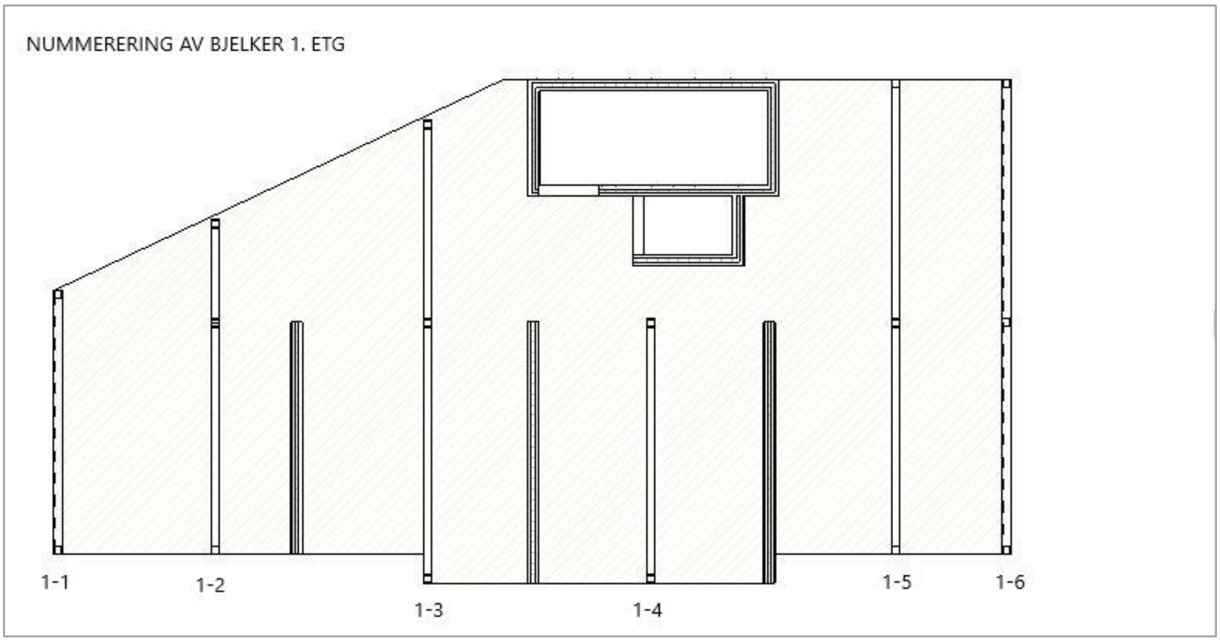
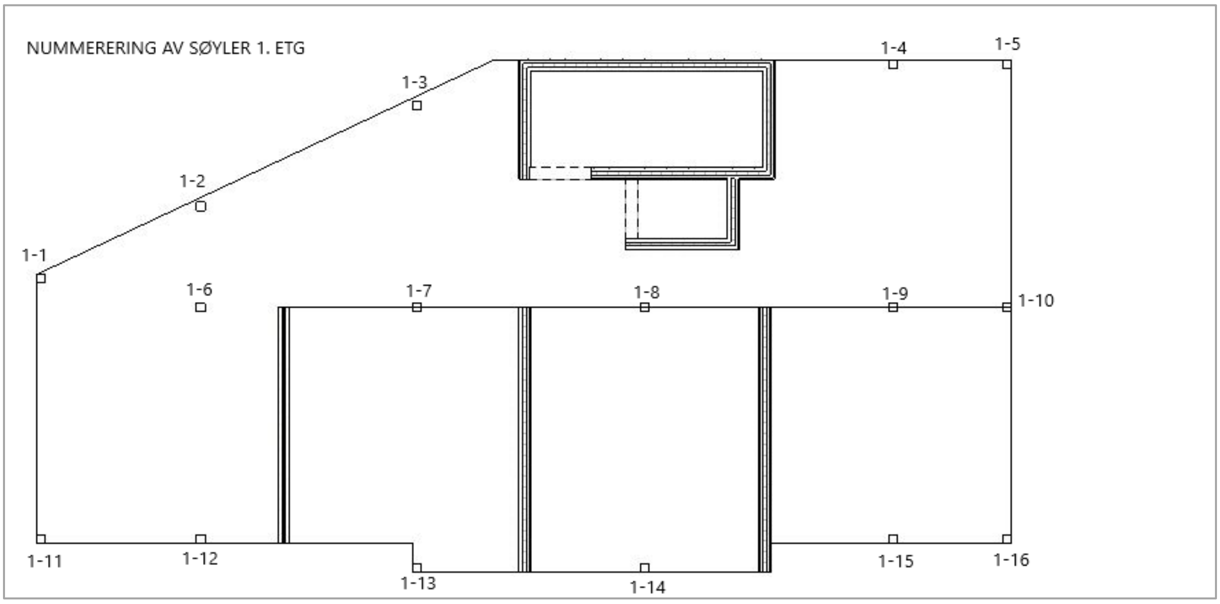
VEDLEGG

