

 $N_{j,Ed} = -135.00$ [kN] Axial force $V_{j,Ed,z} = 41.00$ [kN] Shear force $M_{j,Ed,y} = 120.00$ [kN*m] Bending moment**Results**

Compression zone

COMPRESSION OF CONCRETE

$f_{cd} =$	20.00	[MPa]	Design compressive resistance	EN 1992-1:[3.1.6.(1)]
$f_j =$	32.66	[MPa]	Design bearing resistance under the base plate	[6.2.5.(7)]
$c = t_p \sqrt{(f_{yp}/(3*f_j*\gamma_{M0}))}$				
$c =$	53	[mm]	Additional width of the bearing pressure zone	[6.2.5.(4)]
$b_{eff} =$	123	[mm]	Effective width of the bearing pressure zone under the flange	[6.2.5.(3)]
$l_{eff} =$	400	[mm]	Effective length of the bearing pressure zone under the flange	[6.2.5.(3)]
$A_{c0} =$	49101	[mm ²]	Area of the joint between the base plate and the foundation	EN 1992-1:[6.7.(3)]
$A_{c1} =$	441911	[mm ²]	Maximum design area of load distribution	EN 1992-1:[6.7.(3)]
$F_{rd,u} = A_{c0}*f_{cd}*\sqrt{(A_{c1}/A_{c0})} \leq 3*A_{c0}*f_{cd}$				
$F_{rd,u} =$	2946.08	[kN]	Bearing resistance of concrete	EN 1992-1:[6.7.(3)]
$\beta_j =$	0.67		Reduction factor for compression	[6.2.5.(7)]
$f_{jd} = \beta_j * F_{rd,u} / (b_{eff} * l_{eff})$				
$f_{jd} =$	40.00	[MPa]	Design bearing resistance	[6.2.5.(7)]
$A_{c,n} =$	122377	[mm ²]	Bearing area for compression	[6.2.8.2.(1)]
$A_{c,y} =$	49101	[mm ²]	Bearing area for bending My	[6.2.8.3.(1)]
$F_{c,Rd,I} = A_{c,I} * f_{jd}$				
$F_{c,Rd,n} =$	4895.06	[kN]	Bearing resistance of concrete for compression	[6.2.8.2.(1)]
$F_{c,Rd,y} =$	1964.05	[kN]	Bearing resistance of concrete for bending My	[6.2.8.3.(1)]

COLUMN FLANGE AND WEB IN COMPRESSION

$CL =$	1.00		Section class	EN 1993-1-1:[5.5.2]
$W_{pl,y} =$	2088000	[mm ³]	Plastic section modulus	EN1993-1-1:[6.2.5.(2)]
$M_{c,Rd,y} =$	553.32	[kN*m]	Design resistance of the section for bending	EN1993-1-1:[6.2.5]
$h_{f,y} =$	333	[mm]	Distance between the centroids of flanges	[6.2.6.7.(1)]
$F_{c,fc,Rd,y} = M_{c,Rd,y} / h_{f,y}$				
$F_{c,fc,Rd,y} =$	1664.12	[kN]	Resistance of the compressed flange and web	[6.2.6.7.(1)]

RESISTANCES OF SPREAD FOOTING IN THE COMPRESSION ZONE

$N_{l,Rd} = F_{c,Rd,n}$				
$N_{l,Rd} =$	4895.06	[kN]	Resistance of a spread footing for axial compression	[6.2.8.2.(1)]
$F_{c,Rd,y} = \min(F_{c,Rd,y}, F_{c,fc,Rd,y})$				
$F_{c,Rd,y} =$	1664.12	[kN]	Resistance of spread footing in the compression zone	[6.2.8.3]

Tension zone

STEEL FAILURE

$A_b =$	459	[mm ²]	Effective anchor area	[Table 3.4]
$f_{ub} =$	500.00	[MPa]	Tensile strength of the anchor material	[Table 3.4]
$\beta =$	0.85		Reduction factor of anchor resistance	[3.6.1.(3)]
$F_{t,Rd,s1} = \beta * 0.9 * f_{ub} * A_b / \gamma_{M2}$				
$F_{t,Rd,s1} =$	140.45	[kN]	Anchor resistance to steel failure	[Table 3.4]
$\gamma_{Ms} =$	1.20		Partial safety factor	CEB [3.2.3.2]
$f_{yb} =$	300.00	[MPa]	Yield strength of the anchor material	CEB [9.2.2]
$F_{t,Rd,s2} = f_{yb} * A_b / \gamma_{Ms}$				
$F_{t,Rd,s2} =$	114.75	[kN]	Anchor resistance to steel failure	CEB [9.2.2]
$F_{t,Rd,s} = \min(F_{t,Rd,s1}, F_{t,Rd,s2})$				
$F_{t,Rd,s} =$	114.75	[kN]	Anchor resistance to steel failure	

PULL-OUT FAILURE

$f_{ck} =$	30.00	[MPa]	Characteristic compressive strength of concrete	EN 1992-1:[3.1.2]
$f_{cd} = 0.7 * 0.3 * f_{ck}^{2/3} / \gamma_{c}$				

$$f_{ctd} = 1.35 \text{ [MPa]}$$

Design tensile resistance

$$\eta_1 = 1.00$$

Coeff. related to the quality of the bond conditions and concreting conditions

$$\eta_2 = 1.00$$

Coeff. related to the bar diameter

$$f_{bd} = 2.25 \cdot \eta_1 \cdot \eta_2 \cdot f_{ctd}$$

$$f_{bd} = 3.04 \text{ [MPa]}$$

Design value of the ultimate bond stress

$$h_{ef} = 640 \text{ [mm]}$$

Effective anchorage depth

$$F_{t,Rd,p} = \pi \cdot d \cdot h_{ef} \cdot f_{bd}$$

$$F_{t,Rd,p} = 165.10 \text{ [kN]}$$

Design uplift capacity

CONCRETE CONE FAILURE

$$h_{ef} = 310 \text{ [mm]}$$

Effective anchorage depth

$$N_{RK,c}^0 = 7.5 [N^{0.5}/mm^{0.5}] \cdot f_{ck} \cdot h_{ef}^{1.5}$$

$$N_{RK,c}^0 = 224.21 \text{ [kN]}$$

Characteristic resistance of an anchor

$$s_{cr,N} = 930 \text{ [mm]}$$

Critical width of the concrete cone

$$c_{cr,N} = 465 \text{ [mm]}$$

Critical edge distance

$$A_{c,N0} = 1716000 \text{ [mm}^2\text{]}$$

Maximum area of concrete cone

$$A_{c,N} = 1440000 \text{ [mm}^2\text{]}$$

Actual area of concrete cone

$$\psi_{A,N} = A_{c,N}/A_{c,N0}$$

$$\psi_{A,N} = 0.84$$

Factor related to anchor spacing and edge distance

$$c = 350 \text{ [mm]}$$

Minimum edge distance from an anchor

$$\psi_{s,N} = 0.7 + 0.3 \cdot c/c_{cr,N} \leq 1.0$$

$$\psi_{s,N} = 0.9$$

Factor taking account the influence of edges of the concrete member on the distribution of stresses in the concrete

$$\psi_{ec,N} = 1.0$$

Factor related to distribution of tensile forces acting on anchors

$$\psi_{re,N} = 0.5 + h_{ef}[\text{mm}]/200 \leq 1.0$$

$$\psi_{re,N} = 1.00$$

Shell spalling factor

$$\psi_{ucr,N} = 1.00$$

Factor taking into account whether the anchorage is in cracked or non-cracked concrete

$$\gamma_{Mc} = 2.16$$

Partial safety factor

$$F_{t,Rd,c} = N_{RK,c}^0 \cdot \psi_{A,N} \cdot \psi_{s,N} \cdot \psi_{ec,N} \cdot \psi_{re,N} \cdot \psi_{ucr,N} / \gamma_{Mc}$$

$$F_{t,Rd,c} = 80.64 \text{ [kN]}$$

Design anchor resistance to concrete cone failure

SPLITTING FAILURE

$$h_{ef} = 640 \text{ [mm]}$$

Effective anchorage depth

$$N_{RK,c}^0 = 7.5 [N^{0.5}/mm^{0.5}] \cdot f_{ck} \cdot h_{ef}^{1.5}$$

$$N_{RK,c}^0 = 665.11 \text{ [kN]}$$

Design uplift capacity

$$s_{cr,N} = 1280 \text{ [mm]}$$

Critical width of the concrete cone

$$c_{cr,N} = 640 \text{ [mm]}$$

Critical edge distance

$$A_{c,N0} = 2759000 \text{ [mm}^2\text{]}$$

Maximum area of concrete cone

$$A_{c,N} = 1440000 \text{ [mm}^2\text{]}$$

Actual area of concrete cone

$$\psi_{A,N} = A_{c,N}/A_{c,N0}$$

$$\psi_{A,N} = 0.52$$

Factor related to anchor spacing and edge distance

$$c = 350 \text{ [mm]}$$

Minimum edge distance from an anchor

$$\psi_{s,N} = 0.7 + 0.3 \cdot c/c_{cr,N} \leq 1.0$$

$$\psi_{s,N} = 0.86$$

Factor taking account the influence of edges of the concrete member on the distribution of stresses in the concrete

$$\psi_{ec,N} = 1.00$$

Factor related to distribution of tensile forces acting on anchors

$$\psi_{re,N} = 0.5 + h_{ef}[\text{mm}]/200$$

$$\leq 1.0$$

$$\psi_{re,N} = 1.00$$

Shell spalling factor

$$N_{Rk,c}^0 = 224.21 \quad [kN] \quad \text{Characteristic resistance of an anchor}$$

CEB [9.2.4]

$$\psi_{ucr,N} = 1.00 \quad \text{Factor taking into account whether the anchorage is in cracked or non-cracked concrete}$$

CEB [9.2.5]

$$\psi_{h,N} = (h/(2 \cdot h_{ef}))^{2/3} \leq 1.2$$

$$\psi_{h,N} = 0.79 \quad \text{Coeff. related to the foundation height}$$

CEB [9.2.5]

$$\gamma_{M,sp} = 2.16 \quad \text{Partial safety factor}$$

CEB [3.2.3.1]

$$F_{t,Rd,sp} = N_{Rk,c}^0 \cdot \psi_{A,N} \cdot \psi_{s,N} \cdot \psi_{ec,N} \cdot \psi_{re,N} \cdot \psi_{ucr,N} \cdot \psi_{h,N} / \gamma_{M,sp}$$

$$F_{t,Rd,sp} = 109.80 \quad [kN] \quad \text{Design anchor resistance to splitting of concrete}$$

CEB [9.2.5]

TENSILE RESISTANCE OF AN ANCHOR

$$F_{t,Rd} = \min(F_{t,Rd,s}, F_{t,Rd,p}, F_{t,Rd,c}, F_{t,Rd,sp})$$

$$F_{t,Rd} = 80.64 \quad [kN] \quad \text{Tensile resistance of an anchor}$$

BENDING OF THE BASE PLATE**Bending moment $M_{j,Ed,y}$**

$$l_{eff,1} = 200 \quad [mm] \quad \text{Effective length for a single bolt for mode 1}$$

[6.2.6.5]

$$l_{eff,2} = 200 \quad [mm] \quad \text{Effective length for a single bolt for mode 2}$$

[6.2.6.5]

$$m = 68 \quad [mm] \quad \text{Distance of a bolt from the stiffening edge}$$

[6.2.6.5]

$$M_{pl,1,Rd} = 13.57 \quad [kN \cdot m] \quad \text{Plastic resistance of a plate for mode 1}$$

[6.2.4]

$$M_{pl,2,Rd} = 13.57 \quad [kN \cdot m] \quad \text{Plastic resistance of a plate for mode 2}$$

[6.2.4]

$$F_{T,1,Rd} = 795.64 \quad [kN] \quad \text{Resistance of a plate for mode 1}$$

[6.2.4]

$$F_{T,2,Rd} = 366.00 \quad [kN] \quad \text{Resistance of a plate for mode 2}$$

[6.2.4]

$$F_{T,3,Rd} = 322.58 \quad [kN] \quad \text{Resistance of a plate for mode 3}$$

[6.2.4]

$$F_{t,pl,Rd,y} = \min(F_{T,1,Rd}, F_{T,2,Rd}, F_{T,3,Rd})$$

$$F_{t,pl,Rd,y} = 322.58 \quad [kN] \quad \text{Tension resistance of a plate}$$

[6.2.4]

RESISTANCES OF SPREAD FOOTING IN THE TENSION ZONE

$$F_{T,Rd,y} = F_{t,pl,Rd,y}$$

$$F_{T,Rd,y} = 322.58 \quad [kN] \quad \text{Resistance of a column base in the tension zone}$$

[6.2.8.3]

Connection capacity check

$$N_{j,Ed} / N_{j,Rd} \leq 1.0 \quad (6.24) \quad 0.03 < 1.00 \quad \text{verified} \quad (0.03)$$

$$e_y = 889 \quad [mm] \quad \text{Axial force eccentricity} \quad [6.2.8.3]$$

$$z_{o,y} = 166 \quad [mm] \quad \text{Lever arm } F_{C,Rd,y} \quad [6.2.8.1.(2)]$$

$$z_{t,y} = 250 \quad [mm] \quad \text{Lever arm } F_{T,Rd,y} \quad [6.2.8.1.(3)]$$

$$M_{j,Rd,y} = 165.16 \quad [kN \cdot m] \quad \text{Connection resistance for bending} \quad [6.2.8.3]$$

$$M_{j,Ed,y} / M_{j,Rd,y} \leq 1.0 \quad (6.23) \quad 0.73 < 1.00 \quad \text{verified} \quad (0.73)$$

Shear**BEARING PRESSURE OF AN ANCHOR BOLT ONTO THE BASE PLATE****Shear force $V_{j,Ed,z}$**

$$\alpha_{d,z} = 0.57 \quad \text{Coeff. taking account of the bolt position - in the direction of shear} \quad [\text{Table 3.4}]$$

$$\alpha_{b,z} = 0.57 \quad \text{Coeff. for resistance calculation } F_{1,vb,Rd} \quad [\text{Table 3.4}]$$

$$k_{1,z} = 2.50 \quad \text{Coeff. taking account of the bolt position - perpendicularly to the direction of shear} \quad [\text{Table 3.4}]$$

$$F_{1,vb,Rd,z} = k_{1,z} \cdot \alpha_{b,z} \cdot f_{up} \cdot d \cdot t_p / \gamma_{M2}$$

$$F_{1,vb,Rd,z} = 427.03 \quad [kN] \quad \text{Resistance of an anchor bolt for bearing pressure onto the base plate} \quad [6.2.2.(7)]$$

SHEAR OF AN ANCHOR BOLT

$$\alpha_b = 0.35 \quad \text{Coeff. for resistance calculation } F_{2,vb,Rd} \quad [6.2.2.(7)]$$

$$A_{vb} = 573 \quad [mm^2] \quad \text{Area of bolt section} \quad [6.2.2.(7)]$$

Connection capacity check

$$N_{i,Ed} / N_{i,Rd} \leq 1,0 \text{ (6.2.4)} \quad 0,03 < 1,00 \quad \text{verified} \quad (0,03)$$

$$e_y = 889 \text{ [mm]} \quad \text{Axial force eccentricity} \quad [6.2.8.3]$$

$$f_{ub} = 500,00 \text{ [MPa]} \quad \text{Tensile strength of the anchor material} \quad [6.2.2.(7)]$$

$$\gamma_{M2} = 1,25 \quad \text{Partial safety factor} \quad [6.2.2.(7)]$$

$$F_{2,vb,Rd} = \alpha_b \cdot f_{ub} \cdot A_{vb} / \gamma_{M2}$$

$$F_{2,vb,Rd} = 80,16 \text{ [kN]} \quad \text{Shear resistance of a bolt - without lever arm} \quad [6.2.2.(7)]$$

$$\alpha_M = 2,00 \quad \text{Factor related to the fastening of an anchor in the foundation} \quad \text{CEB [9.3.2.2]}$$

$$M_{Rk,s} = 0,43 \text{ [kN*m]} \quad \text{Characteristic bending resistance of an anchor} \quad \text{CEB [9.3.2.2]}$$

$$l_{sm} = 60 \text{ [mm]} \quad \text{Lever arm length} \quad \text{CEB [9.3.2.2]}$$

$$\gamma_{Ms} = 1,20 \quad \text{Partial safety factor} \quad \text{CEB [3.2.3.2]}$$

$$F_{v,Rd,sm} = \alpha_M \cdot M_{Rk,s} / (l_{sm} \cdot \gamma_{Ms})$$

$$F_{v,Rd,sm} = 11,92 \text{ [kN]} \quad \text{Shear resistance of a bolt - with lever arm} \quad \text{CEB [9.3.1]}$$

CONCRETE PRY-OUT FAILURE

$$N_{Rk,c} = 174,19 \text{ [kN]} \quad \text{Design uplift capacity} \quad \text{CEB [9.2.4]}$$

$$k_3 = 2,00 \quad \text{Factor related to the anchor length} \quad \text{CEB [9.3.3]}$$

$$\gamma_{Mc} = 2,16 \quad \text{Partial safety factor} \quad \text{CEB [3.2.3.1]}$$

$$F_{v,Rd,cp} = k_3 \cdot N_{Rk,c} / \gamma_{Mc}$$

$$F_{v,Rd,cp} = 161,29 \text{ [kN]} \quad \text{Concrete resistance for pry-out failure} \quad \text{CEB [9.3.1]}$$

CONCRETE EDGE FAILURE**Shear force $V_{j,Ed,z}$**

$$V_{Rk,c,z}^0 = 707,84 \text{ [kN]} \quad \text{Characteristic resistance of an anchor} \quad \text{CEB [9.3.4.(a)]}$$

$$\psi_{A,V,z} = 0,89 \quad \text{Factor related to anchor spacing and edge distance} \quad \text{CEB [9.3.4]}$$

$$\psi_{h,V,z} = 1,00 \quad \text{Factor related to the foundation thickness} \quad \text{CEB [9.3.4.(c)]}$$

$$\psi_{s,V,z} = 0,97 \quad \text{Factor related to the influence of edges parallel to the shear load direction} \quad \text{CEB [9.3.4.(d)]}$$

$$\psi_{ec,V,z} = 1,00 \quad \text{Factor taking account a group effect when different shear loads are acting on the individual anchors in a group} \quad \text{CEB [9.3.4.(e)]}$$

$$\psi_{\alpha,V,z} = 1,00 \quad \text{Factor related to the angle at which the shear load is applied} \quad \text{CEB [9.3.4.(f)]}$$

$$\psi_{ucr,V,z} = 1,00 \quad \text{Factor related to the type of edge reinforcement used} \quad \text{CEB [9.3.4.(g)]}$$

$$\gamma_{Mc} = 2,16 \quad \text{Partial safety factor} \quad \text{CEB [3.2.3.1]}$$

$$F_{v,Rd,c,z} = V_{Rk,c,z}^0 \cdot \psi_{A,V,z} \cdot \psi_{h,V,z} \cdot \psi_{s,V,z} \cdot \psi_{ec,V,z} \cdot \psi_{\alpha,V,z} \cdot \psi_{ucr,V,z} / \gamma_{Mc}$$

$$F_{v,Rd,c,z} = 280,30 \text{ [kN]} \quad \text{Concrete resistance for edge failure} \quad \text{CEB [9.3.1]}$$

SPLITTING RESISTANCE

$$C_{f,d} = 0,30 \quad \text{Coeff. of friction between the base plate and concrete} \quad [6.2.2.(6)]$$

Connection capacity check

$$N_{j,Ed} / N_{j,Rd} \leq 1.0 \text{ (6.24)} \quad 0.03 < 1.00 \quad \text{verified} \quad (0.03)$$

$$e_y = 889 \text{ [mm]} \quad \text{Axial force eccentricity} \quad [6.2.8.3]$$

$$N_{c,Ed} = 135.00 \text{ [kN]} \quad \text{Compressive force} \quad [6.2.2.(6)]$$

$$F_{t,Rd} = C_{f,d} \cdot N_{c,Ed}$$

$$F_{t,Rd} = 40.50 \text{ [kN]} \quad \text{Slip resistance} \quad [6.2.2.(6)]$$

SHEAR CHECK

$$V_{j,Rd,z} = n_b \cdot \min(F_{1,vb,Rd,z}, F_{2,vb,Rd}, F_{v,Rd,sm}, F_{v,Rd,op}, F_{v,Rd,c,z}) + F_{t,Rd}$$

$$V_{j,Rd,z} = 135.86 \text{ [kN]} \quad \text{Connection resistance for shear} \quad \text{CEB [9.3.1]}$$

$$V_{j,Ed,z} / V_{j,Rd,z} \leq 1.0 \quad 0.30 < 1.00 \quad \text{verified} \quad (0.30)$$

Welds between the column and the base plate

$$\sigma_{\perp} = 73.83 \text{ [MPa]} \quad \text{Normal stress in a weld} \quad [4.5.3.(7)]$$

$$\tau_{\perp} = 73.83 \text{ [MPa]} \quad \text{Perpendicular tangent stress} \quad [4.5.3.(7)]$$

$$\tau_{yII} = 0.00 \text{ [MPa]} \quad \text{Tangent stress parallel to } V_{j,Ed,y} \quad [4.5.3.(7)]$$

$$\tau_{zII} = 10.85 \text{ [MPa]} \quad \text{Tangent stress parallel to } V_{j,Ed,z} \quad [4.5.3.(7)]$$

$$\beta_W = 0.85 \quad \text{Resistance-dependent coefficient} \quad [4.5.3.(7)]$$

$$\sigma_{\perp} / (0.9 \cdot f_u / \gamma_{M2}) \leq 1.0 \text{ (4.1)} \quad 0.24 < 1.00 \quad \text{verified} \quad (0.24)$$

$$\sqrt{(\sigma_{\perp}^2 + 3.0 (\tau_{yII}^2 + \tau_{zII}^2)) / (f_u / (\beta_W \cdot \gamma_{M2}))} \leq 1.0 \text{ (4.1)} \quad 0.36 < 1.00 \quad \text{verified} \quad (0.36)$$

$$\sqrt{(\sigma_{\perp}^2 + 3.0 (\tau_{zII}^2 + \tau_{yII}^2)) / (f_u / (\beta_W \cdot \gamma_{M2}))} \leq 1.0 \text{ (4.1)} \quad 0.33 < 1.00 \quad \text{verified} \quad (0.33)$$

Connection stiffness**Bending moment $M_{j,Ed,y}$**

$$b_{eff} = 123 \text{ [mm]} \quad \text{Effective width of the bearing pressure zone under the flange} \quad [6.2.5.(3)]$$

$$l_{eff} = 400 \text{ [mm]} \quad \text{Effective length of the bearing pressure zone under the flange} \quad [6.2.5.(3)]$$

$$k_{13,y} = E_c \cdot \sqrt{(b_{eff} \cdot l_{eff})} / (1.275 \cdot E)$$

$$k_{13,y} = 22 \text{ [mm]} \quad \text{Stiffness coeff. of compressed concrete} \quad [\text{Table 6.11}]$$

$$l_{eff} = 200 \text{ [mm]} \quad \text{Effective length for a single bolt for mode 2} \quad [6.2.6.5]$$

$$m = 68 \text{ [mm]} \quad \text{Distance of a bolt from the stiffening edge} \quad [6.2.6.5]$$

$$k_{15,y} = 0.425 \cdot l_{eff} \cdot t_p^3 / (m^3)$$

$$k_{15,y} = 9 \text{ [mm]} \quad \text{Stiffness coeff. of the base plate subjected to tension} \quad [\text{Table 6.11}]$$

$$L_b = 302 \text{ [mm]} \quad \text{Effective anchorage depth} \quad [\text{Table 6.11}]$$

$$k_{16,y} = 1.6 \cdot A_b / L_b$$

$$k_{16,y} = 2 \text{ [mm]} \quad \text{Stiffness coeff. of an anchor subjected to tension} \quad [\text{Table 6.11}]$$

$$\lambda_{0,y} = 0.38 \quad \text{Column slenderness} \quad [5.2.2.5.(2)]$$

$$S_{j,ini,y} = 58115.24 \text{ [kN}\cdot\text{m]} \quad \text{Initial rotational stiffness} \quad [\text{Table 6.12}]$$

$$\lambda_{0,y} \leq 0.5 \text{ RIGID} \quad [5.2.2.5.(2)]$$

Weakest component:

FOUNDATION - CONCRETE CONE PULL-OUT FAILURE

Connection conforms to the code**Ratio** 0.73**SPOJ GREDA U SLJEMENU**

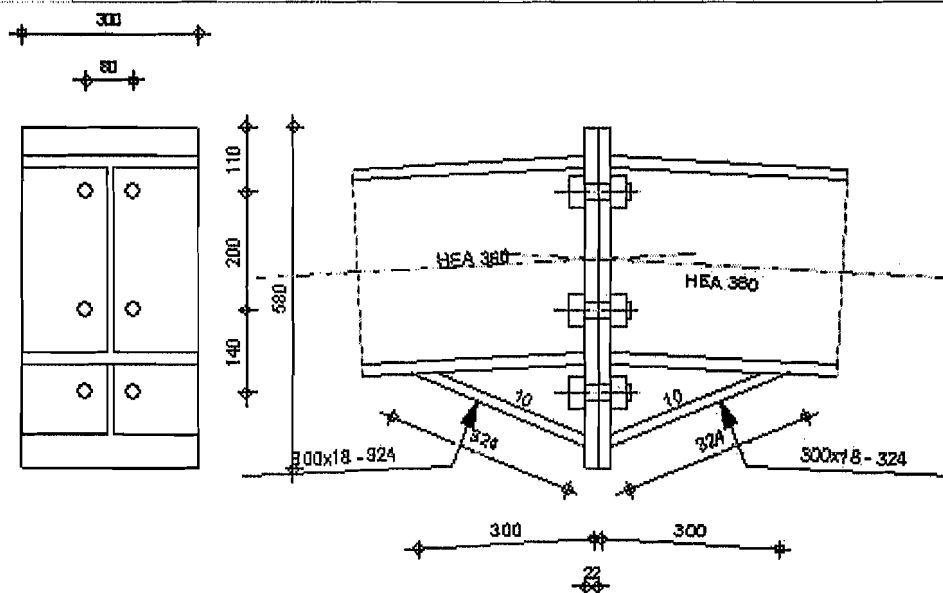


Autodesk Robot Structural Analysis Professional 2016

Design of fixed beam-to-beam connection

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

Ratio
0.58



General

Connection no.: 2

Connection name: Beam-Beam

Geometry

Left side

Beam

Section: HEA 360

$\alpha =$	-177.0	[Deg]	Inclination angle
$h_{bl} =$	350	[mm]	Height of beam section
$b_{tbl} =$	300	[mm]	Width of beam section
$t_{wbl} =$	10	[mm]	Thickness of the web of beam section
$t_{tbl} =$	18	[mm]	Thickness of the flange of beam section
$r_{bl} =$	27	[mm]	Radius of beam section fillet
$A_{bl} =$	14300	[mm ²]	Cross-sectional area of a beam
$I_{xbl} =$	330900000	[mm ⁴]	Moment of inertia of the beam section

Material: STEEL

$f_{yb} =$ 265.00 [MPa] Resistance

Right side

Beam

Section: HEA 360

$\alpha =$	-3.0	[Deg]	Inclination angle
$h_{br} =$	350	[mm]	Height of beam section
$b_{tbr} =$	300	[mm]	Width of beam section
$t_{wbr} =$	10	[mm]	Thickness of the web of beam section
$t_{tbr} =$	18	[mm]	Thickness of the flange of beam section
$r_{br} =$	27	[mm]	Radius of beam section fillet

$\alpha = -3.0$ [Deg] $A_{br} = 14300$ [mm²] Cross-sectional area of a beam $I_{xbr} = 330900000$ [mm⁴] Moment of inertia of the beam section

Material: STEEL

 $f_{yb} = 265.00$ [MPa] Resistance**Bolts**

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

 $d = 27$ [mm] Bolt diameter

Class = 10.9 Bolt class

 $F_{tRd} = 330.48$ [kN] Tensile resistance of a bolt $n_h = 2$ Number of bolt columns $n_v = 3$ Number of bolt rows $h_1 = 110$ [mm] Distance between first bolt and upper edge of front plateHorizontal spacing $e_1 = 80$ [mm]Vertical spacing $p_1 = 200; 140$ [mm]**Plate** $h_{pr} = 580$ [mm] Plate height $b_{pr} = 300$ [mm] Plate width $t_{pr} = 22$ [mm] Plate thickness

Material: STEEL

 $f_{ypr} = 265.00$ [MPa] Resistance**Lower stiffener** $w_{rd} = 300$ [mm] Plate width $t_{rd} = 18$ [mm] Flange thickness $h_{rd} = 140$ [mm] Plate height $t_{wrd} = 10$ [mm] Web thickness $l_{rd} = 300$ [mm] Plate length $\alpha_d = 22.5$ [Deg] Inclination angle

Material: STEEL

 $f_{ybu} = 265.00$ [MPa] Resistance**Fillet welds** $a_w = 5$ [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 $\gamma_{M2} = 1.25$ Partial safety factor $\gamma_{M3} = 1.25$ Partial safety factor

[2.2]

[2.2]

[2.2]

[2.2]

Loads**Ultimate limit state**

Case: Manual calculations.