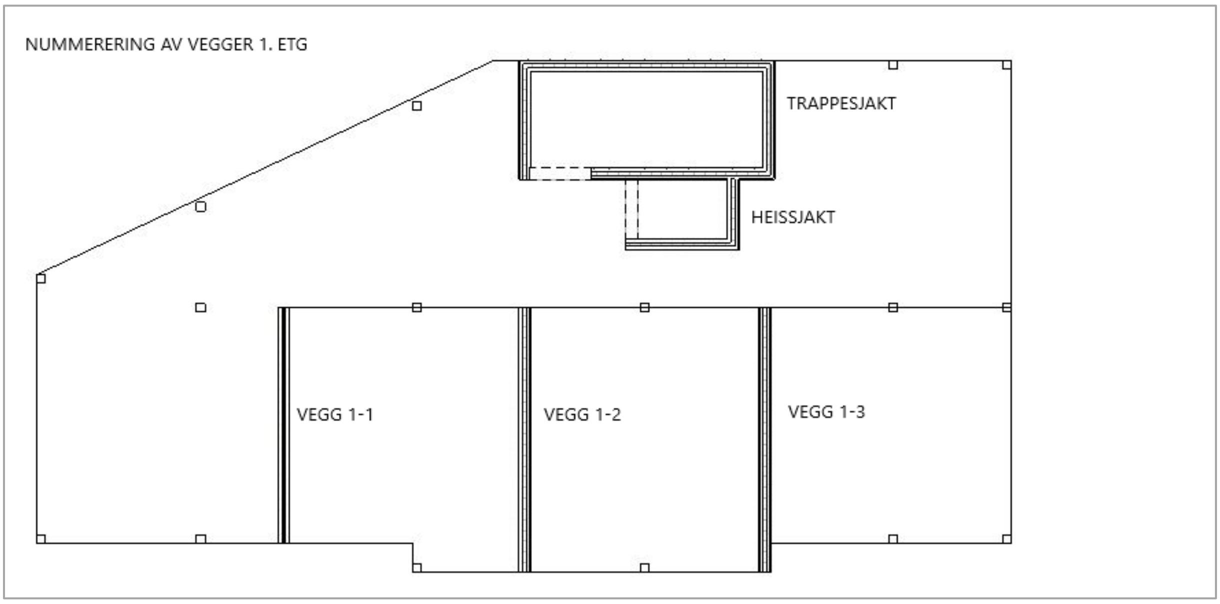
Innholdsfortegnelse

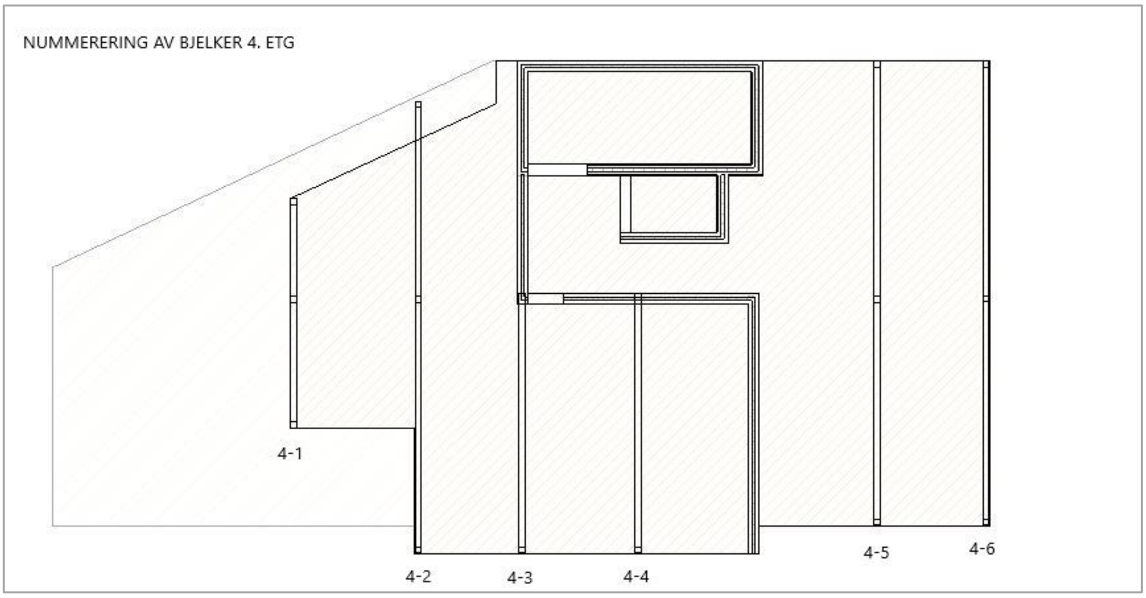
VEDLEGG A

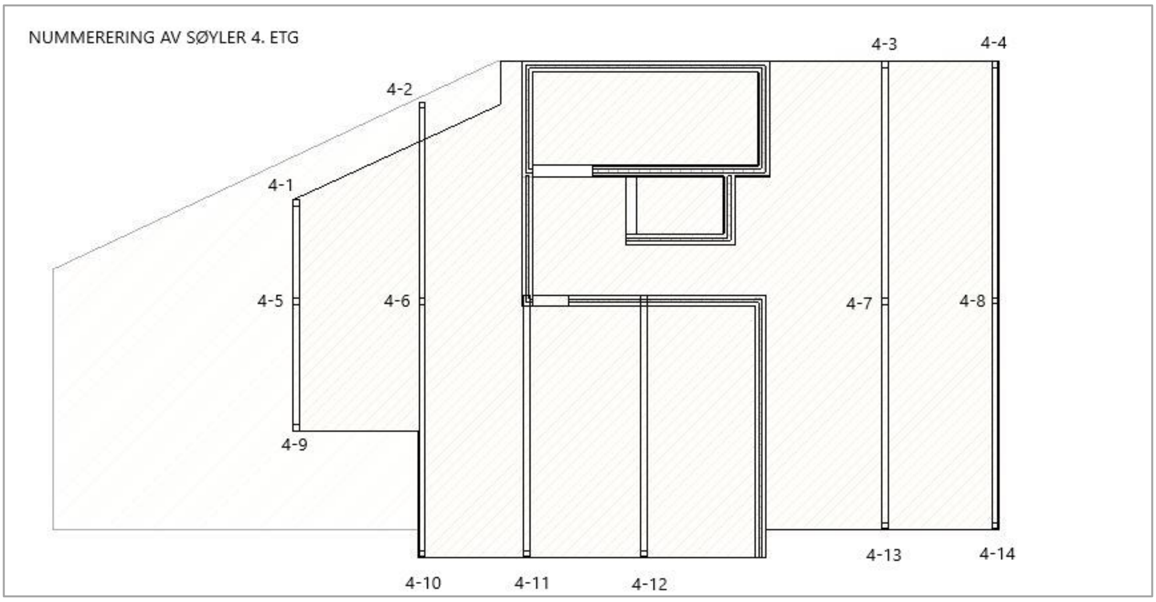
Nummerering av bjelke-, søyle- og veggsystemer


NUMMERERING AV BJELKER 1. ETG
1-1
1-2
1-3
1-4
1-5
1-6


NUMMERERING AV SØYLER 1. ETG
1-1
1-2
1-3
1-4
1-5
1-6
1-7
1-8
1-9
1-10
1-11
1-12
1-13
1-14
1-15
1-16


NUMMERERING AV VEGGER 1. ETG
TRAPPESJAKT
HEISSJAKT
VEGG 1-1
VEGG 1-2
VEGG 1-3


NUMMERERING AV BJELKER 4. ETG
4-1
4-2
4-3
4-4
4-5
4-6


NUMMERERING AV SØYLER 4. ETG
4-3
4-4
4-2
4-1
4-5
4-6
4-7
4-8
4-9
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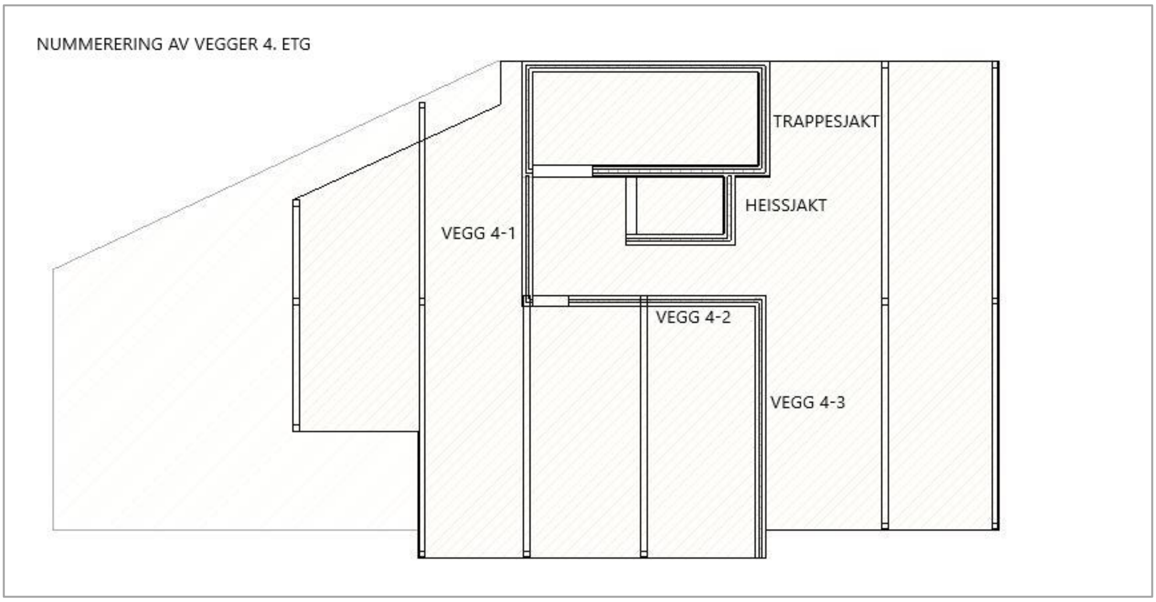
4-11
4-12
4-13
4-14