 $M_{b1,Ed} = 192.00$ [kN*m] Bending moment in the right beam $V_{b1,Ed} = 30.00$ [kN] Shear force in the right beam $N_{b1,Ed} = -10.00$ [kN] Axial force in the right beam**Results**

Beam resistances

COMPRESSION

$A_b = 14300 \text{ [mm}^2\text{]}$ Area EN1993-1-1:[6.2.4]

$$N_{cb,Rd} = A_b f_{yb} / \gamma_{M0}$$

$N_{cb,Rd} = 3789.50 \text{ [kN]}$ Design compressive resistance of the section EN1993-1-1:[6.2.4]

SHEAR

$A_{vb} = 6320 \text{ [mm}^2\text{]}$ Shear area EN1993-1-1:[6.2.6.(3)]

$$V_{cb,Rd} = A_{vb} (f_{yb} / \sqrt{3}) / \gamma_{M0}$$

$V_{cb,Rd} = 966.95 \text{ [kN]}$ Design sectional resistance for shear EN1993-1-1:[6.2.6.(2)]

$$V_{b1,Ed} / V_{cb,Rd} \leq 1.0 \quad 0.03 < 1.00 \quad \text{verified} \quad (0.03)$$

BENDING - PLASTIC MOMENT (WITHOUT BRACKETS)

$W_{plb} = 2088000 \text{ [mm}^3\text{]}$ Plastic section modulus EN1993-1-1:[6.2.5.(2)]

$$M_{b,pl,Rd} = W_{plb} f_{yb} / \gamma_{M0}$$

$M_{b,pl,Rd} = 553.32 \text{ [kN*m]}$ 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

$W_{pl} = 3125990 \text{ [mm}^3\text{]}$ Plastic section modulus EN1993-1-1:[6.2.5]

$$M_{cb,Rd} = W_{pl} f_{yb} / \gamma_{M0}$$

$M_{cb,Rd} = 828.39 \text{ [kN*m]}$ Design resistance of the section for bending EN1993-1-1:[6.2.5]

FLANGE AND WEB - COMPRESSION

$M_{cb,Rd} = 828.39 \text{ [kN*m]}$ Design resistance of the section for bending EN1993-1-1:[6.2.5]

$h_f = 472 \text{ [mm]}$ Distance between the centroids of flanges [6.2.6.7.(1)]

$$F_{c,fb,Rd} = M_{cb,Rd} / h_f$$

$F_{c,fb,Rd} = 1755.15 \text{ [kN]}$ Resistance of the compressed flange and web [6.2.6.7.(1)]

WEB OR BRACKET FLANGE - COMPRESSION - LEVEL OF THE BEAM BOTTOM FLANGE

Bearing:

$\beta = 3.0 \text{ [Deg]}$ Angle between the front plate and the beam

$\gamma = 22.5 \text{ [Deg]}$ Inclination angle of the bracket plate

$b_{eff,c,wb} = 293 \text{ [mm]}$ Effective width of the web for compression [6.2.6.2.(1)]

$A_{vb} = 4920 \text{ [mm}^2\text{]}$ Shear area EN1993-1-1:[6.2.6.(3)]

$\omega = 0.83$ Reduction factor for interaction with shear [6.2.6.2.(1)]

$\sigma_{com,Ed} = 76.42 \text{ [MPa]}$ 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)]

$$F_{c,wb,Rd1} = [\omega k_{wc} b_{eff,c,wb} t_{wb} f_{yb} / \gamma_{M0}] \cos(\gamma) / \sin(\gamma - \beta)$$

$F_{c,wb,Rd1} = 1375.03 \text{ [kN]}$ Beam web resistance [6.2.6.2.(1)]

Buckling:

$d_{wb} = 261 \text{ [mm]}$ Height of compressed web [6.2.6.2.(1)]

$\lambda_p = 0.93$ Plate slenderness of an element [6.2.6.2.(1)]

$\rho = 0.85$ Reduction factor for element buckling [6.2.6.2.(1)]

$$F_{c,wb,Rd2} = [\omega k_{wc} \rho b_{eff,c,wb} t_{wb} f_{yb} / \gamma_{M1}] \cos(\gamma) / \sin(\gamma - \beta)$$

$F_{c,wb,Rd2} = 1164.26 \text{ [kN]}$ Beam web resistance [6.2.6.2.(1)]

Resistance of the bracket flange

$$F_{c,wb,Rd3} = b_b t_b f_{yb} / (0.8 \gamma_{M0})$$

$F_{c,wb,Rd3} = 1788.75 \text{ [kN]}$ Resistance of the bracket flange [6.2.6.7.(1)]

Final resistance:

$$F_{c,wb,Rd,low} = \min(F_{c,wb,Rd1}, F_{c,wb,Rd2}, F_{c,wb,Rd3})$$

$F_{c,wb,Rd,low} = 1164.26 \text{ [kN]}$ Beam web resistance [6.2.6.2.(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}
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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	29	-	110	-	200	184	235	184	235	292	207	207	207
2	29	-	110	-	170	184	255	184	255	340	170	170	170
3	29	-	110	-	140	184	255	184	255	232	197	197	197

- 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

$$N_{j,Rd} = \min (N_{cb,Rd} / 2, F_{c,wb,Rd,low})$$

$$N_{j,Rd} = 2328.52 \text{ [kN]} \quad \text{Connection resistance for compression} \quad [6.2]$$

$$N_{b1,Ed} / N_{j,Rd} \leq 1.0 \quad 0.00 < 1.00 \quad \text{verified} \quad (0.00)$$

Connection resistance for bending

$$F_{t,Rd} = 330.48 \text{ [kN]} \quad \text{Bolt resistance for tension} \quad [\text{Table 3.4}]$$

$$B_{p,Rd} = 577.75 \text{ [kN]} \quad \text{Punching shear resistance of a bolt} \quad [\text{Table 3.4}]$$

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

$$F_{t,fc,Rd} = \min (F_{T,1,fc,Rd}, F_{T,2,fc,Rd}, F_{T,3,fc,Rd}) \quad [6.2.6.4], [\text{Tab.6.2}]$$

$$F_{t,wc,Rd} = \omega b_{eff,t,wc} t_{wc} f_{yc} / \gamma_{MO} \quad [6.2.6.3.(1)]$$

$$F_{t,ep,Rd} = \min (F_{T,1,ep,Rd}, F_{T,2,ep,Rd}, F_{T,3,ep,Rd}) \quad [6.2.6.5], [\text{Tab.6.2}]$$

$$F_{t,wb,Rd} = b_{eff,t,wb} t_{wb} f_{yb} / \gamma_{MO} \quad [6.2.6.8.(1)]$$

RESISTANCE OF THE BOLT ROW NO. 1

F _{t1,Rd,comp} - Formula	F _{t1,Rd,comp}	Component
F _{t1,Rd} = Min (F _{t1,Rd,comp})	488.58	Bolt row resistance
F _{t,ep,Rd(1)} = 595.22	595.22	Front plate - tension
F _{t,wb,Rd(1)} = 488.58	488.58	Beam web - tension
B _{p,Rd} = 1155.49	1155.49	Bolts due to shear punching
F _{c,fb,Rd} = 1755.15	1755.15	Beam flange - compression
F _{c,wb,Rd} = 1164.26	1164.26	Beam web - compression

RESISTANCE OF THE BOLT ROW NO. 2

F _{t2,Rd,comp} - Formula	F _{t2,Rd,comp}	Component
F _{t2,Rd} = Min (F _{t2,Rd,comp})	488.58	Bolt row resistance
F _{t,ep,Rd(2)} = 614.77	614.77	Front plate - tension
F _{t,wb,Rd(2)} = 488.58	488.58	Beam web - tension
B _{p,Rd} = 1155.49	1155.49	Bolts due to shear punching
F _{c,fb,Rd} - $\sum_1^1 F_{ij,Rd}$ = 1755.15 - 488.58	1266.58	Beam flange - compression
F _{c,wb,Rd} - $\sum_1^1 F_{ij,Rd}$ = 1164.26 - 488.58	675.68	Beam web - compression

F_{t2,Rd,comp} - Formula

$$F_{t,ep,Rd(2+1)} - \sum_1^1 F_{tj,Rd} = 1100.90 - 488.58$$

$$F_{t,wb,Rd(2+1)} - \sum_1^1 F_{tj,Rd} = 999.87 - 488.58$$

F_{t2,Rd,comp}

$$612.32$$

$$511.29$$

Component

Front plate - tension - group

Beam web - tension - group

RESISTANCE OF THE BOLT ROW NO. 3

F_{t3,Rd,comp} - Formula

$$F_{t3,Rd} = \text{Min} (F_{t3,Rd,comp})$$

$$F_{t,ep,Rd(3)} = 614.77$$

$$F_{t,wb,Rd(3)} = 488.58$$

$$B_{p,Rd} = 1155.49$$

$$F_{c,fb,Rd} - \sum_1^2 F_{tj,Rd} = 1755.15 - 977.15$$

$$F_{c,wb,Rd} - \sum_1^2 F_{tj,Rd} = 1164.26 - 977.15$$

$$F_{t,ep,Rd(3+2)} - \sum_2^2 F_{tj,Rd} = 1091.31 - 488.58$$

$$F_{t,wb,Rd(3+2)} - \sum_2^2 F_{tj,Rd} = 973.71 - 488.58$$

$$F_{t,ep,Rd(3+2+1)} - \sum_2^1 F_{tj,Rd} = 1659.87 - 977.15$$

$$F_{t,wb,Rd(3+2+1)} - \sum_2^1 F_{tj,Rd} = 1523.07 - 977.15$$

F_{t3,Rd,comp}

$$187.11$$

$$614.77$$

$$488.58$$

$$1155.49$$

$$778.00$$

$$187.11$$

$$602.73$$

$$485.13$$

$$682.72$$

$$545.92$$

Component

Bolt row resistance

Front plate - tension

Beam web - tension

Bolts due to shear punching

Beam flange - compression

Beam web - compression

Front plate - tension - group

Beam web - tension - group

Front plate - tension - group

Beam web - tension - group

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	421	488.58	-	-	595.22	488.58	660.96	1155.49
2	221	488.58	-	-	614.77	488.58	660.96	1155.49
3	81	187.11	-	-	614.77	488.58	660.96	1155.49

CONNECTION RESISTANCE FOR BENDING M_{j,Rd}

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

$$M_{j,Rd} = 328.51 \quad [\text{kN}\cdot\text{m}] \quad \text{Connection resistance for bending}$$

[6.2]

$$M_{b1,Ed} / M_{j,Rd} \leq 1.0$$

$$0.58 < 1.00$$

verified

(0.58)

Connection resistance for shear

$$\alpha_v = 0.60$$

Coefficient for calculation of F_{v,Rd}

[Table 3.4]

$$F_{v,Rd} = 274.83$$

[kN] Shear resistance of a single bolt

[Table 3.4]

$$F_{t,Rd,max} = 330.48$$

[kN] Tensile resistance of a single bolt

[Table 3.4]

$$F_{b,Rd,int} = 415.48$$

[kN] Bearing resistance of an intermediate bolt

[Table 3.4]

$$F_{b,Rd,ext} = 415.48$$

[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	660.96	-3.33	488.58	285.55	282.21	382.02
2	660.96	-3.33	488.58	285.55	282.21	382.02
3	660.96	-3.33	187.11	109.35	106.02	486.68

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

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

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

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

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

$$V_{j,Rd} = n_h \sum_1^n F_{vj,Rd}$$

[Table 3.4]

$$V_{j,Rd} = 1250.71$$

[kN] Connection resistance for shear

[Table 3.4]

$$V_{b1,Ed} / V_{j,Rd} \leq 1.0$$

$$0.02 < 1.00$$

verified

(0.02)

Weld resistance

$A_w =$	20139	[mm ²]	Area of all welds	[4.5.3.2(2)]
$A_{wy} =$	16320	[mm ²]	Area of horizontal welds	[4.5.3.2(2)]
$A_{wz} =$	3819	[mm ²]	Area of vertical welds	[4.5.3.2(2)]
$I_{wy} =$	723040669	[mm ⁴]	Moment of inertia of the weld arrangement with respect to the hor. axis	[4.5.3.2(5)]
$\sigma_{\perp \max} = \tau_{\perp \max} =$	52.48	[MPa]	Normal stress in a weld	[4.5.3.2(5)]
$\sigma_{\perp} = \tau_{\perp} =$	43.13	[MPa]	Stress in a vertical weld	[4.5.3.2(5)]
$\tau_{\parallel} =$	7.86	[MPa]	Tangent stress	[4.5.3.2(5)]
$\beta_w =$	0.85		Correlation coefficient	[4.5.3.2(7)]
$\sqrt{[\sigma_{\perp \max}^2 + 3 \cdot (\tau_{\perp \max}^2)]} \leq f_u / (\beta_w \cdot \gamma_{M2})$				
	104.96	<	404.71	verified (0.26)
$\sqrt{[\sigma_{\perp}^2 + 3 \cdot (\tau_{\perp}^2 + \tau_{\parallel}^2)]} \leq f_u / (\beta_w \cdot \gamma_{M2})$				
	87.32	<	404.71	verified (0.22)
$\sigma_{\perp} \leq 0.9 \cdot f_u / \gamma_{M2}$				
	52.48	<	309.60	verified (0.17)

Connection stiffness

$t_{wash} =$	6	[mm]	Washer thickness	[6.2.6.3(2)]
$h_{head} =$	19	[mm]	Bolt head height	[6.2.6.3(2)]
$h_{nut} =$	27	[mm]	Bolt nut height	[6.2.6.3(2)]
$L_b =$	75	[mm]	Bolt length	[6.2.6.3(2)]
$k_{10} =$	10	[mm]	Stiffness coefficient of bolts	[6.3.2.(1)]

STIFFNESSES OF BOLT ROWS

Nr	h_j	k_3	k_4	k_5	$k_{eff,j}$	$k_{eff,j} h_j$	$k_{eff,j} h_j^2$
					Sum	5523	1779210
1	421			70	8	3235	1361241
2	221			64	8	1666	367845
3	81			70	8	621	50125

$$k_{eff,j} = 1 / (\sum_3^b (1 / k_{i,j})) \quad [6.3.3.1.(2)]$$

$$z_{eq} = \sum_j k_{eff,j} h_j^2 / \sum_j k_{eff,j} h_j$$

$$z_{eq} = 322 \quad [mm] \quad \text{Equivalent force arm} \quad [6.3.3.1.(3)]$$

$$k_{eq} = \sum_j k_{eff,j} h_j / z_{eq}$$

$$k_{eq} = 17 \quad [mm] \quad \text{Equivalent stiffness coefficient of a bolt arrangement} \quad [6.3.3.1.(1)]$$

$$S_{j,ini} = E z_{eq}^2 k_{eq} \quad [6.3.1.(4)]$$

$$S_{j,ini} = 364738.09 \quad [kN \cdot m] \quad \text{Initial rotational stiffness} \quad [6.3.1.(4)]$$

$$\mu = 1.00 \quad \text{Stiffness coefficient of a connection} \quad [6.3.1.(6)]$$

$$S_j = S_{j,ini} / \mu \quad [6.3.1.(4)]$$

$$S_j = 364738.09 \quad [kN \cdot m] \quad \text{Final rotational stiffness} \quad [6.3.1.(4)]$$

Connection classification due to stiffness.

$$S_{j,rig} = 108535.20 \quad [kN \cdot m] \quad \text{Stiffness of a rigid connection} \quad [5.2.2.5]$$

$$S_{j,pin} = 6783.45 \quad [kN \cdot m] \quad \text{Stiffness of a pinned connection} \quad [5.2.2.5]$$

$$S_{j,ini} \geq S_{j,rig} \text{ RIGID}$$

Weakest component:

BEAM WEB OR BRACKET FLANGE - COMPRESSION

Remarks

Distance of bolts from an edge is too large. 130 [mm] > 128 [mm]

Distance of bolts from the beam web is too small. 35 [mm] < 36 [mm]

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GLAVNI PROJEKT - KONSTRUKCIJA

Investitor: AQUACHEM d.o.o.

Gradjevina: GOSPODARSKA GRAĐEVINA - SKLADIŠNA HALA

str. 79

td.: 124/16

z.o.p.: AH-16

Connection conforms to the code

Ratio 0.58

SPOJ GREDA - STUP

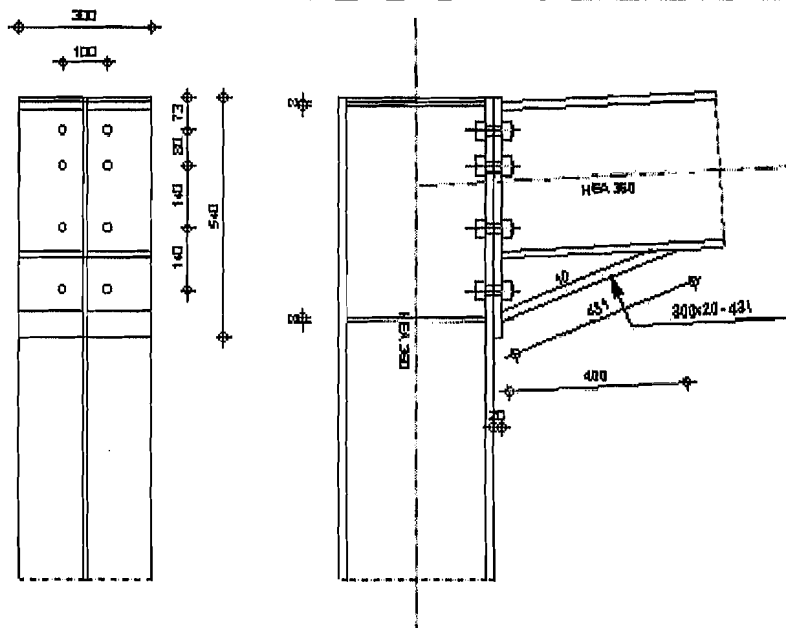


Autodesk Robot Structural Analysis Professional 2016

Design of fixed beam-to-column connection

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

Ratio
0.88



General

Connection no.: 3
Connection name: Frame knee

Geometry

Column

Section: HEA 360

$\alpha = -90.0$ [Deg] Inclination angle
 $h_c = 350$ [mm] Height of column section
 $b_{fc} = 300$ [mm] Width of column section
 $t_{wc} = 10$ [mm] Thickness of the web of column section
 $t_{fc} = 18$ [mm] Thickness of the flange of column section
 $r_c = 27$ [mm] Radius of column section fillet
 $A_c = 14300$ [mm²] Cross-sectional area of a column
 $I_{xc} = 330900000$ [mm⁴] Moment of inertia of the column section
Material: STEEL
 $f_{yc} = 265.00$ [MPa] Resistance

Beam

Section: HEA 360

$\alpha = 3.0$ [Deg] Inclination angle
 $h_b = 350$ [mm] Height of beam section
 $b_f = 300$ [mm] Width of beam section
 $t_{wb} = 10$ [mm] Thickness of the web of beam section
 $t_{fb} = 18$ [mm] Thickness of the flange of beam section
 $r_b = 27$ [mm] Radius of beam section fillet

$\alpha = 3.0$ [Deg] Incline angle
 $r_b = 27$ [mm] Radius of beam section fillet
 $A_b = 14300$ [mm²] Cross-sectional area of a beam
 $I_{xb} = 330900000$ [mm⁴] Moment of inertia of the beam section
 Material: STEEL
 $f_{yb} = 265.00$ [MPa] Resistance

Bolts

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

$d = 22$ [mm] Bolt diameter
 Class = 10.9 Bolt class
 $F_{tRd} = 218.16$ [kN] Tensile resistance of a bolt
 $n_h = 2$ Number of bolt columns
 $n_v = 4$ Number of bolt rows
 $h_1 = 73$ [mm] Distance between first bolt and upper edge of front plate
 Horizontal spacing $e_1 = 100$ [mm]
 Vertical spacing $p_1 = 80; 140; 140$ [mm]

Plate

$h_p = 540$ [mm] Plate height
 $b_p = 300$ [mm] Plate width
 $t_p = 20$ [mm] Plate thickness
 Material: STEEL
 $f_{yp} = 265.00$ [MPa] Resistance

Lower stiffener

$w_d = 300$ [mm] Plate width
 $t_{fd} = 20$ [mm] Flange thickness
 $h_d = 140$ [mm] Plate height
 $t_{wd} = 10$ [mm] Web thickness
 $l_d = 400$ [mm] Plate length
 $\alpha = 21.9$ [Deg] Inclination angle
 Material: STEEL
 $f_{ybu} = 265.00$ [MPa] Resistance

Column stiffener**Upper**

$h_{su} = 315$ [mm] Stiffener height
 $b_{su} = 145$ [mm] Stiffener width
 $t_{hu} = 8$ [mm] Stiffener thickness
 Material: STEEL
 $f_{ysu} = 265.00$ [MPa] Resistance

Lower

$h_{sd} = 315$ [mm] Stiffener height
 $b_{sd} = 145$ [mm] Stiffener width
 $t_{hd} = 8$ [mm] Stiffener thickness
 Material: STEEL
 $f_{ysu} = 265.00$ [MPa] Resistance

Fillet welds

$a_w = 5$ [mm] Web weld
 $a_f = 8$ [mm] Flange weld
 $a_s = 5$ [mm] Stiffener weld

$a_w = 5 \text{ [mm]}$

Web weld

$a_{fd} = 5 \text{ [mm]}$

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: Manual calculations.

$M_{b1,Ed} = 215.00 \text{ [kN*m]}$ Bending moment in the right beam

$V_{b1,Ed} = 91.00 \text{ [kN]}$ Shear force in the right beam

$N_{b1,Ed} = -15.00 \text{ [kN]}$ Axial force in the right beam

Results**Beam resistances****COMPRESSION**

$A_b = 14300 \text{ [mm}^2\text{]}$ Area

EN1993-1-1:[6.2.4]

$N_{cb,Rd} = A_b f_{yb} / \gamma_{M0}$

$N_{cb,Rd} = 3789.50 \text{ [kN]}$ Design compressive resistance of the section

EN1993-1-1:[6.2.4]

SHEAR

$A_{vb} = 6320 \text{ [mm}^2\text{]}$ Shear area

EN1993-1-1:[6.2.6.(3)]

$V_{cb,Rd} = A_{vb} (f_{yb} / \sqrt{3}) / \gamma_{M0}$

$V_{cb,Rd} = 966.95 \text{ [kN]}$ 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)

$W_{plb} = 2088000 \text{ [mm}^3\text{]}$ Plastic section modulus

EN1993-1-1:[6.2.5.(2)]

$M_{b,pl,Rd} = W_{plb} f_{yb} / \gamma_{M0}$

$M_{b,pl,Rd} = 553.32 \text{ [kN*m]}$ 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

$W_{pl} = 3234996 \text{ [mm}^3\text{]}$ Plastic section modulus

EN1993-1-1:[6.2.5]

$M_{cb,Rd} = W_{pl} f_{yb} / \gamma_{M0}$

$M_{cb,Rd} = 857.27 \text{ [kN*m]}$ Design resistance of the section for bending

EN1993-1-1:[6.2.5]

FLANGE AND WEB - COMPRESSION

$M_{cb,Rd} = 857.27 \text{ [kN*m]}$ Design resistance of the section for bending

EN1993-1-1:[6.2.5]

$h_f = 471 \text{ [mm]}$ Distance between the centroids of flanges

[6.2.6.7.(1)]

$F_{c,fb,Rd} = M_{cb,Rd} / h_f$

$F_{c,fb,Rd} = 1820.36 \text{ [kN]}$ Resistance of the compressed flange and web

[6.2.6.7.(1)]

WEB OR BRACKET FLANGE - COMPRESSION - LEVEL OF THE BEAM BOTTOM FLANGE

Bearing:

$\beta = 3.0 \text{ [Deg]}$ Angle between the front plate and the beam

$\gamma = 21.9 \text{ [Deg]}$ Inclination angle of the bracket plate

$b_{eff,c,wb} = 307 \text{ [mm]}$ Effective width of the web for compression

[6.2.6.2.(1)]

$A_{vb} = 4920 \text{ [mm}^2\text{]}$ Shear area

EN1993-1-1:[6.2.6.(3)]

$\omega = 0.82$ Reduction factor for interaction with shear

[6.2.6.2.(1)]

$\sigma_{com,Ed} = 85.84 \text{ [MPa]}$ 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)]

$F_{c,wb,Rd1} = [\omega k_{wc} b_{eff,c,wb} t_{wb} f_{yb} / \gamma_{M0}] \cos(\gamma) / \sin(\gamma - \beta)$

$F_{c,wb,Rd1} = 1892.91 \text{ [kN]}$ Beam web resistance

[6.2.6.2.(1)]

$d_{wb} =$	261	[mm]	Height of compressed web	[6.2.6.2.(1)]
$\lambda_p =$	0.95		Plate slenderness of an element	[6.2.6.2.(1)]
$\rho =$	0.83		Reduction factor for element buckling	[6.2.6.2.(1)]

$$F_{c,wb,Rd2} = [\omega k_{wc} \rho b_{eff,c,wb} t_{wb} f_{yb} / \gamma_{M1} \cos(\gamma) / \sin(\gamma - \beta)]$$

$$F_{c,wb,Rd2} = 1575.34 \quad [kN] \quad \text{Beam web resistance} \quad [6.2.6.2.(1)]$$

Resistance of the bracket flange

$$F_{c,wb,Rd3} = b_b t_b f_{yb} / (0.8 \gamma_{M0})$$

$$F_{c,wb,Rd3} = 1987.50 \quad [kN] \quad \text{Resistance of the bracket flange} \quad [6.2.6.7.(1)]$$

Final resistance:

$$F_{c,wb,Rd,low} = \min(F_{c,wb,Rd1}, F_{c,wb,Rd2}, F_{c,wb,Rd3})$$

$$F_{c,wb,Rd,low} = 1575.34 \quad [kN] \quad \text{Beam web resistance} \quad [6.2.6.2.(1)]$$

Column resistances**WEB PANEL - SHEAR**

$$M_{b1,Ed} = 215.00 \quad [kN \cdot m] \quad \text{Bending moment (right beam)} \quad [5.3.(3)]$$

$$M_{b2,Ed} = 0.00 \quad [kN \cdot m] \quad \text{Bending moment (left beam)} \quad [5.3.(3)]$$

$$V_{c1,Ed} = 0.00 \quad [kN] \quad \text{Shear force (lower column)} \quad [5.3.(3)]$$

$$V_{c2,Ed} = 0.00 \quad [kN] \quad \text{Shear force (upper column)} \quad [5.3.(3)]$$

$$z = 377 \quad [mm] \quad \text{Lever arm} \quad [6.2.5]$$

$$V_{wp,Ed} = (M_{b1,Ed} - M_{b2,Ed}) / z - (V_{c1,Ed} - V_{c2,Ed}) / 2$$

$$V_{wp,Ed} = 570.75 \quad [kN] \quad \text{Shear force acting on the web panel} \quad [5.3.(3)]$$

$$A_{vs} = 4920 \quad [mm^2] \quad \text{Shear area of the column web} \quad \text{EN1993-1-1:[6.2.6.(3)]}$$

$$A_{vc} = 4920 \quad [mm^2] \quad \text{Shear area} \quad \text{EN1993-1-1:[6.2.6.(3)]}$$

$$d_s = 512 \quad [mm] \quad \text{Distance between the centroids of stiffeners} \quad [6.2.6.1.(4)]$$

$$M_{pl,fc,Rd} = 6.09 \quad [kN \cdot m] \quad \text{Plastic resistance of the column flange for bending} \quad [6.2.6.1.(4)]$$

$$M_{pl,stu,Rd} = 1.27 \quad [kN \cdot m] \quad \text{Plastic resistance of the upper transverse stiffener for bending} \quad [6.2.6.1.(4)]$$

$$M_{pl,sti,Rd} = 1.27 \quad [kN \cdot m] \quad \text{Plastic resistance of the lower transverse stiffener for bending} \quad [6.2.6.1.(4)]$$

$$V_{wp,Rd} = 0.9 (A_{vs} f_{y,wc}) / (\sqrt{3} \gamma_{M0}) + \min(4 M_{pl,fc,Rd} / d_s, (2 M_{pl,fc,Rd} + M_{pl,stu,Rd} + M_{pl,sti,Rd}) / d_s)$$

$$V_{wp,Rd} = 706.22 \quad [kN] \quad \text{Resistance of the column web panel for shear} \quad [6.2.6.1]$$

$$V_{wp,Ed} / V_{wp,Rd} \leq 1.0 \quad 0.81 < 1.00 \quad \text{verified} \quad (0.81)$$

WEB - TRANSVERSE COMPRESSION - LEVEL OF THE BEAM BOTTOM FLANGE

Bearing:

$$t_{wc} = 10 \quad [mm] \quad \text{Effective thickness of the column web} \quad [6.2.6.2.(6)]$$

$$b_{eff,c,wc} = 307 \quad [mm] \quad \text{Effective width of the web for compression} \quad [6.2.6.2.(1)]$$

$$A_{vc} = 4920 \quad [mm^2] \quad \text{Shear area} \quad \text{EN1993-1-1:[6.2.6.(3)]}$$

$$\omega = 0.82 \quad \text{Reduction factor for interaction with shear} \quad [6.2.6.2.(1)]$$

$$\sigma_{com,Ed} = 0.00 \quad [MPa] \quad \text{Maximum compressive stress in web} \quad [6.2.6.2.(2)]$$

$$k_{wc} = 1.00 \quad \text{Reduction factor conditioned by compressive stresses} \quad [6.2.6.2.(2)]$$

$$A_s = 1856 \quad [mm^2] \quad \text{Area of the web stiffener} \quad \text{EN1993-1-1:[6.2.4]}$$

$$F_{c,wc,Rd1} = \omega k_{wc} b_{eff,c,wc} t_{wc} f_{yc} / \gamma_{M0} + A_s f_{ys} / \gamma_{M0}$$

$$F_{c,wc,Rd1} = 1154.18 \quad [kN] \quad \text{Column web resistance} \quad [6.2.6.2.(1)]$$

Buckling:

$$d_{wc} = 261 \quad [mm] \quad \text{Height of compressed web} \quad [6.2.6.2.(1)]$$

$$\lambda_p = 0.95 \quad \text{Plate slenderness of an element} \quad [6.2.6.2.(1)]$$

$$\rho = 0.83 \quad \text{Reduction factor for element buckling} \quad [6.2.6.2.(1)]$$

$$\lambda_s = 2.96 \quad \text{Stiffener slenderness} \quad \text{EN1993-1-1:[6.3.1.2]}$$

$$\chi_s = 1.00 \quad \text{Buckling coefficient of the stiffener} \quad \text{EN1993-1-1:[6.3.1.2]}$$

$$F_{c,wc,Rd2} = \omega k_{wc} \rho b_{eff,c,wc} t_{wc} f_{yc} / \gamma_{M1} + A_s \chi_s f_{ys} / \gamma_{M1}$$

$$F_{c,wc,Rd2} = 1043.07 \quad [kN] \quad \text{Column web resistance} \quad [6.2.6.2.(1)]$$

Final resistance:

$$F_{c,wc,Rd,low} = \min(F_{c,wc,Rd1}, F_{c,wc,Rd2})$$

$$F_{c,wc,Rd} = 1043.07 \quad [\text{kN}] \quad \text{Column web resistance} \quad [6.2.6.2.(1)]$$

WEB - TRANSVERSE COMPRESSION - LEVEL OF THE BEAM TOP FLANGE

Bearing:

$$t_{wc} = 10 \quad [\text{mm}] \quad \text{Effective thickness of the column web} \quad [6.2.6.2.(6)]$$

$$b_{eff,c,wc} = 303 \quad [\text{mm}] \quad \text{Effective width of the web for compression} \quad [6.2.6.2.(1)]$$

$$A_{vc} = 4920 \quad [\text{mm}^2] \quad \text{Shear area} \quad \text{EN1993-1-1:[6.2.6.(3)]}$$

$$\omega = 0.82 \quad \text{Reduction factor for interaction with shear} \quad [6.2.6.2.(1)]$$

$$\sigma_{com,Ed} = 0.00 \quad [\text{MPa}] \quad \text{Maximum compressive stress in web} \quad [6.2.6.2.(2)]$$

$$k_{wc} = 1.00 \quad \text{Reduction factor conditioned by compressive stresses} \quad [6.2.6.2.(2)]$$

$$A_s = 1856 \quad [\text{mm}^2] \quad \text{Area of the web stiffener} \quad \text{EN1993-1-1:[6.2.4]}$$

$$F_{c,wc,Rd1} = \omega k_{wc} b_{eff,c,wc} t_{wc} f_{yc} / \gamma_{M0} + A_s f_{ys} / \gamma_{M0}$$

$$F_{c,wc,Rd1} = 1148.35 \quad [\text{kN}] \quad \text{Column web resistance} \quad [6.2.6.2.(1)]$$

Buckling:

$$d_{wc} = 261 \quad [\text{mm}] \quad \text{Height of compressed web} \quad [6.2.6.2.(1)]$$

$$\lambda_p = 0.94 \quad \text{Plate slenderness of an element} \quad [6.2.6.2.(1)]$$

$$\rho = 0.84 \quad \text{Reduction factor for element buckling} \quad [6.2.6.2.(1)]$$

$$\lambda_s = 2.96 \quad \text{Stiffener slenderness} \quad \text{EN1993-1-1:[6.3.1.2]}$$

$$\chi_s = 1.00 \quad \text{Buckling coefficient of the stiffener} \quad \text{EN1993-1-1:[6.3.1.2]}$$

$$F_{c,wc,Rd2} = \omega k_{wc} \rho b_{eff,c,wc} t_{wc} f_{yc} / \gamma_{M1} + A_s \chi_s f_{ys} / \gamma_{M1}$$

$$F_{c,wc,Rd2} = 1040.88 \quad [\text{kN}] \quad \text{Column web resistance} \quad [6.2.6.2.(1)]$$

Final resistance:

$$F_{c,wc,Rd,upp} = \min(F_{c,wc,Rd1}, F_{c,wc,Rd2})$$

$$F_{c,wc,Rd,upp} = 1040.88 \quad [\text{kN}] \quad \text{Column web resistance} \quad [6.2.6.2.(1)]$$

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	23	-	100	-	80	147	187	147	187	154	118	118	118
2	23	-	100	-	110	147	219	147	219	220	110	110	110
3	23	-	100	-	140	147	219	147	219	280	140	140	140
4	23	-	100	-	140	147	187	147	187	214	148	148	148

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	39	-	100	-	80	247	315	247	315	204	214	204	214
2	39	-	100	-	110	247	282	247	282	220	110	110	110
3	39	-	100	-	140	247	282	247	282	280	140	140	140
4	39	-	100	-	140	247	282	247	282	264	211	211	211

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

- m – Bolt distance from the web
- $l_{eff,1,g}$ – 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

$$N_{j,Rd} = \min (N_{cb,Rd} 2 F_{c,wb,Rd,low}, 2 F_{c,wc,Rd,low}, 2 F_{c,wc,Rd,upp})$$

$$N_{j,Rd} = 2081.75 \quad [kN] \quad \text{Connection resistance for compression} \quad [6.2]$$

$$N_{b1,Ed} / N_{j,Rd} \leq 1.0 \quad 0.01 < 1.00 \quad \text{verified} \quad (0.01)$$

Connection resistance for bending

$$F_{t,Rd} = 218.16 \quad [kN] \quad \text{Bolt resistance for tension} \quad [Table 3.4]$$

$$B_{p,Rd} = 374.47 \quad [kN] \quad \text{Punching shear resistance of a bolt} \quad [Table 3.4]$$

$F_{t,fc,Rd}$ – 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

$$F_{t,fc,Rd} = \min (F_{T,1,fc,Rd}, F_{T,2,fc,Rd}, F_{T,3,fc,Rd}) \quad [6.2.6.4], [Tab.6.2]$$

$$F_{t,wc,Rd} = m h_{eff,t,wc} t_{wo} f_{yo} / \gamma_{MO} \quad [6.2.6.3.(1)]$$

$$F_{t,ep,Rd} = \min (F_{T,1,ep,Rd}, F_{T,2,ep,Rd}, F_{T,3,ep,Rd}) \quad [6.2.6.5], [Tab.6.2]$$

$$F_{t,wb,Rd} = b_{eff,t,wb} t_{wb} f_{yb} / \gamma_{MO} \quad [6.2.6.8.(1)]$$

RESISTANCE OF THE BOLT ROW NO. 1

$F_{t1,Rd,comp}$ - Formula	$F_{t1,Rd,comp}$	Component
$F_{t1,Rd} = \min (F_{t1,Rd,comp})$	368.80	Bolt row resistance
$F_{t,fc,Rd(1)} = 386.68$	386.68	Column flange - tension
$F_{t,wc,Rd(1)} = 368.80$	368.80	Column web - tension
$F_{t,ep,Rd(1)} = 430.84$	430.84	Front plate - tension
$F_{t,wb,Rd(1)} = 655.08$	655.08	Beam web - tension
$B_{p,Rd} = 748.93$	748.93	Bolts due to shear punching
$V_{wp,Rd}/\beta = 706.22$	706.22	Web panel - shear
$F_{c,wc,Rd} = 1043.07$	1043.07	Column web - compression
$F_{c,fb,Rd} = 1820.36$	1820.36	Beam flange - compression
$F_{c,wb,Rd} = 1575.34$	1575.34	Beam web - compression

RESISTANCE OF THE BOLT ROW NO. 2

$F_{t2,Rd,comp}$ - Formula	$F_{t2,Rd,comp}$	Component
$F_{t2,Rd} = \min (F_{t2,Rd,comp})$	165.23	Bolt row resistance
$F_{t,fc,Rd(2)} = 410.88$	410.88	Column flange - tension
$F_{t,wc,Rd(2)} = 368.80$	368.80	Column web - tension
$F_{t,ep,Rd(2)} = 411.46$	411.46	Front plate - tension
$F_{t,wb,Rd(2)} = 655.08$	655.08	Beam web - tension
$B_{p,Rd} = 748.93$	748.93	Bolts due to shear punching
$V_{wp,Rd}/\beta - \sum_1^1 F_{ij,Rd} = 706.22 - 368.80$	337.42	Web panel - shear
$F_{c,wc,Rd} - \sum_1^1 F_{ij,Rd} = 1043.07 - 368.80$	674.27	Column web - compression
$F_{c,fb,Rd} - \sum_1^1 F_{ij,Rd} = 1820.36 - 368.80$	1451.56	Beam flange - compression
$F_{c,wb,Rd} - \sum_1^1 F_{ij,Rd} = 1575.34 - 368.80$	1206.54	Beam web - compression
$F_{t,fc,Rd(2+1)} - \sum_1^1 F_{ij,Rd} = 660.45 - 368.80$	291.65	Column flange - tension - group
$F_{t,wc,Rd(2+1)} - \sum_1^1 F_{ij,Rd} = 534.03 - 368.80$	165.23	Column web - tension - group
$F_{t,ep,Rd(2+1)} - \sum_1^1 F_{ij,Rd} = 678.52 - 368.80$	309.72	Front plate - tension - group
$F_{t,wb,Rd(2+1)} - \sum_1^1 F_{ij,Rd} = 857.43 - 368.80$	488.63	Beam web - tension - group

RESISTANCE OF THE BOLT ROW NO. 3

F_{t3,Rd,comp} - Formula	F_{t3,Rd,comp}	Component
$F_{t3,Rd} = \text{Min} (F_{t3,Rd,comp})$	172.19	Bolt row resistance
$F_{t,fc,Rd(3)} = 410.88$	410.88	Column flange - tension
$F_{t,wc,Rd(3)} = 368.80$	368.80	Column web - tension
$F_{t,ep,Rd(3)} = 411.46$	411.46	Front plate - tension
$F_{t,wb,Rd(3)} = 655.08$	655.08	Beam web - tension
$B_{p,Rd} = 748.93$	748.93	Bolts due to shear punching
$V_{wp,Rd}/\beta - \sum_1^2 F_{tj,Rd} = 706.22 - 534.03$	172.19	Web panel - shear
$F_{c,wc,Rd} - \sum_1^2 F_{tj,Rd} = 1043.07 - 534.03$	509.04	Column web - compression
$F_{c,fb,Rd} - \sum_1^2 F_{tj,Rd} = 1820.36 - 534.03$	1286.33	Beam flange - compression
$F_{c,wb,Rd} - \sum_1^2 F_{tj,Rd} = 1575.34 - 534.03$	1041.31	Beam web - compression
$F_{t,fc,Rd(3+2)} - \sum_2^2 F_{tj,Rd} = 677.48 - 165.23$	512.25	Column flange - tension - group
$F_{t,wc,Rd(3+2)} - \sum_2^2 F_{tj,Rd} = 573.24 - 165.23$	408.01	Column web - tension - group
$F_{t,fc,Rd(3+2+1)} - \sum_2^1 F_{tj,Rd} = 1010.75 - 534.03$	476.72	Column flange - tension - group
$F_{t,wc,Rd(3+2+1)} - \sum_2^1 F_{tj,Rd} = 741.90 - 534.03$	207.86	Column web - tension - group
$F_{t,ep,Rd(3+2)} - \sum_2^2 F_{tj,Rd} = 634.48 - 165.23$	469.25	Front plate - tension - group
$F_{t,wb,Rd(3+2)} - \sum_2^2 F_{tj,Rd} = 662.50 - 165.23$	497.27	Beam web - tension - group
$F_{t,ep,Rd(3+2+1)} - \sum_2^1 F_{tj,Rd} = 1004.74 - 534.03$	470.71	Front plate - tension - group
$F_{t,wb,Rd(3+2+1)} - \sum_2^1 F_{tj,Rd} = 1228.43 - 534.03$	694.40	Beam web - tension - group

RESISTANCE OF THE BOLT ROW NO. 4

F_{t4,Rd,comp} - Formula	F_{t4,Rd,comp}	Component
$F_{t4,Rd} = \text{Min} (F_{t4,Rd,comp})$	0.00	Bolt row resistance
$F_{t,fc,Rd(4)} = 386.68$	386.68	Column flange - tension
$F_{t,wc,Rd(4)} = 368.80$	368.80	Column web - tension
$F_{t,ep,Rd(4)} = 411.46$	411.46	Front plate - tension
$F_{t,wb,Rd(4)} = 655.08$	655.08	Beam web - tension
$B_{p,Rd} = 748.93$	748.93	Bolts due to shear punching
$V_{wp,Rd}/\beta - \sum_1^3 F_{tj,Rd} = 706.22 - 706.22$	0.00	Web panel - shear
$F_{c,wc,Rd} - \sum_1^3 F_{tj,Rd} = 1043.07 - 706.22$	336.86	Column web - compression
$F_{c,fb,Rd} - \sum_1^3 F_{tj,Rd} = 1820.36 - 706.22$	1114.14	Beam flange - compression
$F_{c,wb,Rd} - \sum_1^3 F_{tj,Rd} = 1575.34 - 706.22$	869.12	Beam web - compression
$F_{t,fc,Rd(4+3)} - \sum_3^3 F_{tj,Rd} = 706.69 - 172.19$	534.50	Column flange - tension - group
$F_{t,wc,Rd(4+3)} - \sum_3^3 F_{tj,Rd} = 634.65 - 172.19$	462.46	Column web - tension - group
$F_{t,fc,Rd(4+3+2)} - \sum_3^2 F_{tj,Rd} = 1033.87 - 337.42$	696.45	Column flange - tension - group
$F_{t,wc,Rd(4+3+2)} - \sum_3^2 F_{tj,Rd} = 775.18 - 337.42$	437.76	Column web - tension - group
$F_{t,fc,Rd(4+3+2+1)} - \sum_3^1 F_{tj,Rd} = 1367.13 - 706.22$	660.92	Column flange - tension - group
$F_{t,wc,Rd(4+3+2+1)} - \sum_3^1 F_{tj,Rd} = 877.06 - 706.22$	170.84	Column web - tension - group
$F_{t,ep,Rd(4+3)} - \sum_3^3 F_{tj,Rd} = 695.06 - 172.19$	522.87	Front plate - tension - group
$F_{t,wb,Rd(4+3)} - \sum_3^3 F_{tj,Rd} = 930.64 - 172.19$	758.45	Beam web - tension - group
$F_{t,ep,Rd(4+3+2)} - \sum_3^2 F_{tj,Rd} = 1003.32 - 337.42$	665.90	Front plate - tension - group
$F_{t,wb,Rd(4+3+2)} - \sum_3^2 F_{tj,Rd} = 1222.14 - 337.42$	884.72	Beam web - tension - group
$F_{t,ep,Rd(4+3+2+1)} - \sum_3^1 F_{tj,Rd} = 1373.58 - 706.22$	667.36	Front plate - tension - group
$F_{t,wb,Rd(4+3+2+1)} - \sum_3^1 F_{tj,Rd} = 1788.07 - 706.22$	1081.86	Beam web - tension - group

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	417	368.80	386.68	368.80	430.84	655.08	436.32	748.93
2	337	165.23	410.88	368.80	411.46	655.08	436.32	748.93
3	197	172.19	410.88	368.80	411.46	655.08	436.32	748.93

Nr	h_j	$F_{t,Rd}$	$F_{t,fc,Rd}$	$F_{t,wc,Rd}$	$F_{t,ep,Rd}$	$F_{t,wb,Rd}$	$F_{t,Rd}$	$B_{p,Rd}$
4	57	-	386.68	368.80	411.46	655.08	436.32	748.93

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

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

$$M_{j,Rd} = 243.18 \text{ [kN*m]} \text{ Connection resistance for bending}$$

[6.2]

$$M_{b1,Ed} / M_{j,Rd} \leq 1.0$$

$$0.88 < 1.00$$

verified

(0.88)

Connection resistance for shear

$$\alpha_v = 0.60 \text{ Coefficient for calculation of } F_{v,Rd}$$

[Table 3.4]

$$\beta_{Lr} = 0.99 \text{ Reduction factor for long connections}$$

[3.8]

$$F_{v,Rd} = 181.22 \text{ [kN]} \text{ Shear resistance of a single bolt}$$

[Table 3.4]

$$F_{t,Rd,max} = 218.16 \text{ [kN]} \text{ Tensile resistance of a single bolt}$$

[Table 3.4]

$$F_{b,Rd,int} = 285.11 \text{ [kN]} \text{ Bearing resistance of an intermediate bolt}$$

[Table 3.4]

$$F_{b,Rd,ext} = 331.10 \text{ [kN]} \text{ Bearing resistance of an outermost bolt}$$

[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	436.32	-3.75	368.80	326.06	322.31	171.20
2	436.32	-3.75	165.23	146.08	142.33	277.99
3	436.32	-3.75	172.19	152.23	148.48	274.34
4	436.32	-3.75	0.00	0.00	-3.75	362.44

$F_{tj,Rd,N}$ – 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

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

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

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

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

$$V_{j,Rd} = n_h \sum_1^n F_{vj,Rd}$$

[Table 3.4]

$$V_{j,Rd} = 1085.96 \text{ [kN]} \text{ Connection resistance for shear}$$

[Table 3.4]

$$V_{b1,Ed} / V_{j,Rd} \leq 1.0$$

$$0.08 < 1.00$$

verified

(0.08)

Weld resistance

$$A_w = 16854 \text{ [mm}^2\text{]} \text{ Area of all welds}$$

[4.5.3.2(2)]

$$A_{wy} = 13056 \text{ [mm}^2\text{]} \text{ Area of horizontal welds}$$

[4.5.3.2(2)]

$$A_{wz} = 3798 \text{ [mm}^2\text{]} \text{ Area of vertical welds}$$

[4.5.3.2(2)]

$$I_{wy} = 587989052 \text{ [mm}^4\text{]} \text{ Moment of inertia of the weld arrangement with respect to the hor. axis}$$

[4.5.3.2(5)]

$$\sigma_{\perp,max} = \tau_{\perp,max} = 71.48 \text{ [MPa]} \text{ Normal stress in a weld}$$

[4.5.3.2(5)]

$$\sigma_{\perp} = \tau_{\perp} = 58.83 \text{ [MPa]} \text{ Stress in a vertical weld}$$

[4.5.3.2(5)]

$$\tau_{\parallel} = 23.96 \text{ [MPa]} \text{ Tangent stress}$$

[4.5.3.2(5)]

$$\beta_w = 0.85 \text{ Correlation coefficient}$$

[4.5.3.2(7)]

$$\sqrt{(\sigma_{\perp,max}^2 + 3(\tau_{\perp,max}^2))} \leq f_u / (\beta_w \gamma_{M2})$$

$$142.95 < 404.71$$

verified

(0.35)

$$\sqrt{(\sigma_{\perp}^2 + 3(\tau_{\perp}^2 + \tau_{\parallel}^2))} \leq f_u / (\beta_w \gamma_{M2})$$

$$124.77 < 404.71$$

verified

(0.31)

$$\sigma_{\perp} \leq 0.9 f_u / \gamma_{M2}$$

$$71.48 < 309.60$$

verified

(0.23)

Connection stiffness

$$t_{wash} = 5 \text{ [mm]} \text{ Washer thickness}$$

[6.2.6.3.(2)]

$$h_{head} = 16 \text{ [mm]} \text{ Bolt head height}$$

[6.2.6.3.(2)]

$$h_{nut} = 22 \text{ [mm]} \text{ Bolt nut height}$$

[6.2.6.3.(2)]

$$L_b = 67 \text{ [mm]} \text{ Bolt length}$$

[6.2.6.3.(2)]

$t_{wash} = 5$ [mm]
 $k_{10} = 7$ [mm] Stiffness coefficient of bolts

[6.3.2.(1)]

STIFFNESSES OF BOLT ROWS

Nr	h_j	k_3	k_4	k_5	$k_{eff,j}$	$k_{eff,j} h_j$	$k_{eff,j} h_j^2$
					Sum	1714	551341
1	417	3	44	24	2	715	297875
2	337	2	41	13	2	520	175137
3	197	3	53	17	2	366	71917
4	57	3	55	25	2	113	6412

$$k_{eff,j} = 1 / (\sum_3^b (1 / k_{i,j})) \quad [6.3.3.1.(2)]$$

$$z_{eq} = \sum_j k_{eff,j} h_j^2 / \sum_j k_{eff,j} h_j$$

$z_{eq} = 322$ [mm] Equivalent force arm [6.3.3.1.(3)]

$$k_{eq} = \sum_j k_{eff,j} h_j / z_{eq}$$

$k_{eq} = 5$ [mm] Equivalent stiffness coefficient of a bolt arrangement [6.3.3.1.(1)]

$A_{vc} = 4920$ [mm²] Shear area EN1993-1-1:[6.2.6.(3)]

$\beta = 1.00$ Transformation parameter [5.3.(7)]

$z = 322$ [mm] Lever arm [6.2.5]

$k_1 = 6$ [mm] Stiffness coefficient of the column web panel subjected to shear [6.3.2.(1)]

$k_2 =$ Stiffness coefficient of the compressed column web [6.3.2.(1)]

$$S_{j,ini} = E z_{eq}^2 / \sum_j (1 / k_1 + 1 / k_2 + 1 / k_{eq}) \quad [6.3.1.(4)]$$

$S_{j,ini} = 58970.95$ [kN*m] Initial rotational stiffness [6.3.1.(4)]

$\mu = 2.14$ Stiffness coefficient of a connection [6.3.1.(6)]

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

$S_j = 27518.10$ [kN*m] Final rotational stiffness [6.3.1.(4)]

Connection classification due to stiffness.

$S_{j,rig} = 108535.20$ [kN*m] Stiffness of a rigid connection [5.2.2.5]

$S_{j,pin} = 6783.45$ [kN*m] Stiffness of a pinned connection [5.2.2.5]

$S_{j,pin} \leq S_{j,ini} < S_{j,rig}$ SEMI-RIGID

Weakest component:

COLUMN WEB PANEL - SHEAR

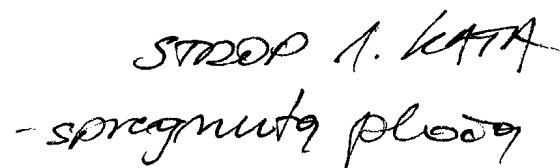
Connection conforms to the code	Ratio 0.88
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ARMIRANOBETONSKI ELEMENTI

KONSTRUKCIJE

beton C25/30; armatura B500B

(stubišta, spregnute ploče, temelji, podna ploča..)



PO7 SP1

PRORAČUN SPREGNUTE PLOČE

1. UVOD

Proračun spregnute ploče biti će proveden za ploču širine 100 cm, te kao niz slobodno oslonjenih greda preko više oslonaca. Spregnuta ploča se izvodi čeličnim limom HiBond 55, $t=0,8$ mm, S250 (TRIMO d.d.) U gornju zonu ploče nad ležajevima se stavlja armaturna mreža Q-188 sa 2.5 cm zaštitnog sloja.