NUMMERERING AV VEGGER 4. ETG
TRAPPESJAKT
HEISSJAKT
VEGG 4-1
VEGG 4-2
VEGG 4-3

VEDLEGG B

Klimaregnskap

B. Klimaregnskap

B1. CO_2-utslipp stålsøyler
B2. One Click LCA massivtre
B3. One Click LCA betong og stål

VEDLEGG B1

Klimaregnskap stålsøyler			
kg/m	lengde per søyle (m)	kg stål per søyle	totalt antall søyler
15	2,6	39	56

kg CO2 per kg stål	kg CO2 per søyle	totalt CO2 utslipp søyler (kg)
2,1	81,9	4586,4

Omtrentlig verdi for 100x100x5 varmvalset hulprofil
Søylene har sammen et totalt CO2 utslipp på ca. 4,6 tonn
CO2 utslipp per norske innbygger i 2014 var 9,27 tonn

VEDLEGG B2

Hoved > Bygg C > Bygg C Tre >Life-cycle assessment, EN-15978

Bygg C Tre - Life-cycle assessment, EN-15978 Grunnleggende prosjektinformasjon

Resultatrapport: Bygg C Tre

Prosjekt	Bygg C - Bygg C Tre
Bruker	Nora Karlsen - 2020.05.24
Verktøy	Life-cycle assessment, EN-15978
Detaljer	Building life-cycle assessment according to the European Standard EN 15978. This LCA software covers life cycle stages from cradle to grave with separate reporting to product stage, construction process, use stage, operational energy, and end of life. This LCA software and related datasets are compliant with ISO 14040/14044 or EN 15804. It is compliant with the Active House Specification requirements.
Prosjektinformasjon og oppgaver	
Type (NS 3547)	15 - Boligblokk
Land	Norge
Adresse	Kronstadparken felt N3 Bygg C
Antall etasjer over bakken	4
Rammetype	notDetermined
Fulgte sertifiseringer	NS 3720
Fulgte sertifiseringer	NS 3720

Obligatorisk data mangler X

- Energiforbruk - Klikk for å taste inn manglende data
- Bygningsareal - Klikk for å taste inn manglende data

Kommersiell bruk er forbudt One Click LCA Student (International) Business license + Carbon Designer, UTDANNING, Nora Karlsen 24.05.2020 19:09

53 Tonn CO_2e

2 661 € Sosiale kostnader for karbon

Carbon Heroes Benchmark

Resultater

Life-cycle assessment results

	Sektor	Klimagassutslipp kg CO2e	Acidification kg SO2e	Eutrophication kg PO4e	Ozone depletion potential kg CFC11e	Formation of ozone of lower atmosphere kg Ethenee	Total use of primary energy ex. raw materials MJ	
A1-A3	Byggematerialer	3,07E4	1,6E2	2,8E1	4,12E−3	1,3E1	1,4E6	Detaljer
A4	Transport til byggeplassen	2,43E3	1,12E1	2,43E0	4,79E−4	1,37E−1	6,91E4	Detaljer
A5	Byggeplass							Skjul tomme
B1-B5	Maintenance and material replacement							Skjul tomme
B6	Energibruk i drift							Skjul tomme
B7	Water use							Skjul tomme
C1-C4	Livsløpets slutt	2,01E4	2,73E1	5,96E0	9,52E−9	2,25E0	7,25E4	Detaljer
D	Utover livsløp (ikke inkludert i totalen)	−1,19E5	−1,26E2	−2,02E1	−2,06E−7	−1,33E1	−2,09E6	Detaljer
	Total	**5,32E4**	**1,98E2**	**3,64E1**	**4,6E−3**	**1,54E1**	**1,54E6**	

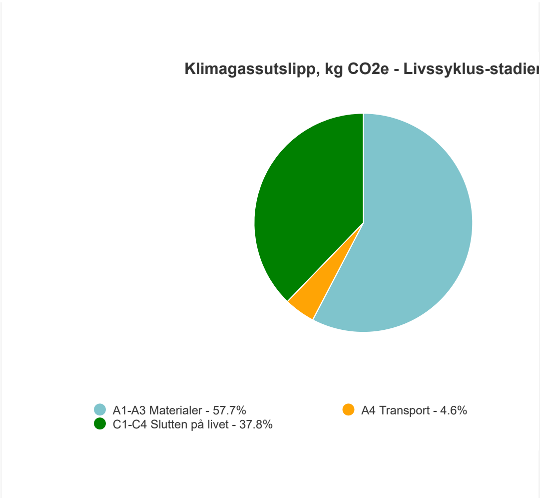
Fullstendighet og plausibilitetskontroll

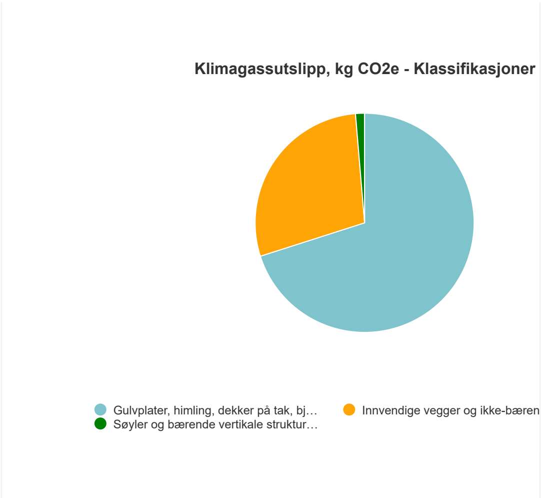
Mest medvirkende materialer (Klimagassutslipp)

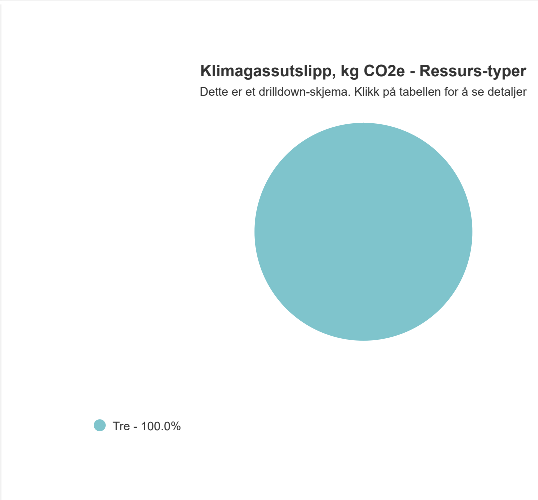
Grafer

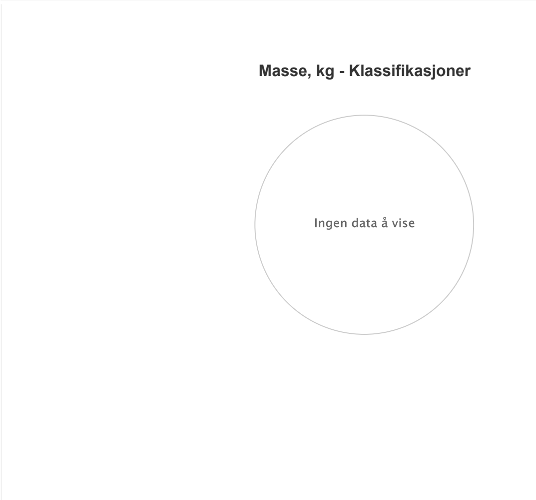
Oversikt over livssyklusen til Klimagassutslipp

Kake Linje Kolumn TreeMap









Vis datatabell: ☐ Klimagassutslipp - Livssyklus-stadier ☐ Klimagassutslipp - Klassifikasjoner ☐ Klimagassutslipp - Ressurs-typer ☐ Masse - Klassifikasjoner

Klimagassutslipp - Livssyklus-stadier

Enhet	Verdi	Enhet	Prosent %
A1-A3 Materialer	30 693,17	kg CO2e	57.68 %
A4 Transport	2 427	kg CO2e	4.56 %
C1-C4 Slutten på livet	20 092,68	kg CO2e	37.76 %

Klimagassutslipp - Klassifikasjoner

Enhet	Verdi	Enhet	Prosent %
Gulvplater, himling, dekker på tak, bjelker og tak (25, 26)	37 288,68	kg CO2e	70.07 %
Innvendige vegger og ikke-bærende strukturer (24)	15 219,02	kg CO2e	28.6 %
Søyler og bærende vertikale strukturer (22)	705,14	kg CO2e	1.33 %

Klimagassutslipp - Ressurs-typer

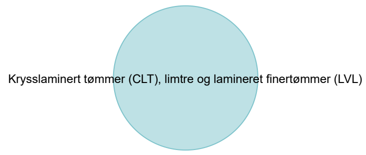
Enhet	Verdi	Enhet	Prosent %
Tre	53 212,85	kg CO2e	100.0 %