2. DIMENZIJE ELEMENTA

širina	$b =$	100 cm
visina spregnute ploče	$h =$	15 cm
efektivna visina spregnute ploče	$d_p =$	12,25 cm
raspon	$l =$	4,20 m
zaštitni sloj	$c =$	2,5 cm

3. KAKVOĆA MATERIJALA

beton

C25/30	$f_{ck} =$	25	N/mm ²
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armatura

MAR 500/560	$f_{yk} =$	50	kN/cm ²
	$E_s =$	210000	N/mm ²

čelični trapezni lim

HiBond 55, $t=0,8$ mm, S250 GD (EN 10147:2000)

$f_{yp} =$	25,00	kN/cm ²
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$W_{p,eff,max} =$	20,9	cm ³
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$W_{p,eff,min} =$	16,9	cm ³
-------------------	------	-----------------

$W_{pl} =$	23,8	cm ³
------------	------	-----------------

$I_{p,eff} =$	50,7	cm ⁴
---------------	------	-----------------

$I_p =$	60,3	cm ⁴
---------	------	-----------------

duljina vala 150 mm

$t_p =$	0,8	mm
---------	-----	----

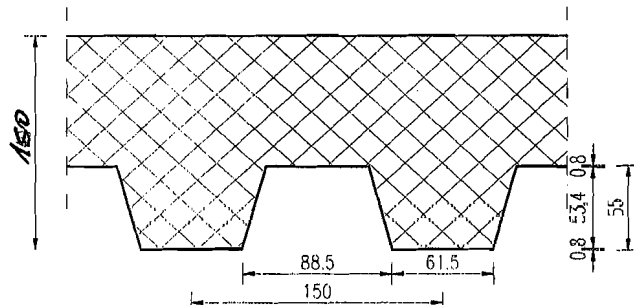
$h_p =$	55	mm
---------	----	----

$b_p =$	61,5	mm
---------	------	----

$A_p =$	12,2	cm ²
---------	------	-----------------

$A_{pe,ff} =$	11,0	cm ²
---------------	------	-----------------

$\phi =$	76,25°
----------	--------



karakterističan poprečni presjek

4. DJELOVANJA

vlastita težina lima

$q_{llma} =$	0,10	kN/m ²
--------------	------	-------------------

vlastita težina betona

$q_c =$	3,06	kN/m ²
---------	------	-------------------

radno opterećenje

$q_{m1} =$	1,50	kN/m ²
------------	------	-------------------

(u fazi betoniranja)

$q_{m2} =$	0,75	kN/m ²
------------	------	-------------------

korisno opterećenje

$q =$	3,00	kN/m ²
-------	------	-------------------

spregnuti presjek preuzima dodatno stalno i korisno opterećenje

-spregnuta ploča se računa kao niz slobodno oslonjenih greda

vlastita težina	3,06	kN/m ²
dodatno stalno	0,50	kN/m ²
korisno opterećenje	3,00	kN/m ²

Računsko opterećenje

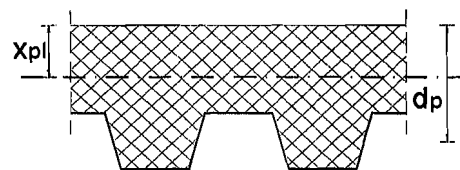
$$p_{sd} = (\gamma_g g + \gamma_q q) b = (1,35g + 1,5q) 1,00 = 9,31 \text{ kN/m}$$

6. Dimenzioniranje

6.1. otpornost spregnute ploče na savijanje za KGS

$$M_{pl,Rd} > M_{y,Sd}$$

$$M_{y,Sd} = p_{sd} l^2 / 8 = 20,53 \text{ kNm}$$



poprečni presjek sa položajem neutralne osi

izračun plastične neutralne osi

$$x_{pl} = \frac{A_p f_{yp} / \gamma_{ap}}{b \cdot 0,85 f_{ck} / \gamma_c} = 1,96 \text{ cm} < 9,50 \text{ cm}$$

neutralna os je u ploči

proračun momenta otpornosti

$$M_{pl,Rd} = A_p \frac{f_{ypk}}{\gamma_{ap}} (d_p - \frac{x_{pl}}{2}) = 31,25 \text{ kNm}$$

$$d_p = d - h_p / 2 = 122,5 \text{ mm}$$

Provjera

$$M_{Sd} = 20,5 \text{ kNm} < 31,25 \text{ kNm}$$

ZADOVOLJAVA

6.2. proračun na podužno smicanje

$$M_{Rd} = N_c z_1 + N_{as} z_2 + M_{pr} > M_{y,Sd}$$

Utjecaj od dodatne armature u rebrima
(nema dodatne armature)

$$A_s = 0,00 \text{ cm}^2$$

$$N_{as} = A_s \frac{f_{sk}}{\gamma_s} = 0,00 \text{ kN}$$

uvjet: $A_s \frac{f_{sk}}{\gamma_s} \leq 0,7 A_p \frac{f_{yp}}{\gamma_a}$

$$0,0 \text{ kN} < 194,1 \text{ kN}$$

ZADOVOLJAVA

$$N_c = N_{c1} + N_{c2} + N_{c3} \leq N_{cf} = N_p$$

Utjecaj od sprezanja limom

$$N_{c1} = 0$$

ne uzima se u obzir sprezanje limom zbog nedostatka eksperimentalnih podataka

Utjecaj trenja na osloncu

$$N_{c2} = \mu R \quad \mu = 0,50$$

R - reakcija na osloncu, za prostu gredu $R = V_{Sd}$

$$N_{c2} = 9,77 \text{ kN}$$

Utjecaj sidrenja na krajevima

sidrenje se vrši moždanicama tipa NELSON Φ 19

$$N_{c3} = V_{ld}$$

$$V_{ld} = P_{pb,Rd} = k_{\phi} d_{d0} t f_{yp} / \gamma_{ap} = 22,89 \text{ kN} \quad f_u = 45,00 \text{ kN/cm}^2$$

$$d = 19 \text{ mm}$$

$$k_{\phi} = 1 + a/d_{d0} \leq 6 \quad d_{d0} = 1,1 d = 2,09 \text{ cm} \quad d - \text{promjer moždanika}$$

$$a \geq 1,5 d_{d0} \quad a \geq 3,14 \text{ cm} \quad a - \text{udaljenost moždanika od ruba lima}$$

$$\text{odabrano } a = 10,50 \text{ cm} \quad (\text{za maksimalni } k_{\phi})$$

$$k_{\phi} = 6,0$$

$$\text{razmak moždanika } l_m = 15 \text{ cm} \quad (\text{moždanik u svakom valu})$$

$$N_{c3} = V_{ld} / l_m = 152,61 \text{ kN}$$

$$N_c = 162,38 \text{ kN}$$

$$z_1 = h - 0,5 x_{pl} - e_p + (e_p - e) N_c / (A_p f_{yp} / \gamma_{ap}) \quad e_p - \text{udaljenost od neutralne osi do donjeg ruba lima}$$

$$z_1 = 7,08 \text{ cm}$$

$$e - \text{udaljenost od težišta lima do donjeg ruba lima}$$

$$z_2 = d_s - 0,5 x_{pl}$$

$$e_p = 13,85 \text{ cm}$$

$$z_2 = 14,43 \text{ cm}$$

$$e = 2,75 \text{ cm}$$

$$d_s - \text{udaljenost armature od gornjeg ruba ploče} \approx h_{ploče}$$

$$x_{pl} = \frac{N_c + N_{as}}{b \cdot 0,85 f_{ck} / \gamma_c} \quad x_{pl} = 1,15 \text{ cm}$$

Utjecaj lima na nosivost na savijanje

$$M_{pr} = 1,25 M_{pa} \left(1 - \frac{N_c}{A_p \frac{f_{yp}}{\gamma_{ap}}} \right) \leq M_{pa} \quad \text{proračunski moment pune plastičnosti}$$

$$\text{za efektivni presjek lima}$$

$$M_{pr} = 2,46 \text{ kNm}$$

Provjera

$$M_{Rd} = N_c z_1 + N_{as} z_2 + M_{pr}$$

$$M_{Rd} = 22,10 \text{ kNm} > 20,53 \text{ kNm}$$

ZADOVOLJAVA

6.3. proračun na vertikalno smicanje

$$V_{v,Rd} = b_0 d_p \tau_{Rd} k_v (1,2 + 40\rho) = 55,42 \text{ kN}$$

$$b_0 = 500 \text{ mm} \quad (\text{srednja širina svih rebara unutar 1m})$$

$$\tau_{Rd} = 0,25 f_{ctk} / \gamma_c = 0,30 \text{ N/mm}^2$$

$$k_v = 1,6 - d_p = 1,48 \text{ mm} \quad (d_p \text{ u metrima})$$

$$\rho = (A_p + A_s) / b_0 d_p = 0,0210 < 0,02$$

Provjera

$$V_{sd} = 19,5 \text{ kN} < 55,42 \text{ kN} = V_{v,Rd}$$

ZADOVOLJAVA

6.4. poprečna armatura - kontrola uzdužnog posmika

- otkazivanje nosivosti moždanika	$P_{Rd} \leq 0,8f_u \pi d^2 / (4\gamma_v)$	$P_{t,Rd} \leq$	81,61	kN
- drobljenje betona u okolini moždanika	$P_{Rd} \leq 0,29\alpha d^2 (f_{ck} E_{cm})^{1/2} / (\gamma_v)$	$P_{t,Rd} \leq$	72,53	kN
- mjerodavna nosivost (manja vrijednost pomnožena s k_r):		$P_{l,Rd} =$	61,66	kN
		$k_r =$	0,85	

Posmično djelovanje: $V_{Sd} = P_{Rd}/e = 61,66/0,15 = 411,07$ kN/m

$$V_{Sd}/2 = 205,53 \text{ kN/m (jedna strana čeličnog nosača)}$$

Proračunska nosivost na posmično djelovanje: $V_{Rd} = 2,5A_{cv}\eta\tau_{Rd} + A_e f_{sk}/\gamma_s + v_{pd}$

$A_{cv} =$	1225	cm ² /m (površina betonskog presjeka)
$\eta =$	1	(za beton normalne težine)
$\tau_{Rd} =$	0,30	N/mm ²
$A_e = Q188+Q335 =$	5,23	cm ² /m (površina ukupne armature)
$f_{sk} =$	50	kN/cm ²
$v_{pd} = A_p f_{yp}/\gamma_{ap} =$	277,3	kN/m'

$V_{Rd} = 2,5 \times 1225 \times 0,03 + 5,23 \times 50 / 1,15 + 277,3 = 596,57 > 205,5 \text{ kN/m}' = V_{Sd}$
Predviđena armatura na ležaju spregnute ploče je dostatna za osiguranje nosivosti na uzdužni posmik.

6.5. Interakcija poprečnih sila na moždanik u dva okomita smjera

$F_l = 53,92$ kN računski poprečna sila od spregnutog nosača
 $F_t = P_{pb,Rd} = 22,89$ kN računski poprečna sila od spregnute ploče

$$\frac{F_l^2}{P_{l,Rd}^2} + \frac{F_t^2}{P_{t,Rd}^2} \leq 1$$

$P_{t,Rd} = 72,53$ kN računski nosivost moždanika
 $P_{l,Rd} = 61,65$ kN reducirana računski nosivost moždanika

$$0,865 < 1 \quad \text{ZADOVOLJAVA}$$

7. Granično stanje uporabivosti

7.1. ograničavanje širine pukotina

prema EC2 minimalna količina armature kada je spregnuta ploča prosta greda je 0,4% A_c za poduprte ploče, a 0,2% A_c za nepoduprte ploče

$A_s = 3,35$ cm² (Q-335 postavljena u gornju zonu nad ležajem)
armatura mora pokrivati minimalno 20% raspona

$$\rho = \frac{A_s}{b \cdot h_c} = 0,4 \% \geq 0,4 \%$$

ZADOVOLJAVA

odabrano Q-335 postavljena u gornju zonu nad ležajem

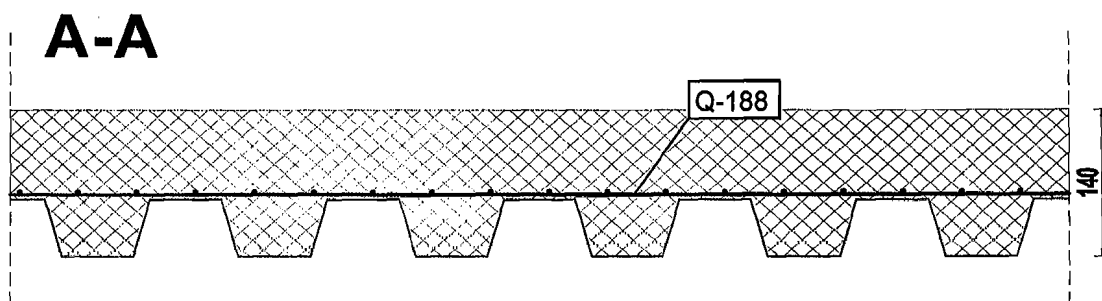
proračun dodatne armature

za prijenos eventualnih koncentriranih sila (pregradne stijene, lakša koncentrirana oprema, nejednoliko korisno opterećenje)

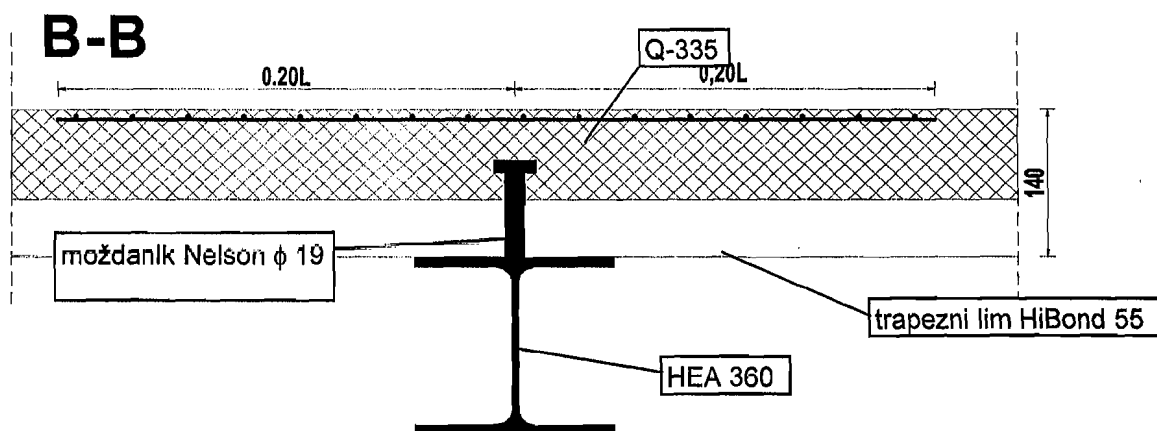
$$A_s = 0,002 b(h-h_{lima}) = 1,90 \text{ cm}^2 \quad (0,2\% A_c)$$

odabrano Q-188 postavljena na lim po cijeloj površini

Presjek spregnute ploče u polju

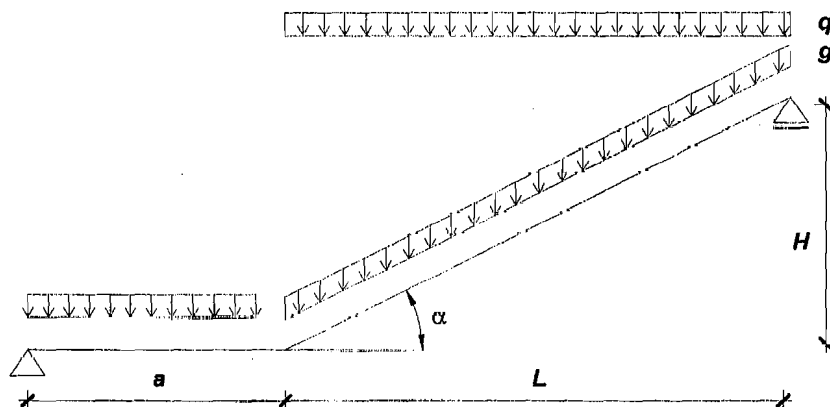


Presjek spregnute ploče na ležaju



PRORAČUN KRAKA ARMIRANOBETONSKOG STUBIŠTA

1. STATIČKI SUSTAV



dimenzije stubišta

L =	4,20	m
H =	2,78	m
$\alpha =$	32	°
a =	0,20	m
L' =	4,95	m

$$L_{uk} = a + L' = 5,15 \text{ m}$$

dimenzija stuba

$\bar{s} =$	26,00	cm
v =	18,00	cm

dimenzije ploče stubišta

širina b =	100	cm
visina h =	16	cm
zašt. sloj c =	2,0	cm
statička visina presjeka		
$d = h - \phi_1 - \phi_2 / 2 - c =$	12,70	cm

2. ANALIZA OPTEREĆENJA

- stalno opterećenje	ploča	0,16 x 25	4,00	kN/m ²
	stepenice	0,18 x 0,5x25	2,25	kN/m ²
	žbuka	0,01 x 19	0,19	kN/m ²
	obloga stepenica		0,10	kN/m ²
			g =	6,54 kN/m ²
- pokretno opterećenje			q =	3,00 kN/m ²
		$q' = q \cos \alpha$	q' =	2,54 kN/m ²
- računski vrijednost opterećenja		$q_{sd} = 1.35 g + 1.50 q'$	q_{sd} =	12,65 kN/m ²

3. REZNE SILE

- računski vrijednost momenta savijanja	$M_{sd} = q_{sd} L'^2 / 8$	M_{sd} =	41,96	kNm
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4. DIMENZIONIRANJE

C25/30
S 500 (MAG 500/560)

$f_{ck} =$	25	N/mm ²
$f_{yk} =$	50	kN/cm ²
$f_{cd} = f_{ck} / 1.5 =$	16,67	N/mm ²
$f_{yd} = f_{yk} / 1.15 =$	43,48	kN/cm ²

glavna vlačna armatura

$$\mu_{sd} = M_{sd} / (b d^2 f_{cd}) = 0,160 < \mu_{rd,lim} = 0,252$$

očitano:

koeficijent kraka unutrašnjih sila $\zeta =$	0,892	deformacija betona $\epsilon_{c2} =$	-3,50	‰
koeficijent položaja neutralne osi $\xi =$		deformacija čelika $\epsilon_{s1} =$	9,92	‰

$$A_{s1} = M_{sd} / (\zeta d f_{yd}) = 8,52 \text{ cm}^2$$

$$A_{s,min} = 0,0015 b d = 2,40 \text{ cm}^2$$

$$A_{s,max} = 0,04 A_c = 64,00 \text{ cm}^2$$

$$\text{odabrano: } \Phi 10 / 7,5 \text{ cm} \quad A_{s1} = 10,47 \text{ cm}^2$$

razdjelna armatura

$$A_{s,razdjelno} = 0,20 A_{s,odabrano} = 2,09 \text{ cm}^2$$

$$\text{odabrano: } \Phi 8 / 20 \text{ cm} \quad A_{s,razdjelno} = 2,51 \text{ cm}^2$$

ili Q-257 + $\Phi 10 / 10$ uzdužno

OPTEREĆENJA NA GREĐNICE

BETON C25/30

NIVO 5.78

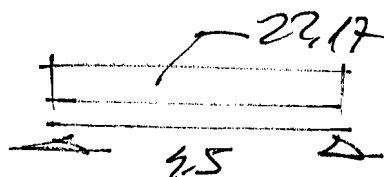
- po 92 - opterećenje

- od SP1 $(8,16 + 3) \times 3,0 / 2 = 9,24 \text{ kN/m}'$

- od v.l. težine

$$0,35 \times 0,16 \times 25 = 1,4 \text{ kN/m}'$$

- od opreme = $1,0 \text{ kN/m}'$



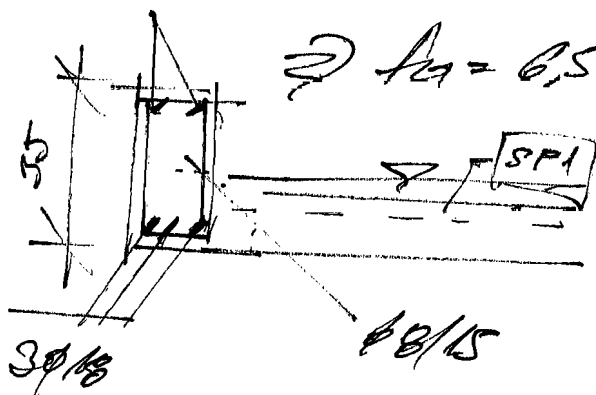
$$L = 22,17 \text{ kN/m}'$$

$$M = \frac{22,17 \times 4,5^2}{8} = 56,11 \text{ kNm}$$

2φ14

$$M_{ed} = 78,56 \text{ kNm}$$

$$\Rightarrow A_s = 6,58 \text{ m}^2$$



viša φ8/15m

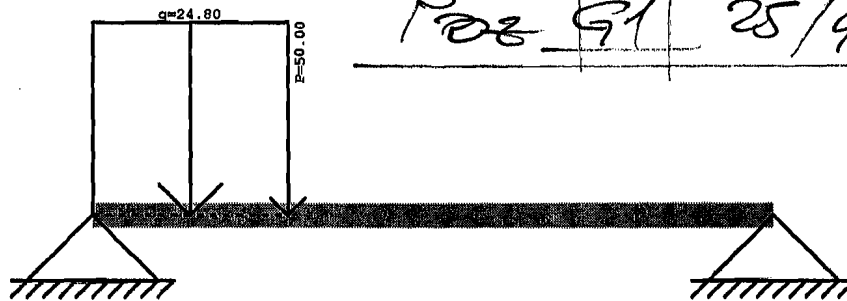
ALTERNATIVNO:

HEA 220

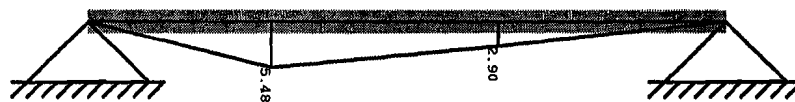
Po 91 - gređ 25/35

- od stepenica SB1 - $9,54 \times 2,6$
= $24,80 \text{ kN/m}'$

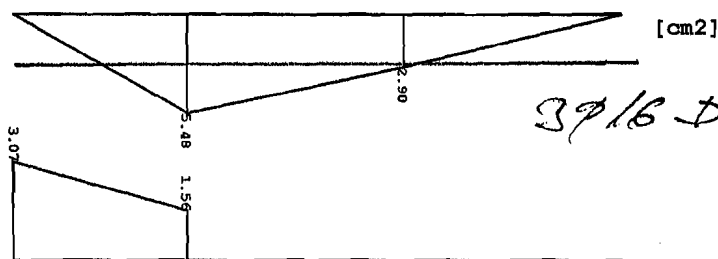
- od 92 = $22,17 \times 4,5 / 2 = 50 \text{ kN}$



Opt. 1: sve (g) - Ulazni podaci



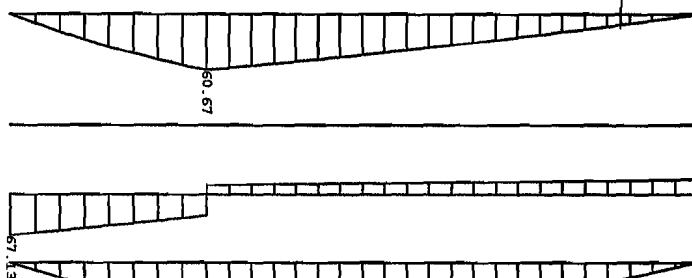
Potrebna armatura - Aa2/Aa1 (S500H, u 25)



Potrebna armatura - Greda: 1 - 3 (S500H, C 25)

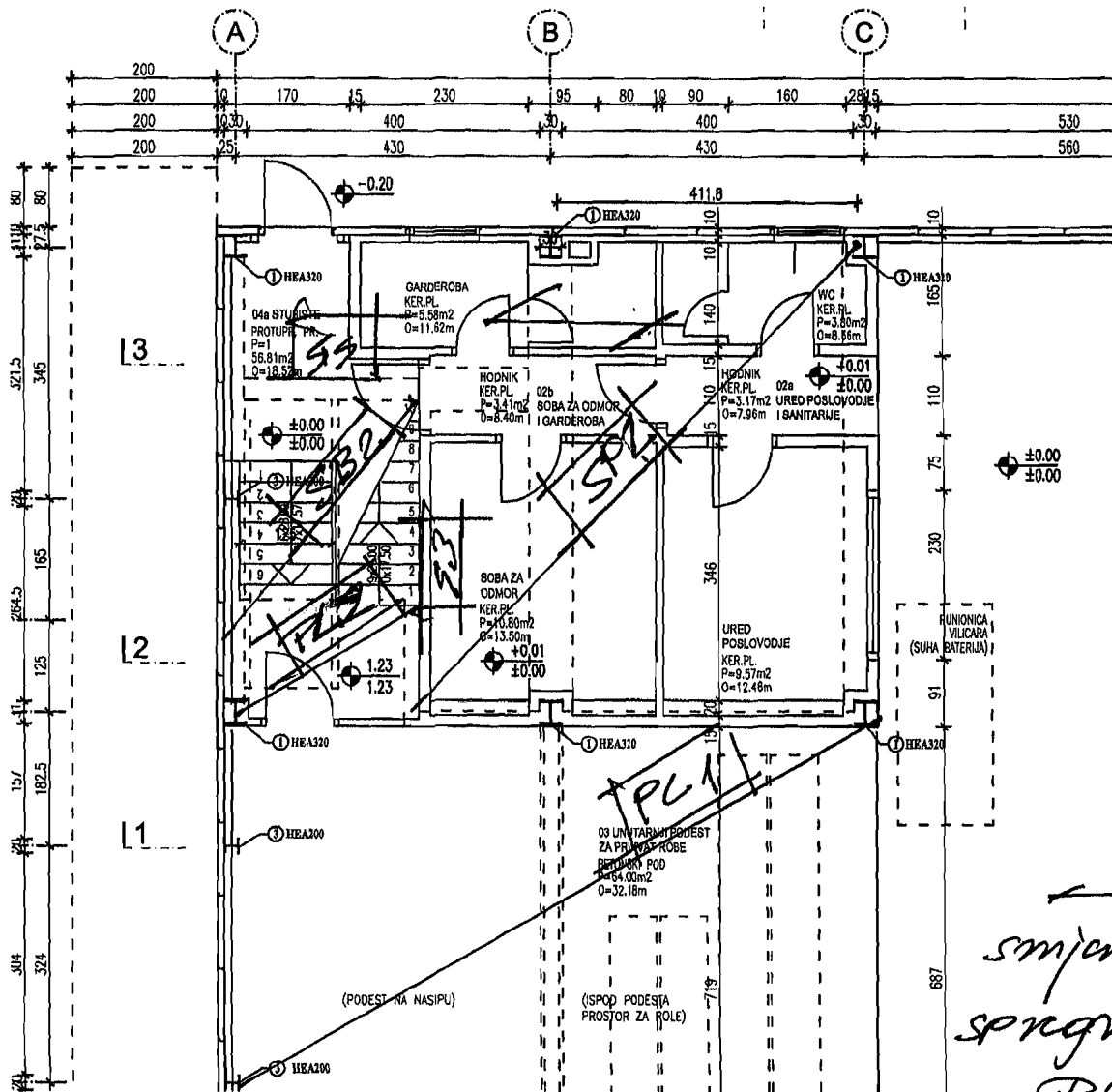


Opt. 1: sve (g) - Reakcije ležajeva (kN-kNm)



Opt. 1: sve (g) - Utjecaji u gredi: 1 - 3

Handwritten notes: $l_{max} = 9.5m$ and a signature.



STANJA PRIZEMLJA
step prizemlja

mg 1:100

POZ SP2

PRORAČUN SPREGNUTE PLOČE

1. UVOD

Proračun spregnute ploče biti će proveden za ploču širine 100 cm, te kao niz slobodno oslonjenih greda preko više oslonaca. Spregnuta ploča se izvodi čeličnim limom HiBond 55, $t=0,8$ mm, S250 (TRIMO d.d.) U gornju zonu ploče nad ležajevima se stavlja armaturna mreža Q-188 sa 2.5 cm zaštitnog sloja.