Masse - Klassifikasjoner

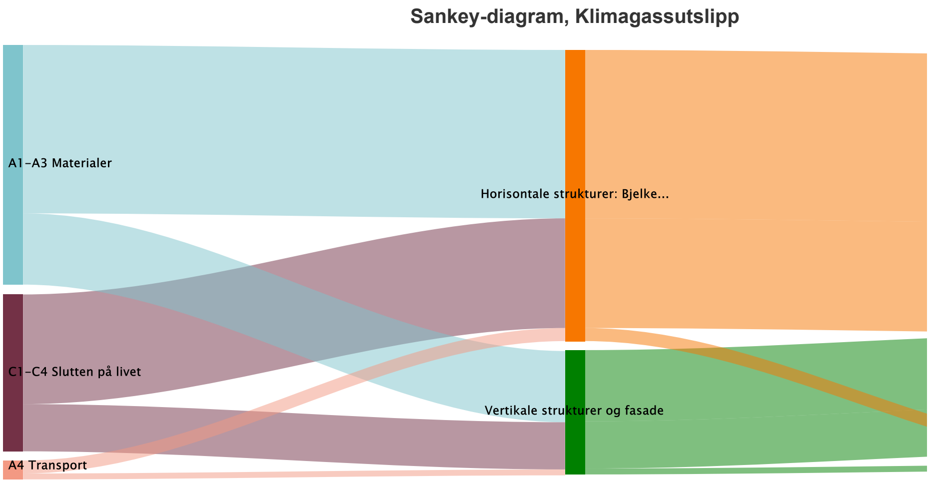
Enhet	Verdi	Enhet	Prosent %

Boblediagram, total livssykluspåvirkning etter ressurstype og undertype, Klimagas

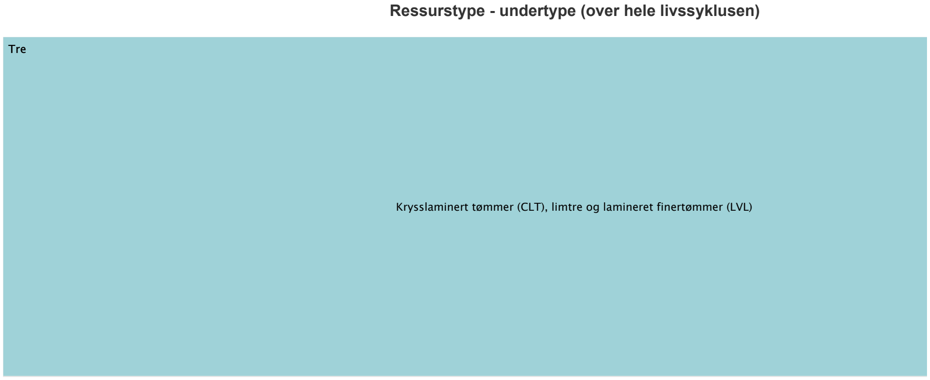
Hold musen over legendene eller boblene i diagrammet for å vise påvirkningene. Boblenes minste- og største størrelser er begrenset for les