2. DIMENZIJE ELEMENTA

širina	$b =$	100 cm
visina spregnute ploče	$h =$	15 cm
efektivna visina spregnute ploče	$d_p =$	12,25 cm
raspon	$l =$	4,20 m
zaštitni sloj	$c =$	2,5 cm

3. KAKVOĆA MATERIJALA

beton

C25/30	$f_{ck} =$	25	N/mm ²
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armatura

MAR 500/560	$f_{yk} =$	50	kN/cm ²
	$E_s =$	210000	N/mm ²

čelični trapezni lim

HiBond 55, $t=0,8$ mm, S250 GD (EN 10147:2000)

$f_{yp} =$	25,00	kN/cm ²
------------	-------	--------------------

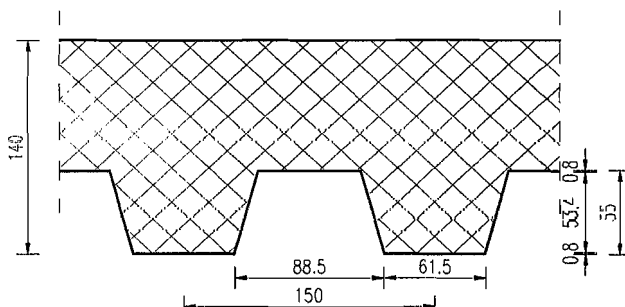
$W_{p,eff,max} =$	20,9	cm ³
-------------------	------	-----------------

$W_{p,eff,min} =$	16,9	cm ³
-------------------	------	-----------------

$W_{pl} =$	23,8	cm ³
------------	------	-----------------

$I_{p,eff} =$	50,7	cm ⁴
---------------	------	-----------------

$I_p =$	60,3	cm ⁴
---------	------	-----------------



karakterističan poprečni presjek

duljina vala	150	mm
$t_p =$	0,8	mm
$h_p =$	55	mm
$b_p =$	61,5	mm
$A_p =$	12,2	cm ²
$A_{pe,ff} =$	11,0	cm ²
$\phi =$	76,25°	

4. DJELOVANJA

vlastita težina lima	$q_{lima} =$	0,10	kN/m ²
vlastita težina betona	$q_c =$	3,06	kN/m ²
radno opterećenje	$q_{m1} =$	1,50	kN/m ²
(u fazi betoniranja)	$q_{m2} =$	0,75	kN/m ²
korisno opterećenje	$q =$	1,50	kN/m ²

spregnuti presjek preuzima dodatno stalno i korisno opterećenje

-spregnuta ploča se računa kao niz slobodno oslonjenih greda

vlastita težina	3,06	kN/m ²
dodatno stalno	2,00	kN/m ²
korisno opterećenje	1,50	kN/m ²

Računsko opterećenje

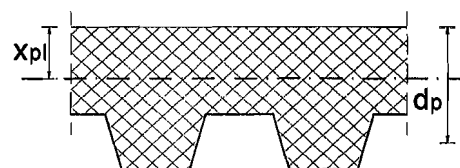
$$p_{sd} = (\gamma_g g + \gamma_q q) b = (1,35g + 1,5q) 1,00 = 9,08 \text{ kN/m}$$

6. Dimenzioniranje

6.1. otpornost spregnute ploče na savijanje za KGS

$$M_{pl,Rd} > M_{y,Sd}$$

$$M_{y,Sd} = p_{sd} l^2 / 8 = 20,03 \text{ kNm}$$



poprečni presjek sa položajem neutralne osi

izračun plastične neutralne osi

$$x_{pl} = \frac{A_p f_{yp} / \gamma_{ap}}{b \cdot 0,85 f_{ck} / \gamma_c} = 1,96 \text{ cm} < 9,50 \text{ cm}$$

neutralna os je u ploči

proračun momenta otpornosti

$$M_{pl,Rd} = A_p \frac{f_{ypk}}{\gamma_{ap}} (d_p - \frac{x_{pl}}{2}) = 31,25 \text{ kNm}$$

$$d_p = d - h_p / 2 = 122,5 \text{ mm}$$

Provjera

$$M_{Sd} = 20,0 \text{ kN/m} < 31,25 \text{ kN/m}$$

ZADOVOLJAVA

6.2. proračun na podužno smicanje

$$M_{Rd} = N_c z_1 + N_{as} z_2 + M_{pr} > M_{y,Sd}$$

Utjecaj od dodatne armature u rebrima

(nema dodatne armature)

$$A_s = 0,00 \text{ cm}^2$$

$$N_{as} = A_s \frac{f_{sk}}{\gamma_s} = 0,00 \text{ kN}$$

uvjet: $A_s \frac{f_{sk}}{\gamma_s} \leq 0,7 A_p \frac{f_{yp}}{\gamma_a}$

$$0,0 \text{ kN} < 194,1 \text{ kN}$$

ZADOVOLJAVA

$$N_c = N_{c1} + N_{c2} + N_{c3} \leq N_{cf} = N_p$$

Utjecaj od sprezanja limom

$$N_{c1} = 0$$

ne uzima se u obzir sprezanje limom zbog nedostatka eksperimentalnih podataka

Utjecaj trenja na osloncu

$$N_{c2} = \mu R \quad \mu = 0,50$$

R - reakcija na osloncu, za prostu gredu $R = V_{Sd}$

$$N_{c2} = 9,54 \text{ kN}$$

Utjecaj sidrenja na krajevima

sidrenje se vrši moždanicama tipa NELSON Φ 19

$$N_{c3} = V_{ld}$$

$$V_{ld} = P_{pb,Rd} = k_{\phi} d_{d0} t f_{yp} / \gamma_{ap} = 22,89 \text{ kN} \quad f_u = 45,00 \text{ kN/cm}^2$$

$$d = 19 \text{ mm}$$

$$k_{\phi} = 1 + a/d_{d0} \leq 6 \quad d_{d0} = 1,1 d = 2,09 \text{ cm} \quad d - \text{promjer moždanika}$$

$$a \geq 1,5 d_{d0} \quad a \geq 3,14 \text{ cm} \quad a - \text{udaljenost moždanika od ruba lima}$$

$$\text{odabrano } a = 10,50 \text{ cm} \quad (\text{za maksimalni } k_{\phi})$$

$$k_{\phi} = 6,0$$

razmak moždanika $l_m = 15 \text{ cm}$ (moždanik u svakom valu)

$$N_{c3} = V_{ld} / l_m = 152,61 \text{ kN}$$

$$N_c = 162,14 \text{ kN}$$

$$z_1 = h - 0,5 x_{pl} - e_p + (e_p - e) N_c / (A_p f_{yp} / \gamma_{ap}) \quad e_p - \text{udaljenost od neutralne osi do donjeg ruba lima}$$

$$z_1 = 7,07 \text{ cm} \quad e - \text{udaljenost od težišta lima do donjeg ruba lima}$$

$$z_2 = d_s - 0,5 x_{pl} \quad e_p = 13,86 \text{ cm}$$

$$z_2 = 14,43 \text{ cm} \quad e = 2,75 \text{ cm}$$

d_s - udaljenost armature od gornjeg ruba ploče $\approx h_{ploče}$

$$x_{pl} = \frac{N_c + N_{as}}{b \cdot 0,85 f_{ck} / \gamma_c} \quad x_{pl} = 1,14 \text{ cm}$$

Utjecaj lima na nosivost na savijanje

$$M_{pr} = 1,25 M_{pa} \left(1 - \frac{N_c}{A_p \frac{f_{yp}}{\gamma_{ap}}} \right) \leq M_{pa} \quad \text{proračunski moment pune plastičnosti}$$

$$\text{za efektivni presjek lima}$$

$$M_{pr} = 2,47 \text{ kNm}$$

Provjera

$$M_{Rd} = N_c z_1 + N_{as} z_2 + M_{pr}$$

$$M_{Rd} = 23,20 \text{ kNm} > 20,03 \text{ kNm}$$

ZADOVOLJAVA

6.3. proračun na vertikalno smicanje

$$V_{v,Rd} = b_0 d_p \tau_{Rd} k_v (1,2 + 40\rho) = 55,42 \text{ kN}$$

$$b_0 = 500 \text{ mm} \quad (\text{srednja širina svih rebara unutar 1m})$$

$$\tau_{Rd} = 0,25 f_{ctk} / \gamma_c = 0,30 \text{ N/mm}^2$$

$$k_v = 1,6 - d_p = 1,48 \text{ mm} \quad (d_p \text{ u metrima})$$

$$\rho = (A_p + A_s) / b_0 d_p = 0,0210 < 0,02$$

Provjera

$$V_{sd} = 19,1 \text{ kN} < 55,42 \text{ kN} = V_{v,Rd}$$

ZADOVOLJAVA

6.4. poprečna armatura - kontrola uzdužnog posmika

- otkazivanje nosivosti moždanika	$P_{Rd} \leq 0,8f_u \pi d^2 / (4\gamma_v)$	$P_{t,Rd} \leq$	81,61	kN
- drobljenje betona u okolici moždanika	$P_{Rd} \leq 0,29\alpha d^2 (f_{ck} E_{cm})^{1/2} / (\gamma_v)$	$P_{t,Rd} \leq$	72,53	kN
- mjerodavna nosivost (manja vrijednost pomnožena s k_r):		$P_{t,Rd} =$	61,66	kN
		$k_r =$	0,85	

Posmično djelovanje: $V_{Sd} = P_{Rd}/e = 61,66/0,15 = 411,07$ kN/m

$$V_{Sd}/2 = 205,53 \text{ kN/m (jedna strana čeličnog nosača)}$$

Proračunska nosivost na posmično djelovanje: $V_{Rd} = 2,5A_{cv}\eta\tau_{Rd} + A_s f_{sk} / \gamma_s + v_{pd}$

$A_{cv} =$	1225	cm ² /m (površina betonskog presjeka)
$\eta =$	1	(za beton normalne težine)
$\tau_{Rd} =$	0,30	N/mm ²
$A_s = Q188+Q335 =$	5,23	cm ² /m (površina ukupne armature)
$f_{sk} =$	50	kN/cm ²
$v_{pd} = A_p f_{vp} / \gamma_{ap} =$	277,3	kN/m'

$$V_{Rd} = 2,5 \times 1225 \times 0,03 + 5,23 \times 50 / 1,15 + 277,3 = 596,57 > 205,5 \text{ kN/m}' = V_{Sd}$$

Predviđena armatura na ležaju spregnute ploče je dostatna za osiguranje nosivosti na uzdužni posmik.

6.5. interakcija poprečnih sila na moždanik u dva okomita smjera

$F_l = 41$ kN računski poprečna sila od spregnutog nosača
 $F_t = P_{pb,Rd} = 22,89$ kN računski poprečna sila od spregnute ploče

$$\frac{F_l^2}{P_{l,Rd}^2} + \frac{F_t^2}{P_{t,Rd}^2} \leq 1$$

$P_{t,Rd} = 72,53$ kN računski nosivost moždanika
 $P_{l,Rd} = 61,65$ kN reducirana računski nosivost moždanika

$$0,542 < 1 \quad \text{ZADOVOLJAVA}$$

7. Granično stanje uporabivosti

7.1. ograničavanje širine pukotina

prema EC2 minimalna količina armature kada je spregnuta ploča prosta greda je 0,4% A_c za poduprte ploče, a 0,2% A_c za nepoduprte ploče

$A_s = 3,35$ cm² (Q-335 postavljena u gornju zonu nad ležajem)
armatura mora pokrivati minimalno 20% raspona

$$\rho = \frac{A_s}{b \cdot h_c} = 0,4 \% \geq 0,4 \%$$

ZADOVOLJAVA

odabrano Q-335 postavljena u gornju zonu nad ležajem

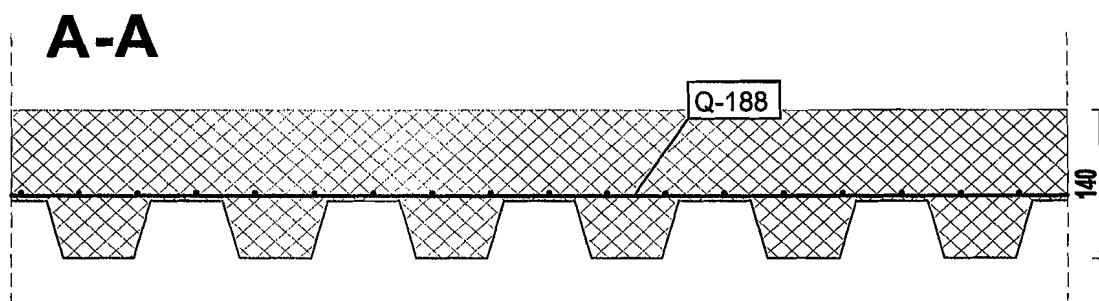
proračun dodatne armature

za prijenos eventualnih koncentriranih sila (pregradne stijene, lakša koncentrirana oprema, nejednoliko korisno opterećenje)

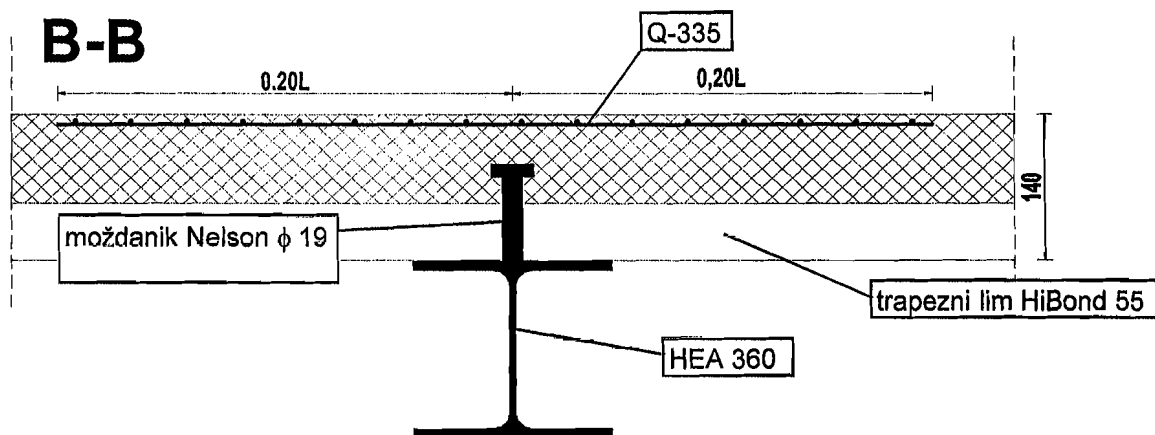
$$A_s = 0,002 b(h-h_{lma}) = 1,90 \text{ cm}^2 \quad (0,2\% A_c)$$

odabrano Q-188 postavljena na lim po cijeloj površini

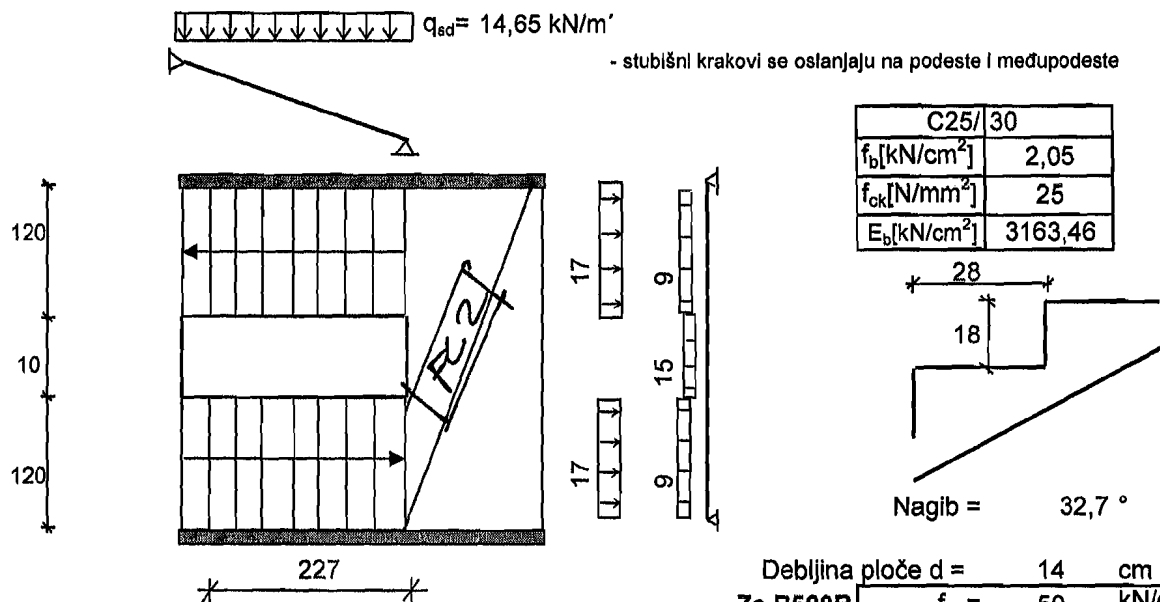
Presjek spregnute ploče u polju



Presjek spregnute ploče na ležaju



Poz. SB2 i PL2



Opterećenje:

Stubišnih krakova:

- ploča.....	4,16 kN/m ²
- gazište.....	2,16 kN/m ²
- habajući sloj	1,2 kN/m ²
$g =$	7,52 kN/m²
- korisno	3 kN/m ²
$q_{sd} =$	14,65 kN/m²

Podesta:

- ploča.....	3,50 kN/m ²
- habajući sloj	1,2 kN/m ²
$g =$	4,70 kN/m²
- korisno	1,5 kN/m ²
$q_{sd} =$	8,60 kN/m²

sudjelujuća širina b_s 62,5 cm

PRORAČUN - DIMENZIONIRANJE

Stubišni krak:

$$M_{sd} = 10,40 \text{ kNm/m'}$$

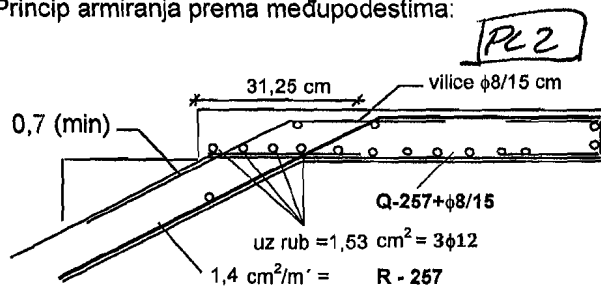
$$A_{s1} = 1,40 \text{ cm}^2/\text{m'}$$

Reakcija na podeste

$$R_{sd} = 16,63 \text{ kN/m'}$$

$$A_{s2gore} = 0 \text{ cm}^2/\text{m'}$$

Princip armiranja prema međupodestima:



Podesta:

Slobodni rub na širini 31,25 cm

$$M_{sd} = 27,67 \text{ kNm}$$

$$A_{s1\text{dodatno uz rub}} = 1,53 \text{ cm}^2/31,25 \text{ cm}$$

Ploča međupodesta:

$$M_{sd} = 24,14 \text{ kNm/m'}$$

$$A_{s1} = 5,11 \text{ cm}^2/\text{m'}$$

$$\text{Reakcija na oslonce: } R_A = R_B = 31,01 \text{ kN/m'}$$

$$A_{a\text{gore}} = 0 \text{ cm}^2 (\text{slob.ruba})$$

progib

podesta:

$f_{k(g+p)} [\text{cm}]$	0,69
$f_{dg} [\text{cm}]$	-0,22
$f_{uk} [\text{cm}]$	0,47
$f_{dop} [\text{cm}]$	1,25

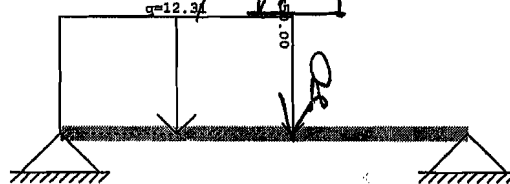
- 53 greda 20/35 = 92 NIVO.
2.98

- može i HEA 220 ALTERNATIVNO

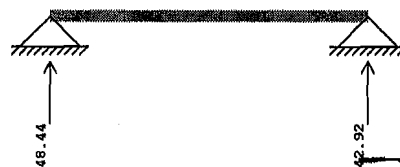
54 greda 25/45

- od stepenica 12.31 kN/m'

- od 53 = 50 kN...



Opt. 1: sve (g) - Ulazni potaci

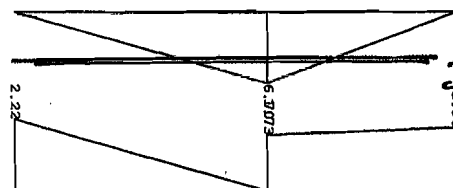


Opt. 1: sve (g) - Reakcije ležajeva (kN-kNm)

Armiranje
cp14 (3.2)

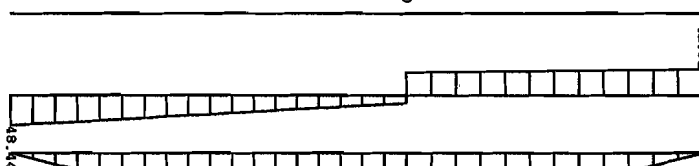
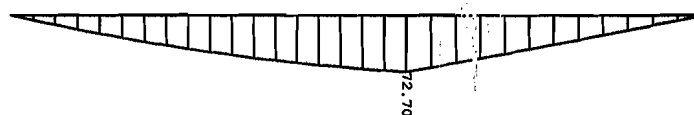


Potrebna armatura - Aa2/Aa1 (S500H, C 25)



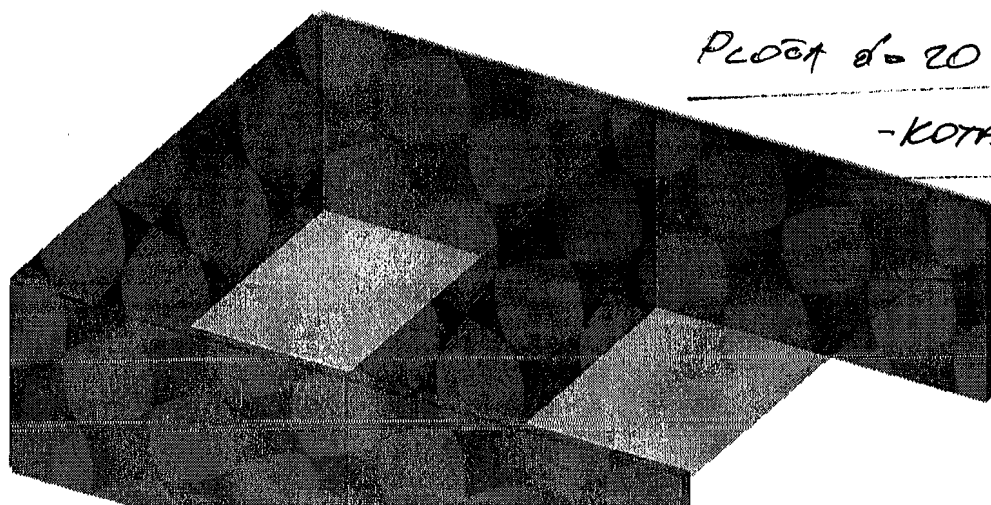
Potrebna armatura - Greda: 1 - 3 (S500H, C 25)

[cm²/m]



Opt. 1: sve (g) - Utjecaji u gredi: 1 - 3

Progibi fura 0.61m
Lozp.

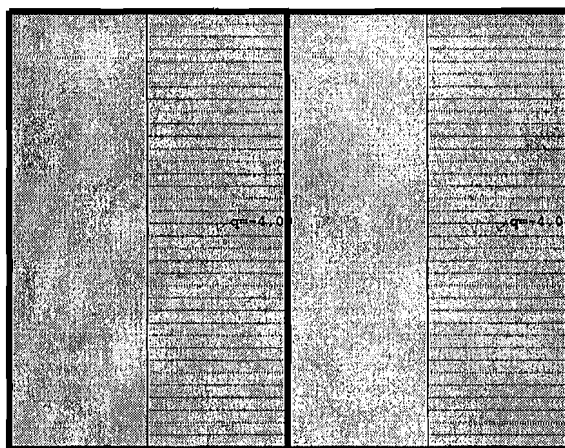


Ortogonalan 3D Prikaz

Ploča d= 20 cm

- KOTA +1.23

Poz PL 1



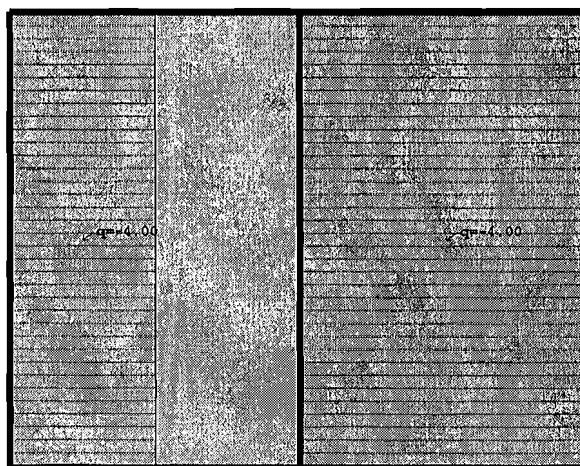
Opt. 2: korisno1 - Ulazni podaci

Korisnik

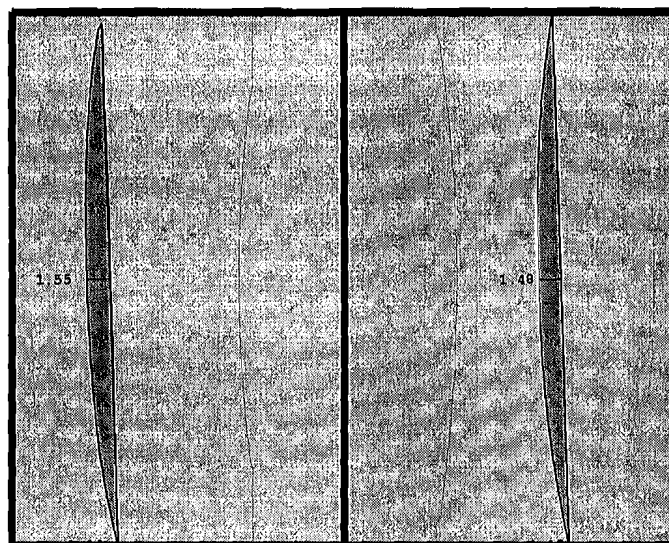
Optimizacija

- istovremeno robe

$q = 4.0 \text{ kN/m}^2$



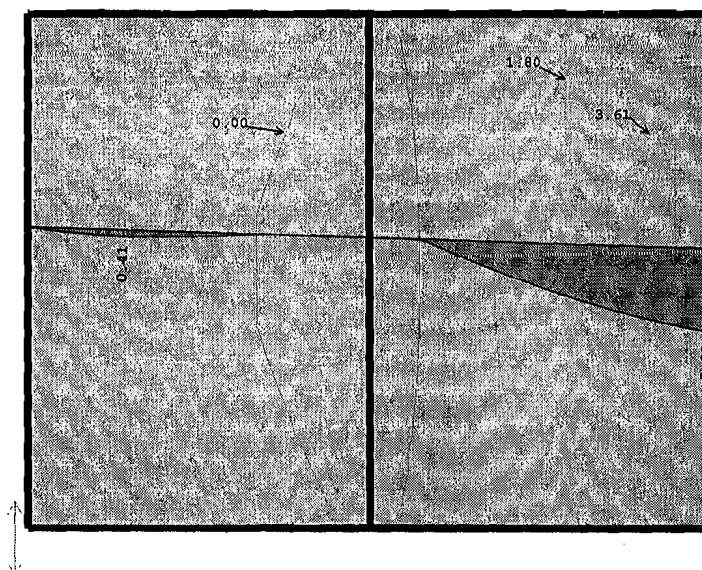
Opt. 3: korisno2 - Ulazni podaci



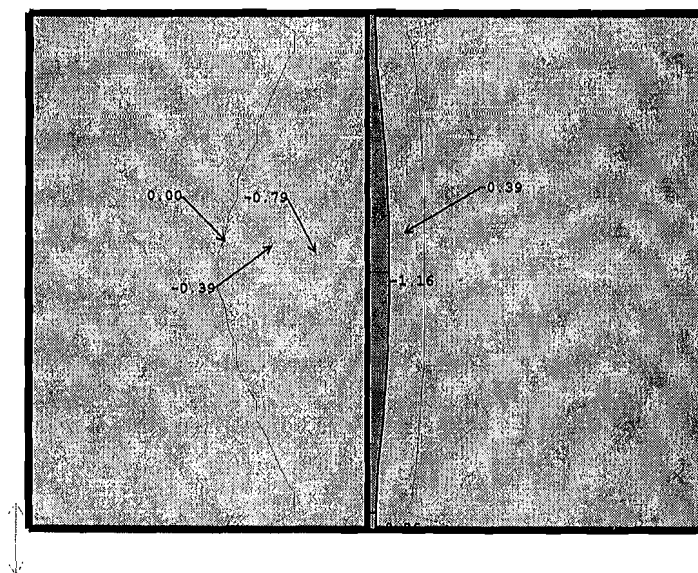
ARMATURA

Donje
zone

Anvelope: 5,6 - Aa - d. zona (S500H,C 25,a=3 cm)



Anvelope: 5,6 - Aa - d. zona (S500H,C 25,a=3 cm)

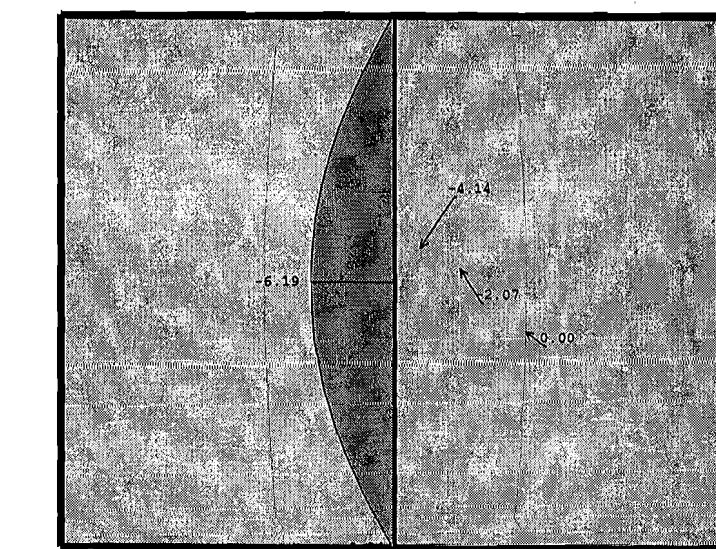


Anvelope: 5,6 - Aa - g. zona (S500N,C 25,a=3 cm)

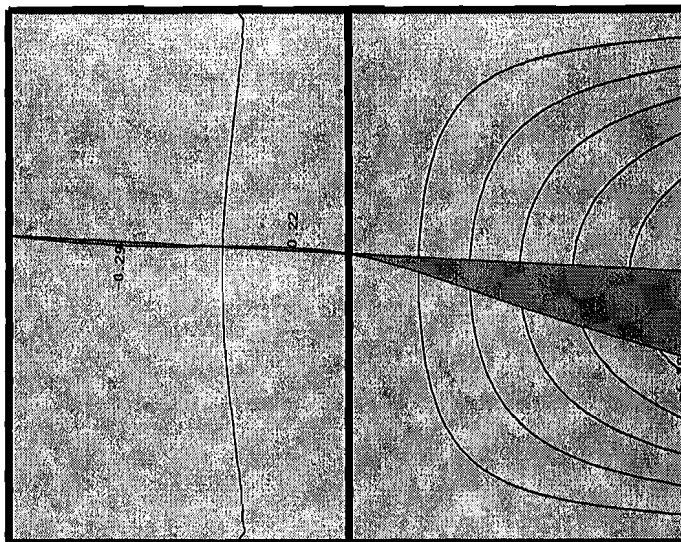
ARMATURA

GORJE

ROVE

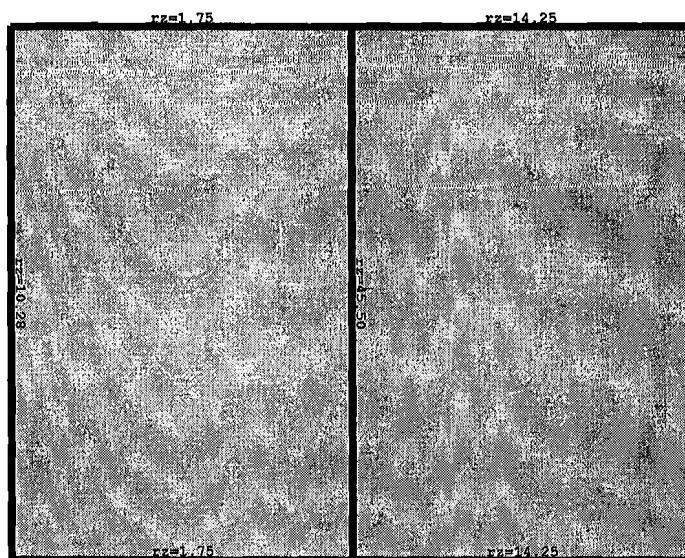


Anvelope: 5,6 - Aa - g. zona (S500N,C 25,a=3 cm)



Opt. 4: cisto - Utjecaji u ploči: Z_p (m) ($f_s=1000$)

Progibi
ploče
Ina - 17 mm
 $\leq f_{dep}$

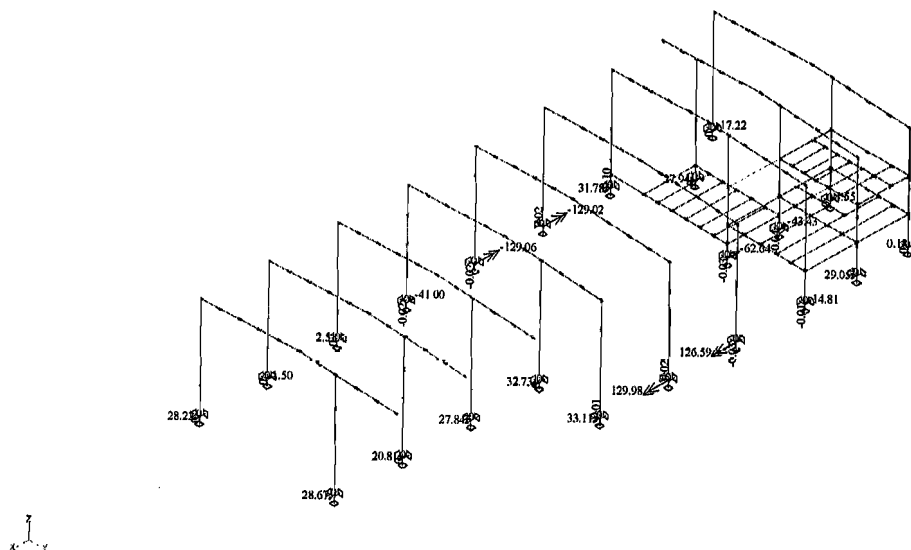


Opt. 4: cisto - Reakcije ležajeva (kN-kNm)

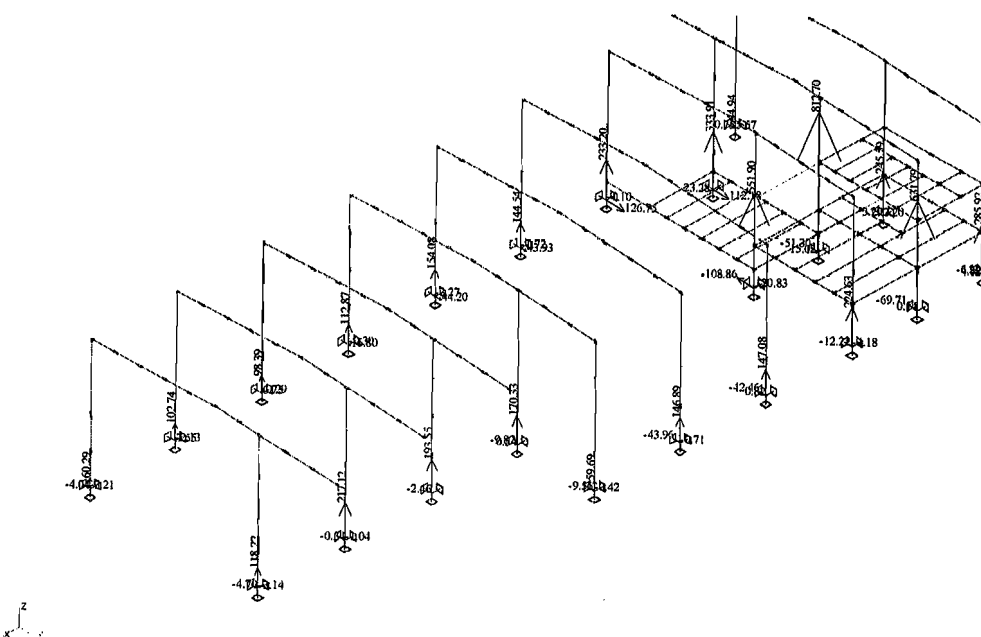
Reakcije
na
oslonacima

REAKCIJE (faktorirane)

$1.35 \times \text{stalno} + 1.5 \times \text{korisno} + 1.5 \times \text{snijeg}$



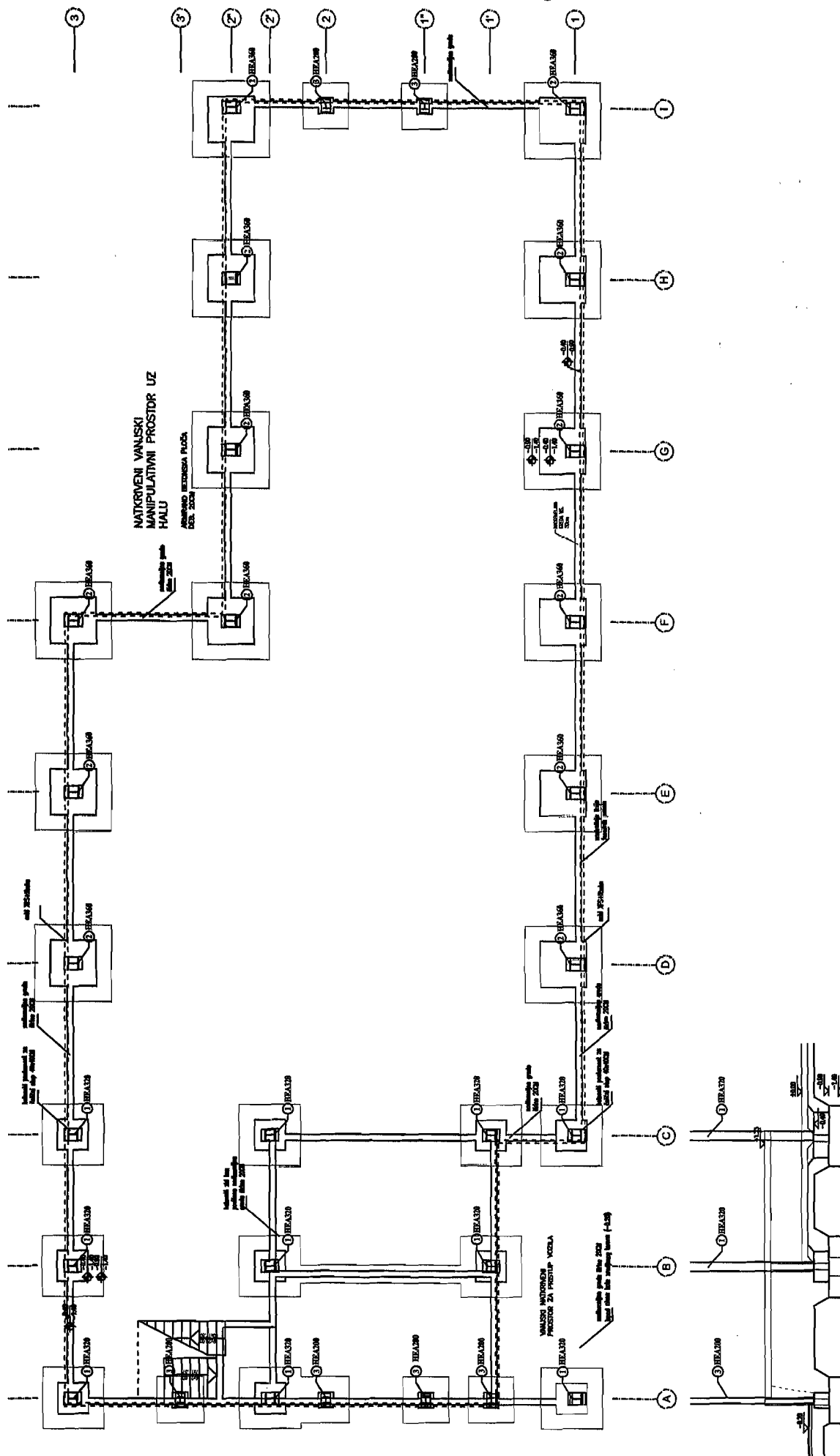
Reakcije M



Reakcije F

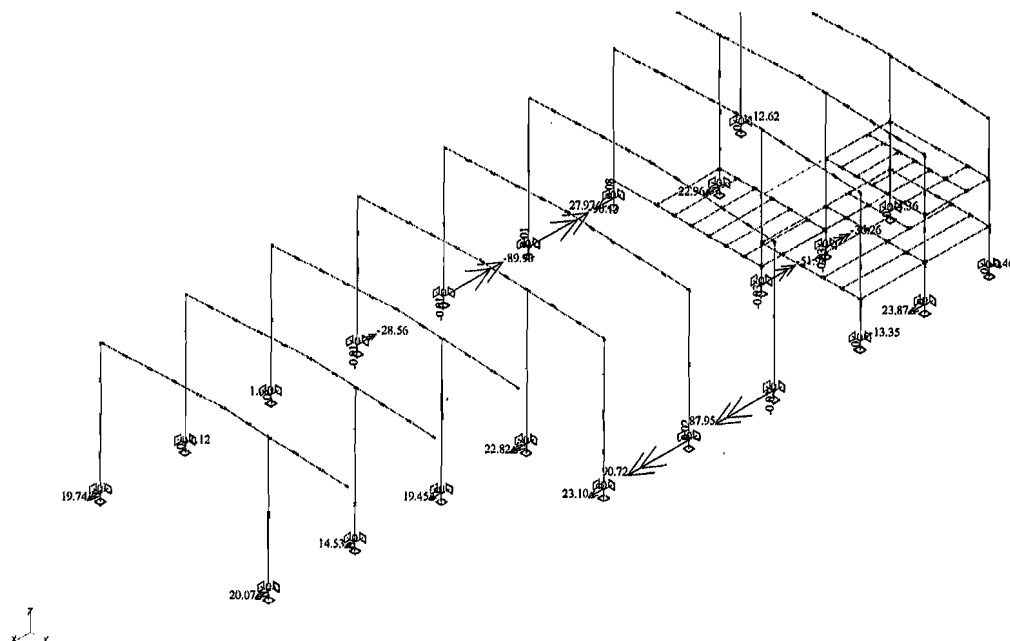
2.

Reakcije F

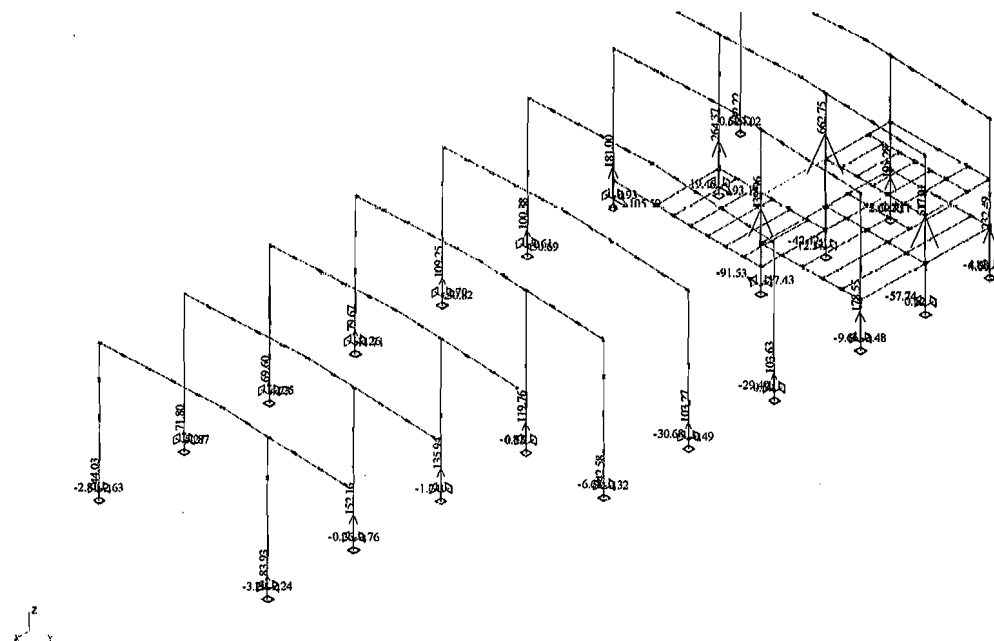


REAKCIJE (nefaktorirane)

Stalno + korisno + snijeg

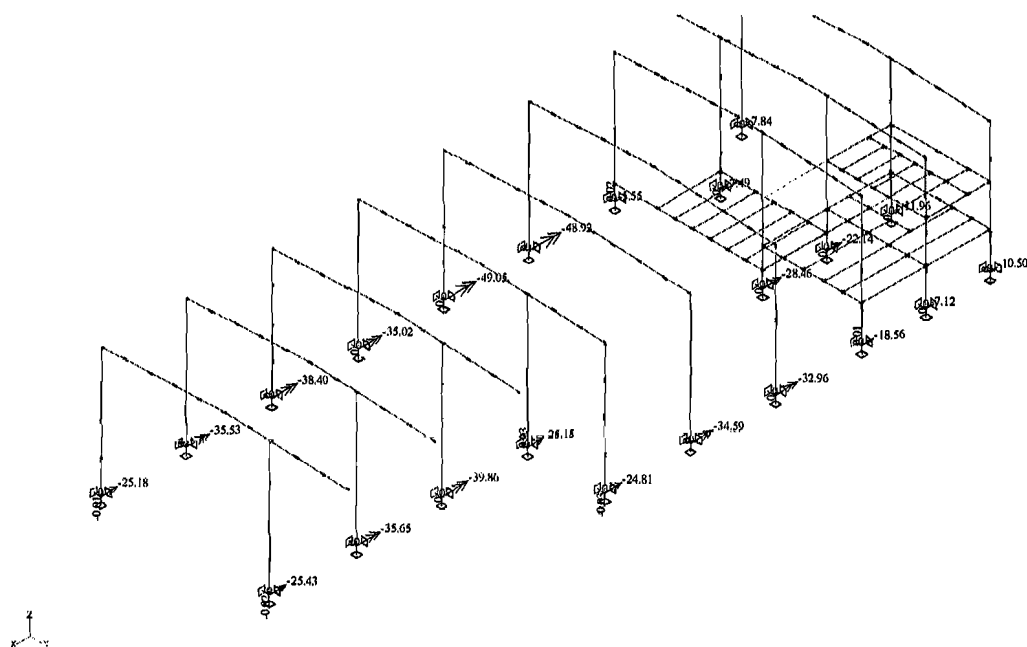


Reakcije M

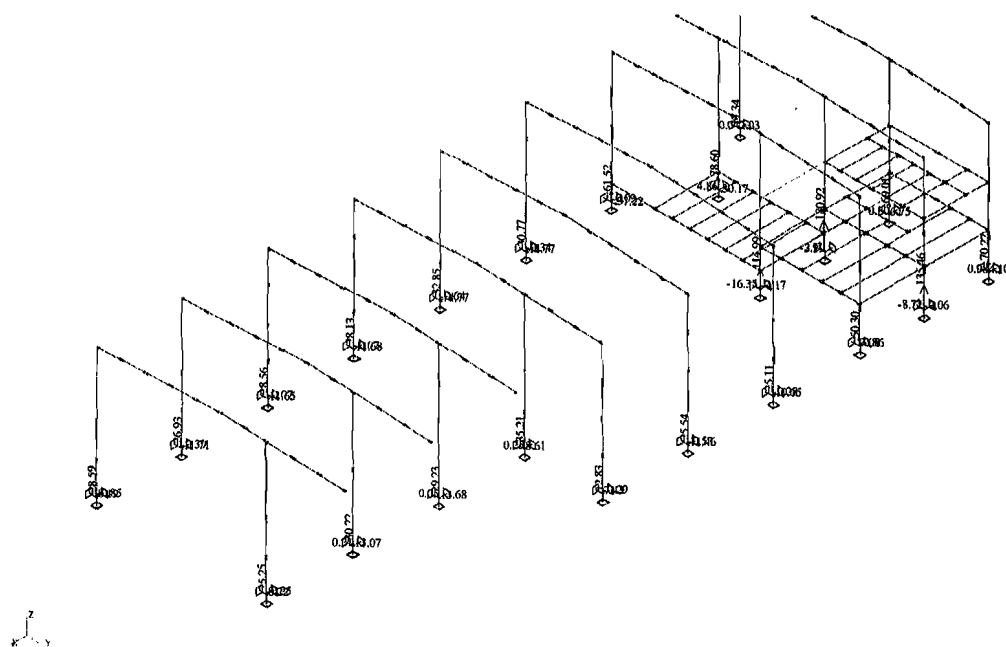


Reakcije F

Stalno + vjetar



Reakcije M



Reakcije F

3. GEOSTATIČKI PRORAČUNI

3.1 PODACI O GRAĐEVINI

Prema dobivenim podacima planirana hala je regalno skladište tlocrtnih dimenzija 42.5×17.0 m, visine 9.26 m do sljemena. Nosiva konstrukcija je čelična.

Temeljenje građevine je predviđeno kao plitko na temeljnim stopama. Geostatičkim proračunom provjeriti će se karakteristične temeljne stope L×B=2.5×2.5 m i 1.5×1.5 m, na dubini temeljenja D_r=1.0 m od površine urođenog terena.

Djelovanja na temelje dobivena su od statičara i prikazana su u slijedećoj tabeli:

	LxB		N (kN)	T (Kn)	M (kNm)	Muk (kNm)	Gt (kN)	Nuk (kN)
TEMELJ 1	2,5x2,5	nefaktorizirano	662	42	51	93	156	818
		faktorizirano	813	51	43	94	211	1024
TEMELJ 2	2,0x2,0	nefaktorizirano	103	31	87	118	100	203
		faktorizirano	147	44	130	174	135	282

Podaci o geometriji temeljenja te opterećenjima su prikazani u skici proračunskog modela tla i tabelama pripadnih proračuna.

3.2 OPISI PRORAČUNA

3.2.1 NOSIVOST TLA

- Proračunska nosivost tla ispod plitko temeljenog krutog pravokutnog temelja je određena za drenirane uvjete u tlu u skladu s Eurocode 7, prema formuli D.2. i proračunskom pristupu 3 (PP3). Kod PP3 parcijalni koeficijenti za djelovanja (γ) (GEO) iznose za stalno nepovoljno $\gamma_{G,sup} = 1.35$, i za promjenjivo nepovoljno $\gamma_Q = 1.5$.

$$R / A' = q_f = c' N_c b_c s_c i_c + q' N_q b_q s_q i_q + 0,5\gamma' B' N_\gamma b_\gamma s_\gamma i_\gamma \quad D.2.$$

3.2.1.1 PRETPOSTAVKE I OGRANIČENJA

- U proračunu nosivosti po kriteriju sloma tla su primjenjeni parcijalni koeficijenti : $\gamma_\phi = 1.25$ i $\gamma_c = 1.25$.

3.2.2 RAČUN SLIJEGANJA

Proračun slijeganja se zasniva na idealiziranom modelu tla kao elastičnom, homogenom i izotropnom poluprostoru. Račun slijeganja za koherentno tlo se provodi za linearno deformabilan medij u skladu s Hookeovim zakonom, a za nekoherentno tlo na osnovi otpora prodiranju šiljka pri izvođenju statičkog ili dinamičkog penetracionog pokusa. Raspodjela naprezanja u dubini opisanog poluprostora opterećenog na površini koncentriranom silom je određena Boussinesqovim izrazom (1885.g.). Integracijom navedenog izraza po pravokutnoj opterećenoj površini (Steinbrenner) je dobiven izraz za distribuciju naprezanja ($\sigma_{zi} = p_w * I_\sigma$) po vertikali u bilo kojoj točki ispod ili pokraj apsolutno savitljivog pravokutnog temelja opterećenog jednoliko raspodijeljenim opterećenjem. Aproksimacijom površina proizvoljnih oblika i opterećenja, nizom pravokutnih ploha s pripadnim jednolikim opterećenjima, moguće je primjenom navedenog izraza i superpozicije uticaja svih opterećenih pravokutnih ploha izračunati slijeganje bilo koje točke ispod ili pokraj opterećenih ploha (temelja).

GOSPODARSKA GRAĐEVINA SKLADIŠNA HALA ZA BAZENSKU TEHNIKU AQUACHEM – IVANIĆ GRAD

095/2016.

PG AQUACHEM
 NUMBER OF LAYER GROUPS --> 1
 NUMBER OF AREAS --> 1
 REFERENCE STRESS --> 1.00
 PERCENTAGE OF SIG-G --> 20.00
 POSITION OF AREAS
 I X1 Y1 X2 Y2 P
 1 .00 .00 2.00 2.00 32.00
 NUMBER OF LAYER GROUP --> 1
 NUMBER OF LAYERS --> 1
 NUMBER OF POINTS --> 1
 LAYER GAMMA D BETA MJ ITER
 1 9.00 10.00 1.00 4000.00 100
 POINT X Y SD-GEO
 1 .26 .26 19.00
 POINT 1
 X .26
 Y .26
 LAYER Z SIG-G SIG-P W1 W2
 1 2.05 37.45 7.30 .008 .008
 TOTAL SETTLEMENT FOR POINT 1 IS .0078
 POINT X Y W
 1 .26 .26 .008

PRORAČUN NOSIVOSTI																										
TEMELJ	ULAZNI PODACI												REZULTATI													
	PARAMETRI ČVRSTOĆE		PARCIJALNI KOEFICIJENTI		PODACI O TLU				DJELOVANJA				GEOMETRIJA TEMELJENJA				PRORAČUNSKA NOSIVOST - OTPORNOST		PRORAČUNSKO DJELOVANJE		EKSCENTRICITETI		DOPUSTENI EKSCENTRICITETI		SLIJEGANJE	
	ϕ'	c'	$\gamma\phi'$	$\gamma c'$	γ_v	γ	γ'	q'	V	H	M _I	M _b	D _r	L	B	α	R _d /A'	R _d	V _d	e _I	e _I dop	w				
	°	kPa	1	1	m	kN/m ³	kN/m ³	kPa	kN	kN	kNm	kNm	m	m	m	°	kPa	kN	kN	m	m	cm				
1	26	8	1,25	1,25	0	19	9,5	19	1024,0	51	94		1,0	2,5	2,5	0,0	350	2027	1024	0,09	0,83	4,40				
2	26	8	1,25	1,25	0	19	9,5	19	282,0	44	174		1,0	2,0	2,0	0,0	400	612	282	0,62	0,67	0,80				

3.4 KOMENTAR REZULTATA GEOSTATIČKIH PRORAČUNA

Proračunska nosivost tla ispod plitko temeljenog krutog pravokutnog temelja je određena za drenirane uvjete u tlu u skladu s Eurocode 7, prema formuli D.2. i proračunskom pristupu 3 (PP3) te je dobiveno:

TEMELJNA STOPA 1:

$V_d = 1024 \text{ kN} < R_d = 2027 \text{ kN}$, nosivost je zadovoljena.

Proračunsko slijeganje i modul reakcije tla iznose:

$w = 4.4 \text{ cm}$; $k_s = 3.0 \text{ MN/m}^3$.

TEMELJNA STOPA 2:

$V_d = 282 \text{ kN} < R_d = 612 \text{ kN}$, nosivost je zadovoljena.

Proračunsko slijeganje i modul reakcije tla iznose:

$w = 0.8 \text{ cm}$; $k_s = 6.4 \text{ MN/m}^3$.

Obzirom na sastav tla, slijeganja će se realizirati u dužem periodu po izgradnji objekta.

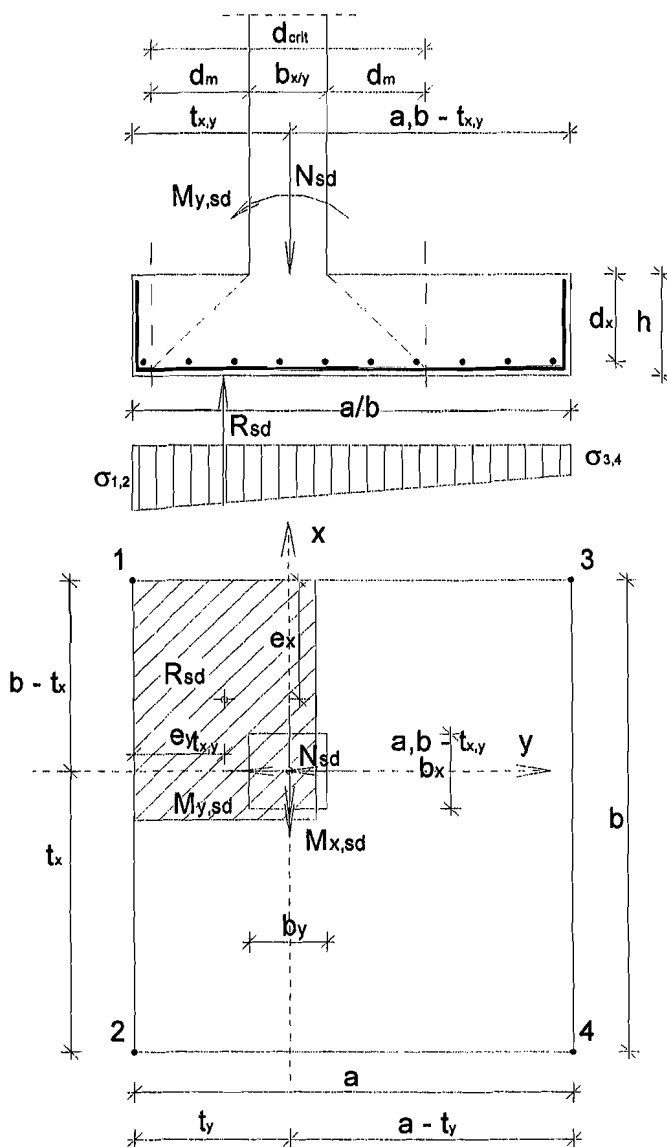
O prihvatljivosti izračunatih slijeganja treba odlučiti projektant nadtemeljne konstrukcije.

Posebno se napominje da provedeni proračuni vrijede samo za navedene parametre, pa je u slučaju promjene bilo kojeg parametra (geometrije temeljenja ili karakter, odnosno intenzitet opterećenja), potrebno provesti nove proračune.