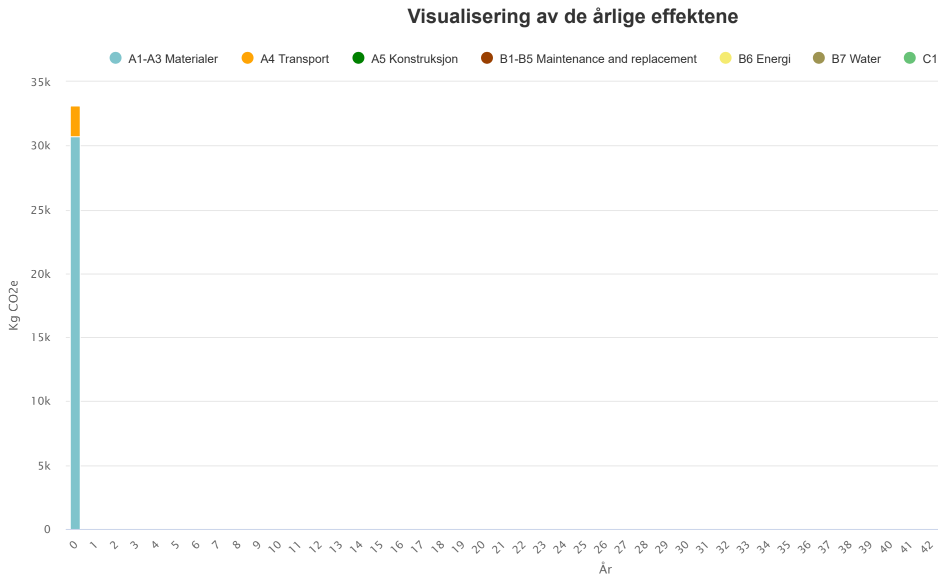


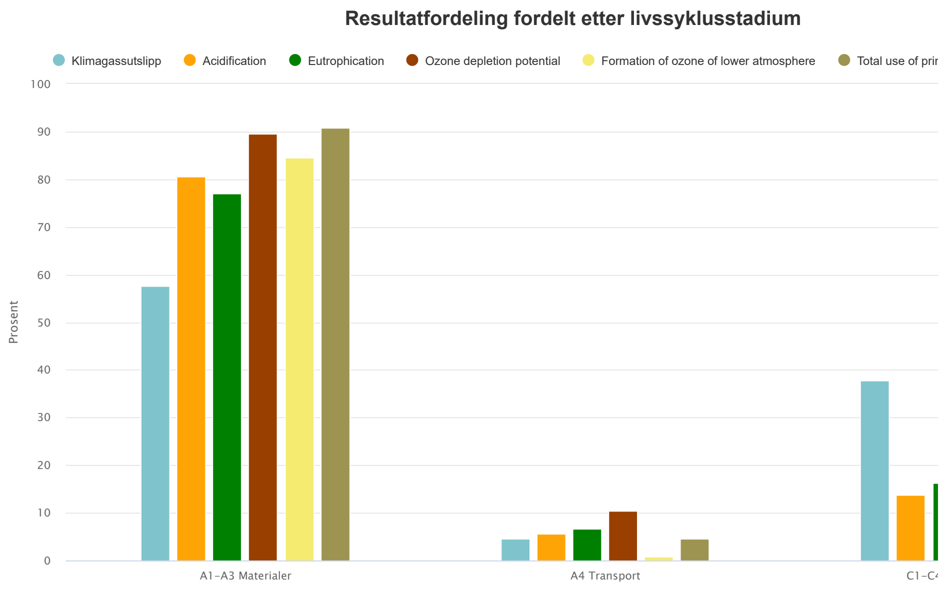
Konfigurer diagrammet ditt

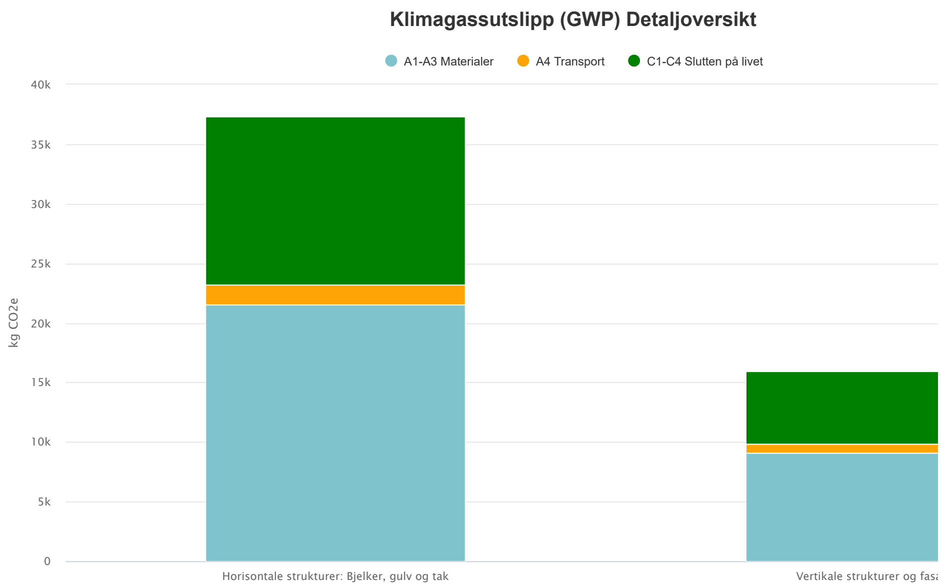


TreeMap , Klimagassutslipp


Ressurstype - undertype (over hele livssyklusen)
Tre
Krysslaminert tømmer (CLT), limtre og lamineret finertømmer (LVL)


Visualisering av de årlige effektene
A1-A3 Materialer
A4 Transport
A5 Konstruksjon
B1-B5 Maintenance and replacement
B6 Energi
B7 Water
C1
Kg CO2e
35k
30k
25k
20k
15k
10k
5k
0
År


Resultatfordeling fordelt etter livssyklusstadium
Klimagassutslipp
Acidification
Eutrophication
Ozone depletion potential
Formation of ozone of lower atmosphere
Total use of prir
Prosent
100
90
80
70
60
50
40
30
20
10
0
A1-A3 Materialer
A4 Transport
C1-C4


Klimagassutslipp (GWP) Detaljoversikt
A1-A3 Materialer
A4 Transport
C1-C4 Slutten på livet
kg CO2e
40k
35k
30k
25k
20k
15k
10k
5k
0
Horisontale strukturer: Bjelker, gulv og tak
Vertikale strukturer og fasa

+ Vis detaljert oversikt for alle kategorier

Datakilder

VEDLEGG B3

Hoved > Bygg C > Bygg C Betong/stål >Life-cycle assessment, EN-15978

Bygg C Betong/stål - Life-cycle assessment, EN-15978 Grunnleggende prosjektinformasjon

Resultatrapport: Bygg C Betong/stål

Prosjekt	Bygg C - Bygg C Betong/stål
Bruker	Nora Karlsen - 2020.05.16
Verktøy	Life-cycle assessment, EN-15978
Detaljer	Building life-cycle assessment according to the European Standard EN 15978. This LCA software covers life cycle stages from cradle to grave with separate reporting to product stage, construction process, use stage, operational energy, and end of life. This LCA software and related datasets are compliant with ISO 14040/14044 or EN 15804. It is compliant with the Active House Specification requirements.
Prosjektinformasjon og oppgaver	
Type (NS 3547)	15 - Boligblokk
Land	Norge
Adresse	Kronstadparken felt N3 Bygg C
Antall etasjer over bakken	4
Rammetype	notDetermined
Fulgte sertifiseringer	NS 3720
Fulgte sertifiseringer	NS 3720