152

1. TEMELJNE STOPE RUBNIH STUPOVA

1. STATIČKI SISTEM I POPREČNI PRESJEK



Dimenzije temelja:

$a = 200$ cm
 $b = 200$ cm
 $h = 80,00$ cm

$t_x = 100,00$ cm
 $t_y = 30,00$ cm

$c = 4,00$ cm zaštitni sloj

$d_s = 16$ mm

$d_x = h - c - d_s / 2 = 75,2$ cm statička visina

$d_y = d_x - d_s = 73,6$ cm

$d_m = (d_x + d_y) / 2 = 74,4$ cm

dubina zemlje

$d_z = 0$ cm

dubina temeljne stope

$d_t = d_z + h = 80$ cm

Dimenzije stupa:

širina $b_x = 40$ cm

visina $b_y = 60$ cm

2. KAKVOĆA MATERIJALA

Beton C25/30 (MB 30)

$f_{ck} = 25$ N/mm²

$f_{cd} = f_{ck} / 1.5 = 16,7$ N/mm²

$\gamma_b = 25$ kN/m³

Čelik S-500

$f_{yk} = 50$ kN/cm²

$f_{yd} = f_{yk} / 1.15 = 43,48$ kN/cm²

težina zemlje: $\gamma_z = 19,00$ kN/m³

$a/2 - t_y = 70,00$ cm

$b/2 - t_x = 0,00$ cm

3. OPTEREĆENJA

opterećenje	$V_{x,Sd}$	$V_{y,Sd}$	N_{Sd}	$M_{x,Sd}$	$M_{y,Sd}$
stalno i korisno (LCC4)	0,00	44,00	147,00	130,00	130,00

4. NAPREZANJA ISPOD TEMELJNE STOPE

4.1. VERTIKALNO OPTEREĆENJE

- uzdužna vertikalna sila za proračun temeljne stope $N_{sd,uk} = N_{sd} + N_{sd,ts} + N_{sd,z}$

vlastita težina temeljne stope $N_{sd,ts} = ab\gamma_b = 80,00 \text{ kN}$

težina zemlje iznad temeljne stope $N_{sd,z} = (ab - d_x d_y) d_z \gamma_z = 0,00 \text{ kN}$

$N_{sd,uk} = 80,00 \text{ kN}$

- horizontalne sile za proračun temeljne stope: $V_{x,sd} = 0,00 \text{ kN}$

$V_{y,sd} = 0,00 \text{ kN}$

- momenti za proračun temeljne stope:

$M_{x,sd,uk} = M_{x,sd} + N_{sd} (a/2 - t_y) + V_{y,sd} d_t$ $M_{x,sd,uk} = 56,00 \text{ kNm}$

$M_{y,sd,uk} = M_{y,sd} + N_{sd} (b/2 - t_x) + V_{x,sd} d_t$ $M_{y,sd,uk} = 0,00 \text{ kNm}$

ekscentricitet vertikalne sile

$e_x = M_{y,sd,uk} / N_{sd,uk} = 0,00 \text{ cm}$

$e_y = M_{x,sd,uk} / N_{sd,uk} = 70,00 \text{ cm}$

$e_x / b + e_y / a = 0,35 >$

$1/6 = 0,17 \text{ m}$ sila nije u jezgri poprečnog presjeka

naprezanja ispod temeljne stope

$\sigma_1 = N_{sd,uk} / A + M_{x,sd,uk} / W_x + M_{y,sd,uk} / W_y = 62,00 \text{ kN/m}^2$

$A_{\text{temelja}} = a \cdot b = 4 \text{ m}^2$

$\sigma_2 = N_{sd,uk} / A + M_{x,sd,uk} / W_x - M_{y,sd,uk} / W_y = 62,00 \text{ kN/m}^2$

$W_x = a^2 b / 6 = 1,33 \text{ m}^3$

$\sigma_3 = N_{sd,uk} / A - M_{x,sd,uk} / W_x + M_{y,sd,uk} / W_y = -22,00 \text{ kN/m}^2$

$W_y = ab^2 / 6 = 1,33 \text{ m}^3$

$\sigma_4 = N_{sd,uk} / A - M_{x,sd,uk} / W_x - M_{y,sd,uk} / W_y = -22,00 \text{ kN/m}^2$

efektivna širina temelja $a' = a (1 - 2e_y/a) = 60 \text{ cm}$

$b' = b (1 - 2e_x/b) = 200 \text{ cm}$

efektivna površina $A'_{\text{temelja}} = a' \cdot b' = 1,20 \text{ m}^2$

$\sigma_0 = N_{sd} / A' = 66,67 \text{ kN/m}^2$

dopušteno naprezanje za osnovno opterećenje

$\sigma_{doz} = 160,00 \text{ kN/m}^2$

☒ ZADOVOLJAVA

smjer Y $b_y/a = 0,30 \approx 0,30$

traka 1

$$\Delta M_{Sd,x,1} = \alpha_1 M_{Sd,x} = 0,09 \times 101,75 = 9,16 \text{ kNm} \quad \text{očitano:} \quad \zeta = 0,987 \quad \varepsilon_{c2} = -0,75 \text{ ‰}$$

$$\mu_{Sd} = M_{Sd} / (b d^2 f_{cd}) = 0,01 \quad \xi = 0,036 \quad \varepsilon_{s1} = 20,00 \text{ ‰}$$

$$A_{s1} = M_{Sd} / (\zeta d f_{yd}) = 0,29 \text{ cm}^2$$

odabrano: 4 $\Phi 10$ $A_{s1} = 3,14 \text{ cm}^2$

traka 2

$$\Delta M_{Sd,x,2} = \alpha_2 M_{Sd,x} = 0,11 \times 101,75 = 11,19 \text{ kNm} \quad \text{očitano:} \quad \zeta = 0,987 \quad \varepsilon_{c2} = -0,75 \text{ ‰}$$

$$\mu_{Sd} = M_{Sd} / (b d^2 f_{cd}) = 0,01 \quad \xi = 0,036 \quad \varepsilon_{s1} = 20,00 \text{ ‰}$$

$$A_{s1} = M_{Sd} / (\zeta d f_{yd}) = 0,35 \text{ cm}^2$$

odabrano: 4 $\Phi 10$ $A_{s1} = 3,14 \text{ cm}^2$

traka 3

$$\Delta M_{Sd,x,3} = \alpha_3 M_{Sd,x} = 0,14 \times 101,75 = 14,25 \text{ kNm} \quad \text{očitano:} \quad \zeta = 0,987 \quad \varepsilon_{c2} = -0,75 \text{ ‰}$$

$$\mu_{Sd} = M_{Sd} / (b d^2 f_{cd}) = 0,01 \quad \xi = 0,036 \quad \varepsilon_{s1} = 20,00 \text{ ‰}$$

$$A_{s1} = M_{Sd} / (\zeta d f_{yd}) = 0,45 \text{ cm}^2$$

odabrano: 4 $\Phi 10$ $A_{s1} = 3,14 \text{ cm}^2$

traka 4

$$\Delta M_{Sd,x,4} = \alpha_4 M_{Sd,x} = 0,16 \times 101,75 = 16,28 \text{ kNm} \quad \text{očitano:} \quad \zeta = 0,987 \quad \varepsilon_{c2} = -0,75 \text{ ‰}$$

$$\mu_{Sd} = M_{Sd} / (b d^2 f_{cd}) = 0,01 \quad \xi = 0,036 \quad \varepsilon_{s1} = 20,00 \text{ ‰}$$

$$A_{s1} = M_{Sd} / (\zeta d f_{yd}) = 0,51 \text{ cm}^2$$

odabrano: 4 $\Phi 10$ $A_{s1} = 3,14 \text{ cm}^2$

smjer X $b_x/b = 0,20 \approx 0,20$

traka 1

$$\Delta M_{Sd,y,1} = \alpha_1 M_{Sd,y} = 0,08 \times 101,75 = 8,14 \text{ kNm} \quad \text{očitano:} \quad \zeta = 0,987 \quad \varepsilon_{c2} = -0,75 \text{ ‰}$$

$$\mu_{Sd} = M_{Sd} / (b d^2 f_{cd}) = 0,01 \quad \xi = 0,036 \quad \varepsilon_{s1} = 20,00 \text{ ‰}$$

$$A_{s1} = M_{Sd} / (\zeta d f_{yd}) = 0,25 \text{ cm}^2$$

odabrano: 4 $\Phi 10$ $A_{s1} = 3,14 \text{ cm}^2$

traka 2

$$\Delta M_{Sd,y,2} = \alpha_2 M_{Sd,y} = 0,1 \times 101,75 = 11,19 \text{ kNm} \quad \text{očitano:} \quad \zeta = 0,987 \quad \varepsilon_{c2} = -0,75 \text{ ‰}$$

$$\mu_{Sd} = M_{Sd} / (b d^2 f_{cd}) = 0,01 \quad \xi = 0,036 \quad \varepsilon_{s1} = 20,00 \text{ ‰}$$

$$A_{s1} = M_{Sd} / (\zeta d f_{yd}) = 0,35 \text{ cm}^2$$

odabrano: 4 $\Phi 10$ $A_{s1} = 3,14 \text{ cm}^2$

traka 3

$$\Delta M_{Sd,y,3} = \alpha_3 M_{Sd,y} = 0,14 \times 101,75 = 14,25 \text{ kNm} \quad \text{očitano:} \quad \zeta = 0,987 \quad \varepsilon_{c2} = -0,75 \text{ ‰}$$

$$\mu_{Sd} = M_{Sd} / (b d^2 f_{cd}) = 0,01 \quad \xi = 0,036 \quad \varepsilon_{s1} = 20,00 \text{ ‰}$$

$$A_{s1} = M_{Sd} / (\zeta d f_{yd}) = 0,45 \text{ cm}^2$$

odabrano: 4 $\Phi 10$ $A_{s1} = 3,14 \text{ cm}^2$

traka 4

$$\Delta M_{Sd,y,4} = \alpha_4 M_{Sd,y} = 0,18 \times 101,75 = 16,28 \text{ kNm} \quad \text{očitano:} \quad \zeta = 0,987 \quad \varepsilon_{c2} = -0,75 \text{ ‰}$$

$$\mu_{Sd} = M_{Sd} / (b d^2 f_{cd}) = 0,01 \quad \xi = 0,036 \quad \varepsilon_{s1} = 20,00 \text{ ‰}$$

$$A_{s1} = M_{Sd} / (\zeta d f_{yd}) = 0,51 \text{ cm}^2$$

odabrano: 4 $\Phi 10$ $A_{s1} = 3,14 \text{ cm}^2$

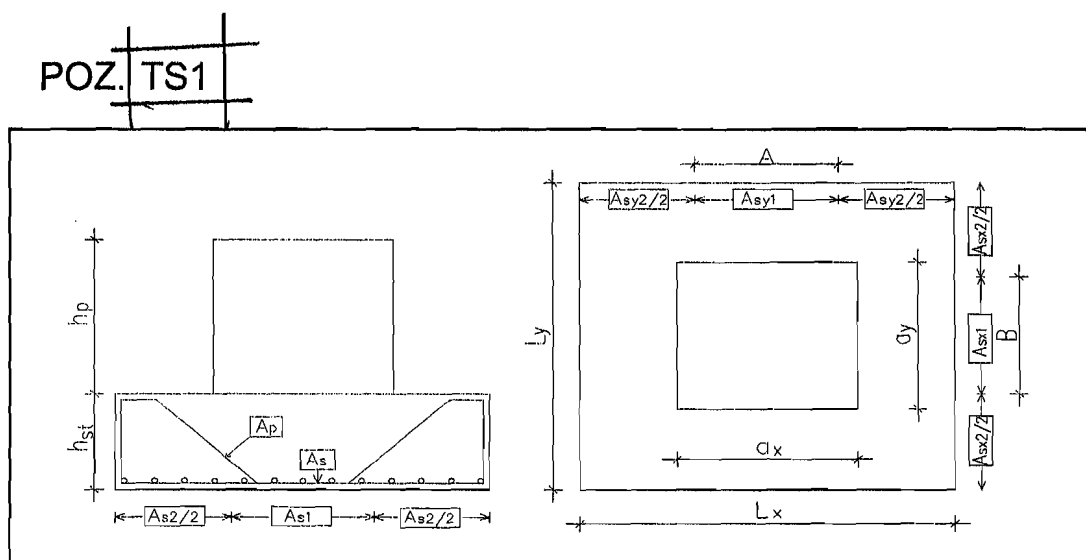
$$A_{s,min} = 0,0015 b d = 24,00 \text{ cm}^2$$

$$A_{s,max} = 0,04 A_c = 640 \text{ cm}^2$$

$$\Sigma A_{s1,X} = 25,12 \text{ cm}^2$$

$$\Sigma A_{s1,Y} = 25,12 \text{ cm}^2$$

☒ ZADOVOLJAVA



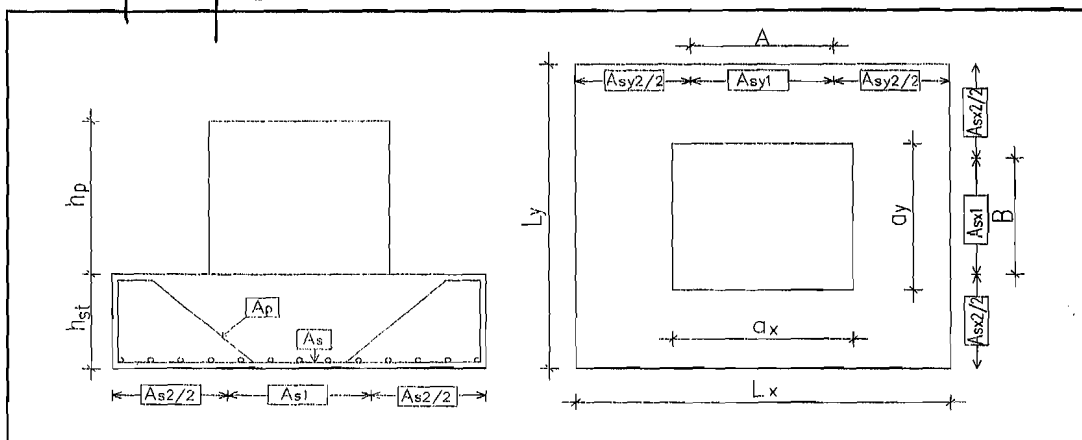
Ulazni podaci:

Geometrijske karakteristike		Opterećenje		Materijal		
L_x [cm]	150,0	N [kN]	150,00	Beton	C 25/30	
L_y [cm]	150,0	M_x [kNm]	30,00	Čelik	σ_v [kN/cm ²]	50,0
a_x [cm]	80,0	M_y [kNm]	5,00			
a_y [cm]	80,0	H_x [kN]	10,00			
h_p [cm]	40,0	H_y [kN]	0,00			
h_{st} [cm]	40,0					

Rezultati proračuna:

Sile na dnu temelja		Naponi na tlu		Armatura stope	
N_t [kN]	191,78	σ_{tla1} [kN/m ²]	161,68	A_{sx} [cm ²]	0,00
M_{xt} [kNm]	30,00	σ_{tla1} [kN/m ²]	8,79	A_{sx1} [cm ²]	0,00
M_{yt} [kNm]	13,00			A_{sx2} [cm ²]	0,00
				A_{sy} [cm ²]	0,00
				A_{sy1} [cm ²]	0,00
				A_{sy2} [cm ²]	0,00
				A [m]	1,15
				B [m]	1,15

POZ. TS3



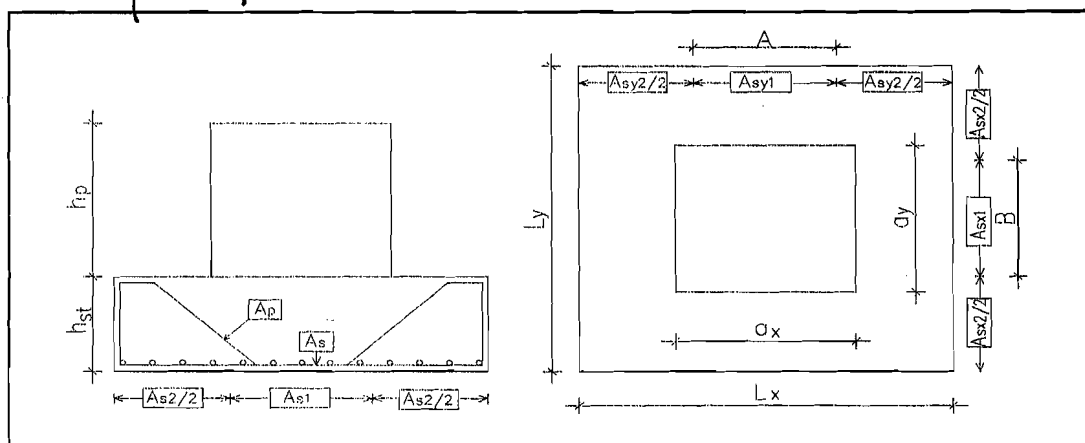
Ulazni podaci:

Geometrijske karakteristike		Opterećenje		Materijal	
L_x [cm]	200,0	N [kN]	500,00	Beton	C 25/30
L_y [cm]	200,0	M_x [kNm]	23,00	Čelik	σ_v [kN/cm ²] 50,0
a_x [cm]	100,0	M_y [kNm]	5,00		
a_y [cm]	100,0	H_x [kN]	10,00		
h_p [cm]	40,0	H_y [kN]	0,00		
h_{st} [cm]	40,0				

Rezultati proračuna:

Sile na dnu temelja		Naponi na tlu		Armatura stope	
N_t [kN]	574,00	σ_{tlat} [kN/m ²]	170,50	A_{sx} [cm ²]	4,63
M_{xt} [kNm]	23,00	σ_{tlat} [kN/m ²]	116,50	A_{sx1} [cm ²]	3,73
M_{yt} [kNm]	13,00			A_{sx2} [cm ²]	0,90
				A_{sy} [cm ²]	4,82
				A_{sy1} [cm ²]	3,88
				A_{sy2} [cm ²]	0,94
				A [m]	1,35
				B [m]	1,35

POZ. TS4



Ulazni podaci:

Geometrijske karakteristike		Opterećenje		Materijal		
L_x [cm]	250,0	N [kN]	662,00	Beton	C 25/30	
L_y [cm]	250,0	M_x [kNm]	51,00	Čelik	σ_v [kN/cm ²]	50,0
a_x [cm]	130,0	M_y [kNm]	10,00			
a_y [cm]	130,0	H_x [kN]	42,00			
h_p [cm]	40,0	H_y [kN]	0,00			
h_{st} [cm]	40,0					

Rezultati proračuna:

Sile na dnu temelja		Naponi na tlu		Armatura stope	
N_i [kN]	777,88	σ_{tla1} [kN/m ²]	160,79	A_{sx} [cm ²]	7,61
M_{xi} [kNm]	51,00	σ_{tla1} [kN/m ²]	88,13	A_{sx1} [cm ²]	6,05
M_{yi} [kNm]	43,60			A_{sx2} [cm ²]	1,56
				A_{sy} [cm ²]	7,74
				A_{sy1} [cm ²]	6,16
				A_{sy2} [cm ²]	1,59
				A [m]	1,65
				B [m]	1,65

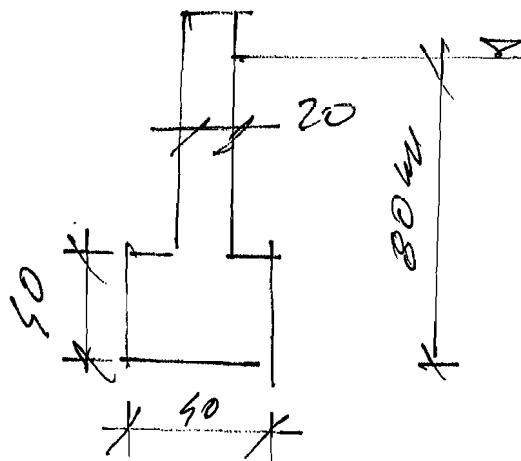
Nadtemeljni zid - oslonac fasade



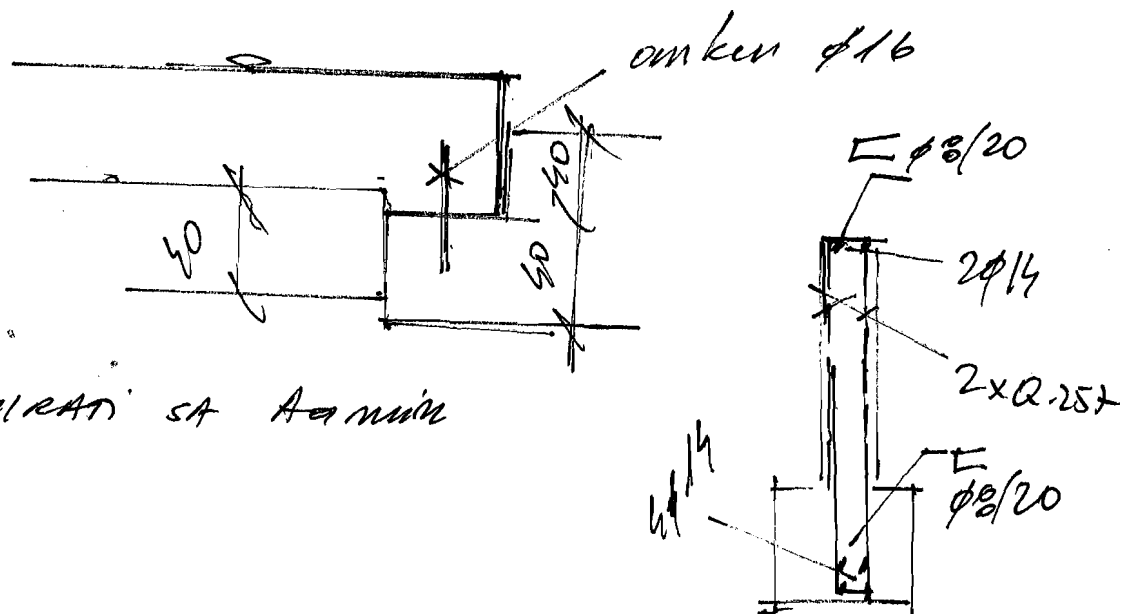
(Temeljna traka, nadozid i greda između stopa)

Opterećenje:

- vl.t. fasade - termoizolacijski paneli cca 22 kg/m^2
⇒ za visinu 9,5 m = $0,22 \times 9,5 = 2,09 \text{ kN/m}^1$
⇒ raspon između temeljnih samaca: max 4,2 m



OSLANJANJE NA STOPU



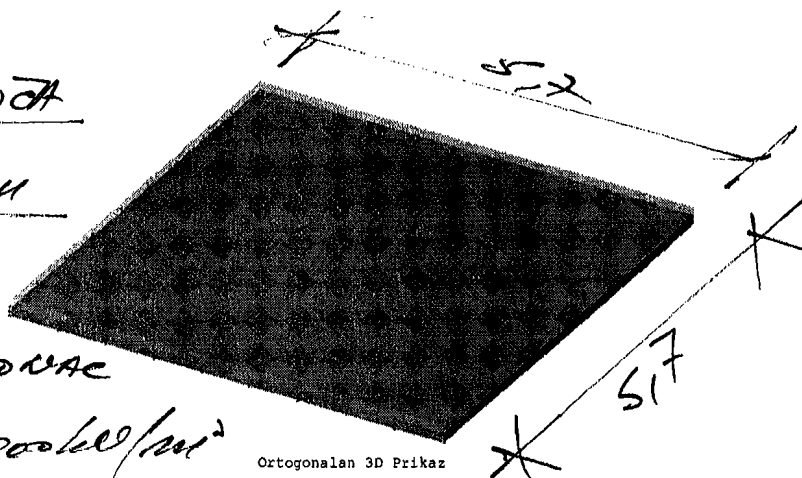
POONA PLOŠTA

$d = 20 \text{ m}$

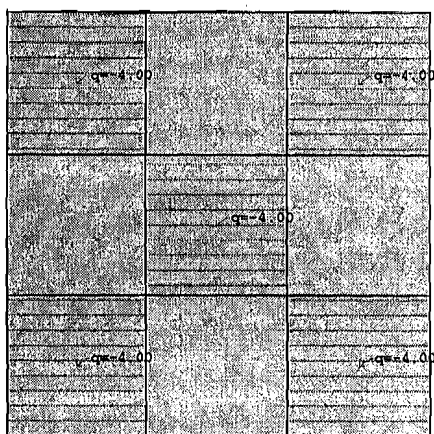
ELASTIČNI OSNOVAC

$k_s = 6.000 \text{ kN/m}^2$

Ortogonalan 3D Prikaz

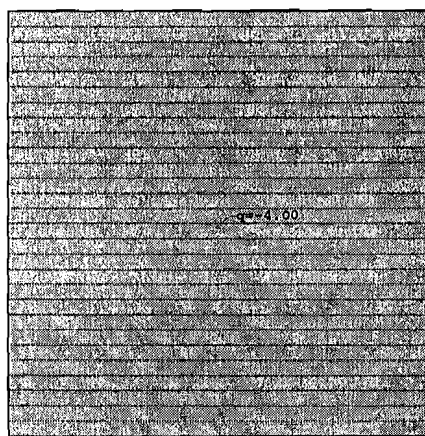


TIPU OPTEREĆENJA



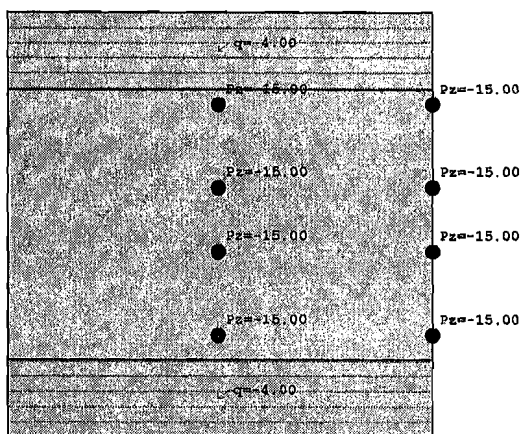
Opt. 2: korisno1 - Ulazni podaci

KORISNO 1



Opt. 3: korisno 2 - Ulazni podaci

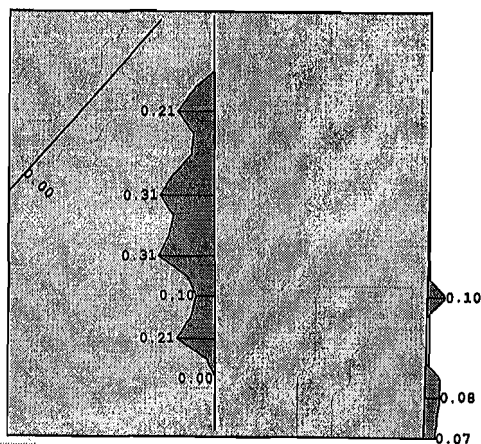
KORISNO 2



Opt. 4: korisno 3 - Ulazni podaci

KORISNO 3

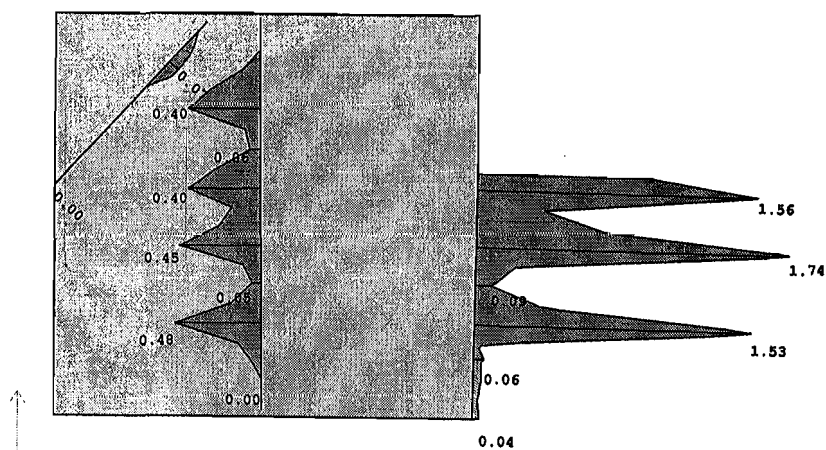
- regavno skladište
- osnovac reakcija
1500 kN



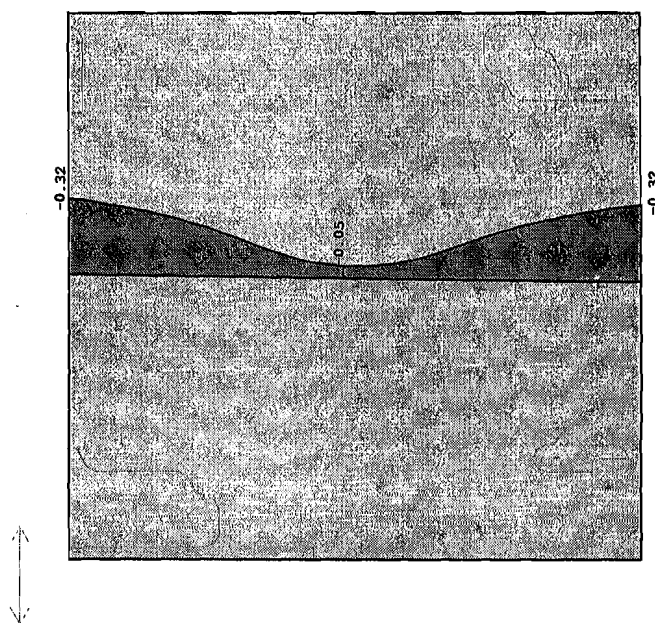
ARNATUR

Danijel Zovko

Anvelope: 6,7,8 - Aa - d. zona (S500H,C 25,a=3 cm)



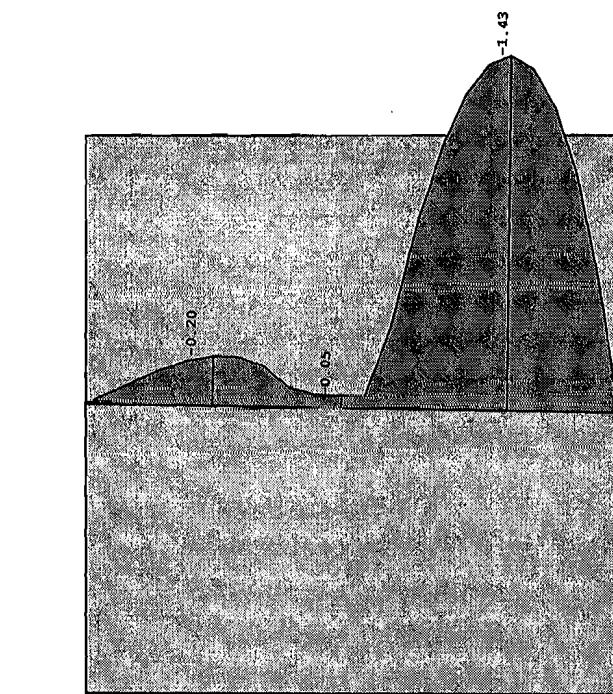
Anvelope: 6,7,8 - Aa - d. zona (S500H,C 25,a=3 cm)



ARMATURA

SONJE BOND

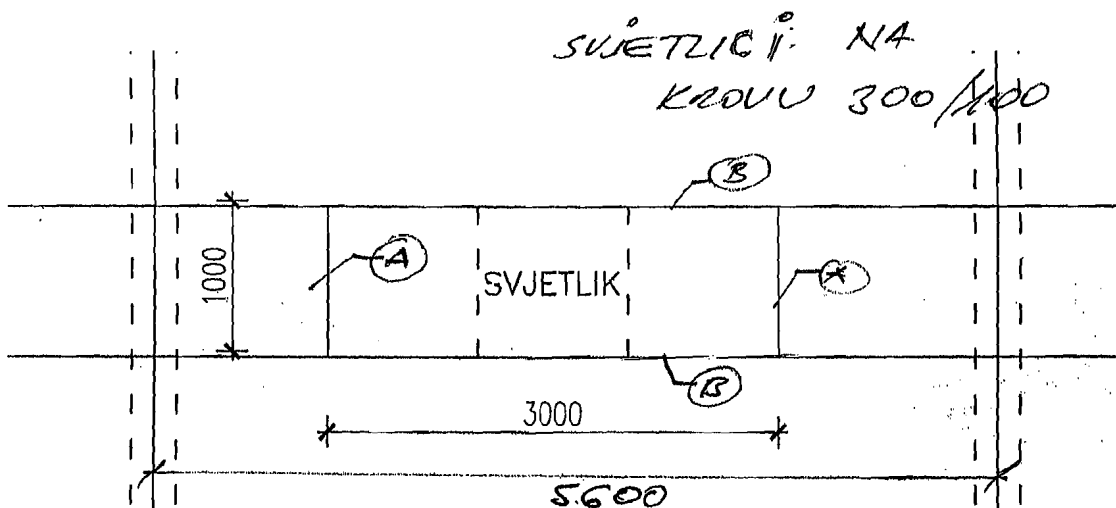
Anvelope: 6,7,8 - Aa - g. zona (S500N,C 25,a=3 cm)



Anvelope: 6,7,8 - Aa - g. zona (S500N,C 25,a=3 cm)

- ARMIRATI OŠTRAVO SA Q-252
- ZARJEZATI 2/3 PRESJeka PLOŠE ZA FORMIRANJE POJA MAX 6/6 ~ 36 m²
- ISPUNITI FUGE TRAJNOERASTIČNOM KITOM

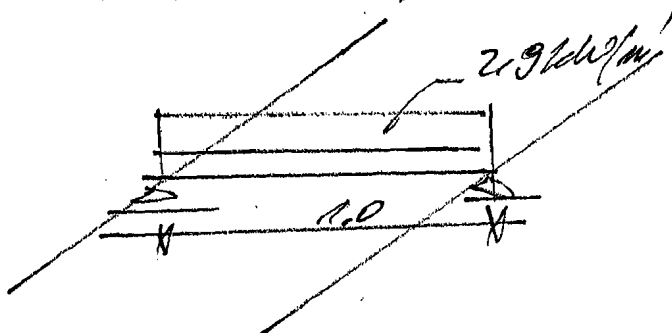
**POZICIJE ELEMENATA ČELIČNE
KONSTRUKCIJE**



OJAVLJANJE OKO SVJETLIKA

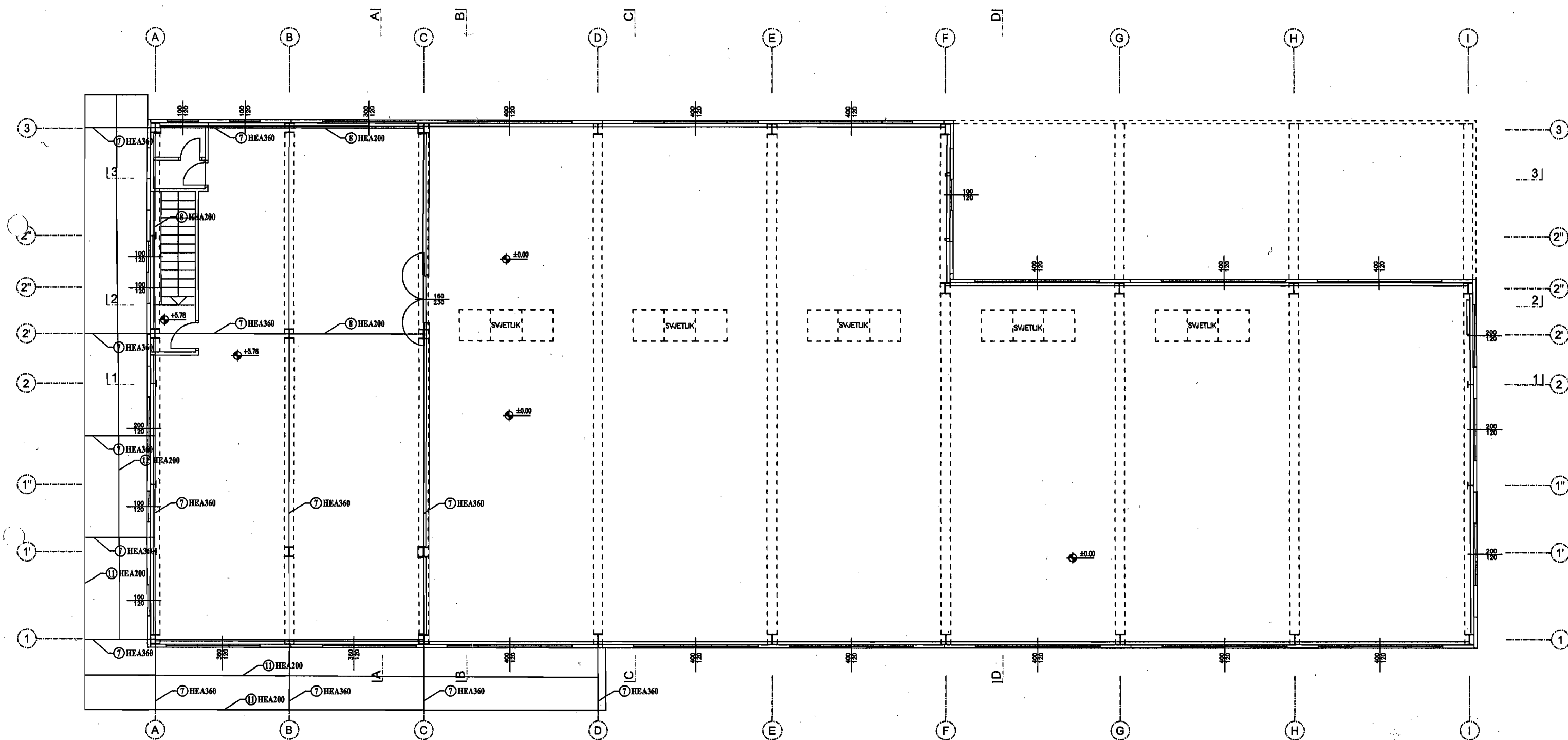
- opterećenja na (A)
- od T.I. PANELE - $0,6 \times 1,5 = 0,9 \text{ kN/m}^2$
- od svjetlika $1, \times 2,0 = \frac{2,0 \text{ kN/m}^2}{2,9 \text{ kN/m}^2}$

STAT. SHETA



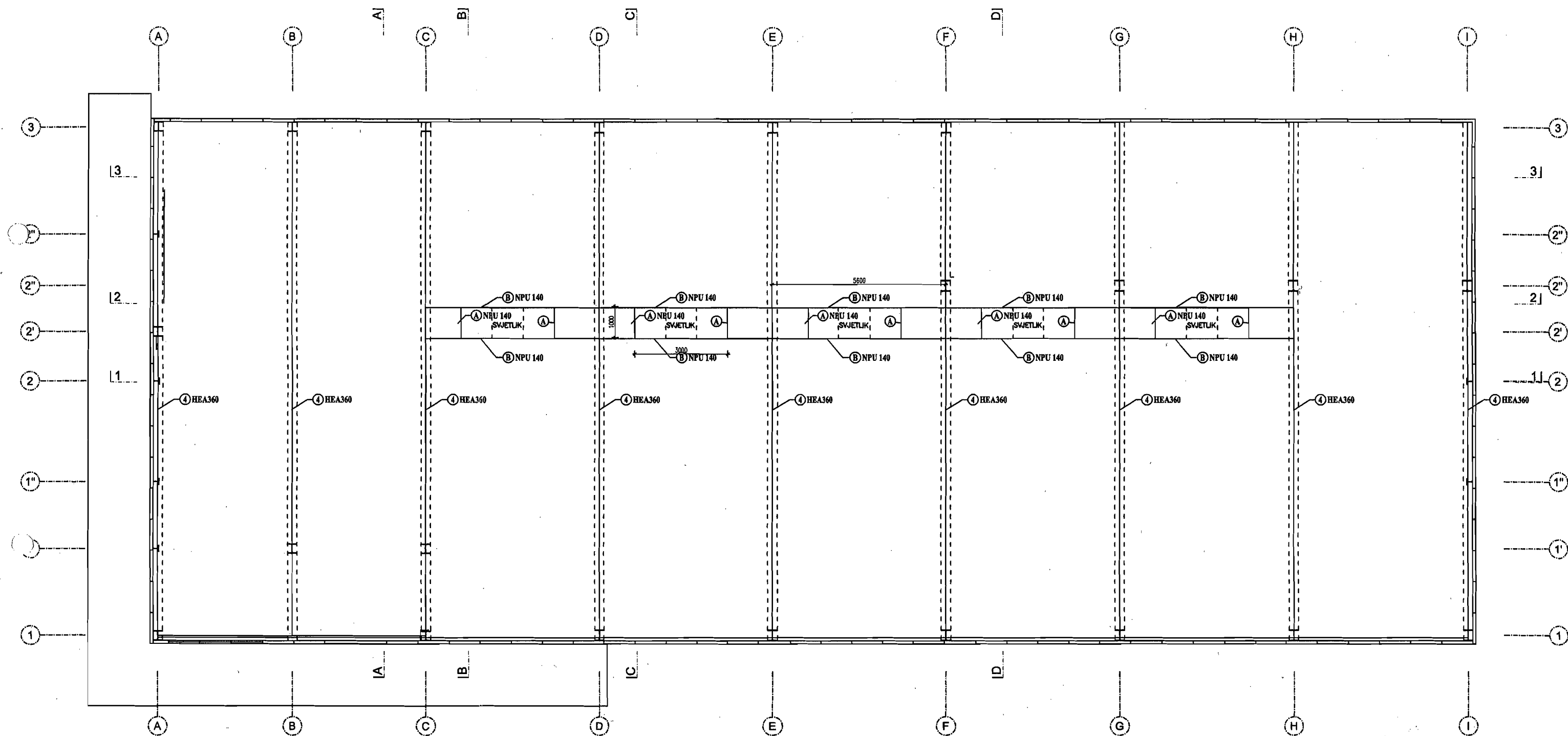
OPREĆEĆENJA NA (B)

- od (A) $\downarrow = 2,9 \times 1,0 \times 0,5 = 1,45 \text{ kN}$
- od svjetlika $= 2 \times 1 \times 0,5 = 1,0 \text{ kN/m}^2$
- + v.h. t.



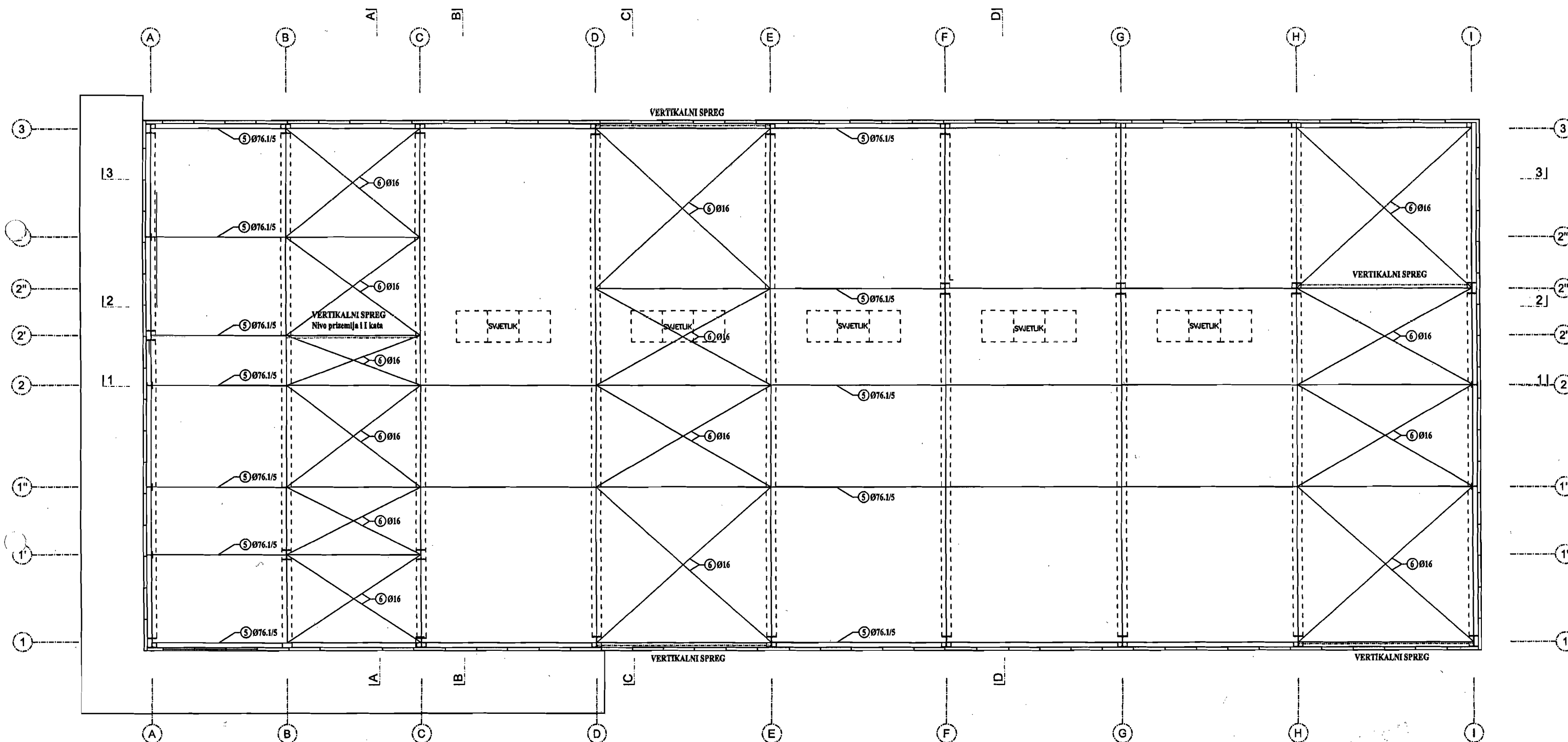
PLAN POZICIJA NA KOTI +5.780

STANOGRAD STUDIO d.o.o. Ulica Republike Austrije 7, 10000 Zagreb		t.d. 124/16
GRADJEVINA:	GOSPODARSKA GRAĐEVINA SKLADIŠNA HALA ZA BAZENSKU TEHNIKU	
INVESTITOR:	AQUACHEM d.o.o. za proizvodnju i trgovinu Ivanič-Grad	



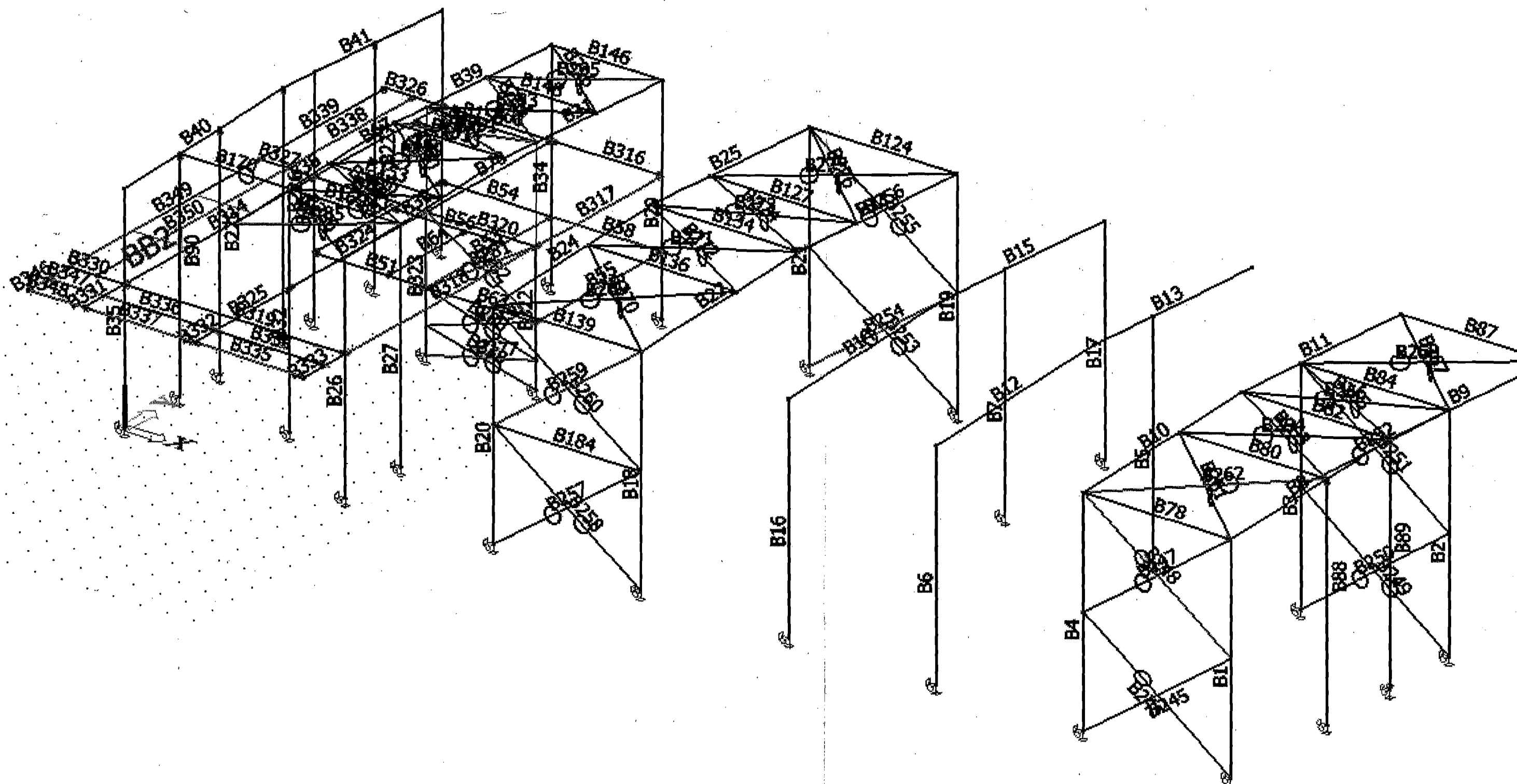
PLAN POZICIJA - KROVNA KONSTRUKCIJA

STANOGRAD STUDIO d.o.o. Ulica Republike Austrije 7, 10000 Zagreb		t.d. 124/16
GRADJEVINA:	GOSPODARSKA GRAĐEVINA SKLADIŠNA HALA ZA BAZENSKU TEHNIKU	
INVESTITOR:	AQUACHEM d.o.o. za proizvodnju i trgovinu Ivanič-Grad	

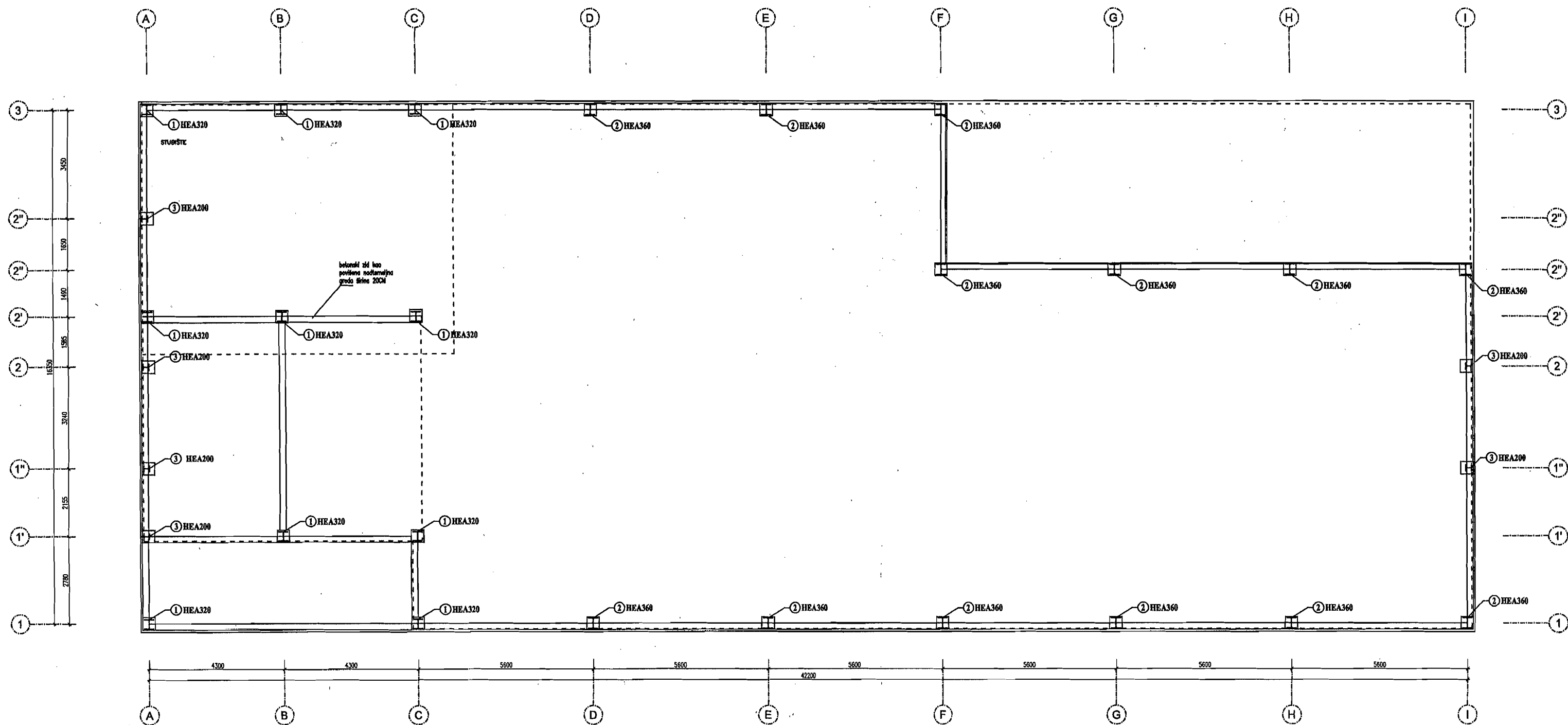


PLAN POZICIJA - KROVNI SPREGOVI

STANOGRAD STUDIO d.o.o.		t.d.
Ulica Republike Austrije 7, 10000 Zagreb		124/16
GRADJEVINA:	GOSPODARSKA GRAĐEVINA SKLADIŠNA HALA ZA BAZENSKU TEHNIKU	
INVESTITOR:	AQUACHEM d.o.o. za proizvodnju i trgovinu Ivanič-Grad	

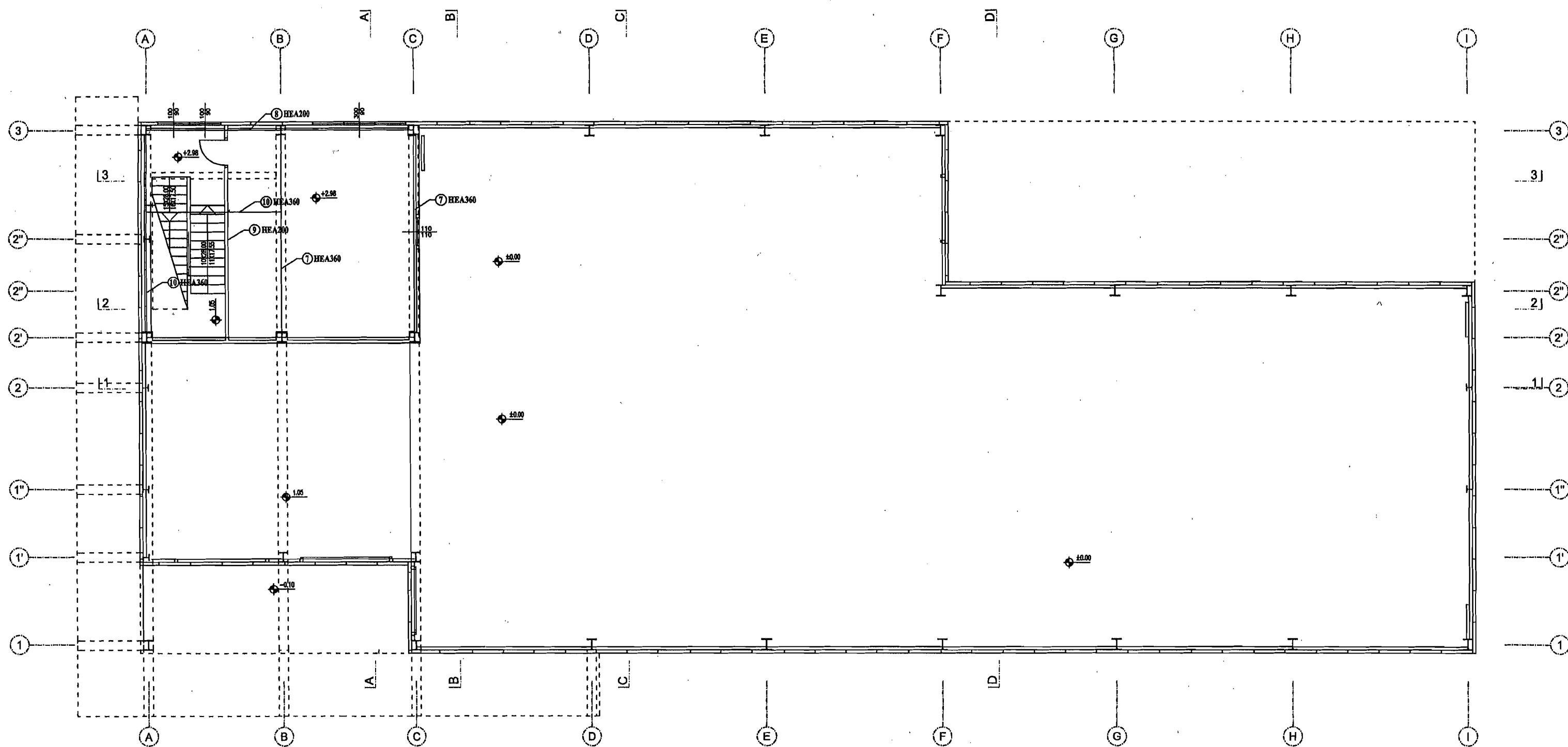


Pozicije elemenata



PLAN POZICIJA STUPOVA

STANOGRAD STUDIO d.o.o. Ulica Republike Austrije 7, 10000 Zagreb		t.d. 124/16
GRADJEVINA:	GOSPODARSKA GRAĐEVINA SKLADIŠNA HALA ZA BAZENSKU TEHNIKU	
INVESTITOR:	AQUACHEM d.o.o. za proizvodnju i trgovinu Ivanić-Grad	



PLAN POZICIJA NA KOTI +2.980

STANOGRAD STUDIO d.o.o. Ulica Republike Austrije 7, 10000 Zagreb		t.d. 124/16
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