Obligatorisk data mangler X

- Energiforbruk - Klikk for å taste inn manglende data

Kommersiell bruk er forbudt One Click LCA Student (International) Business license + Carbon Designer, UTDANNING, Nora Karlsen 16.05.2020 12:15

156 Tonn CO_2e **3 kg CO_2e / m2 / år** **7 782 € Sosiale kostnader for karbon**

Carbon Heroes Benchmark

Resultater

Life-cycle assessment results

	Sektor	Klimagassutslipp kg CO2e	Acidification kg SO2e	Eutrophication kg PO4e	Ozone depletion potential kg CFC11e	Formation of ozone of lower atmosphere kg Ethenee	Total use of primary energy ex. raw materials MJ	
A1-A3	Byggematerialer	1,52E5	2,74E2	4,78E1	3,53E−4	1,09E3	2,09E6	Detaljer
A4	Transport til byggeplassen	7,31E2	3,37E0	7,33E−1	1,44E−4	4,12E−2	2,08E4	Detaljer
A5	Byggeplass							Skjul tomme
B1-B5	Maintenance and material replacement							Skjul tomme
B6	Energibruk i drift							Skjul tomme
B7	Water use							Skjul tomme
C1-C4	Livsløpets slutt	2,68E3	2,09E1	4,32E0	5,4E−6	2,06E0	5,6E4	Detaljer
D	Utover livsløp (ikke inkludert i totalen)	−3,11E4	−6,87E1	−1,95E1	−6,71E−4	−8,31E0	−1,93E5	Detaljer
	Total	**1,56E5**	**2,98E2**	**5,29E1**	**5,02E−4**	**1,09E3**	**2,16E6**	
	Resultater per nevner							
	Brutto internt gulvareal (IPMS / RICS), m2 1119.0 m2	1,39E2	2,66E−1	4,73E−2	4,49E−7	9,76E−1	1,93E3	

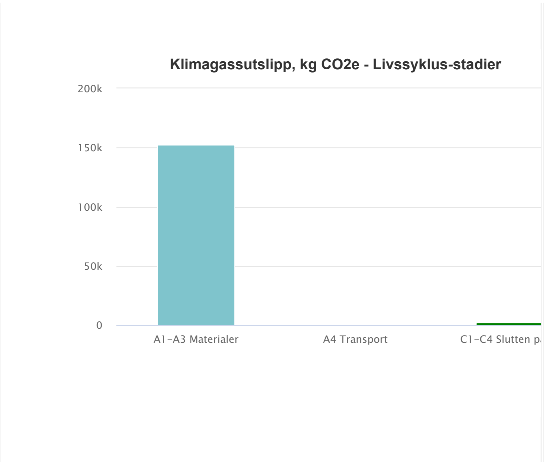
Fullstendighet og plausibilitetskontroll

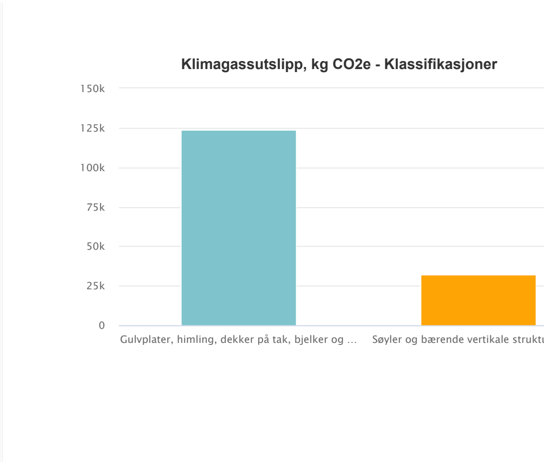
Mest medvirkende materialer (Klimagassutslipp)

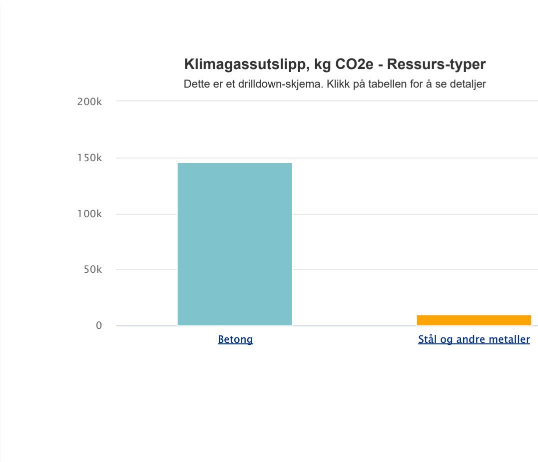
Grafer

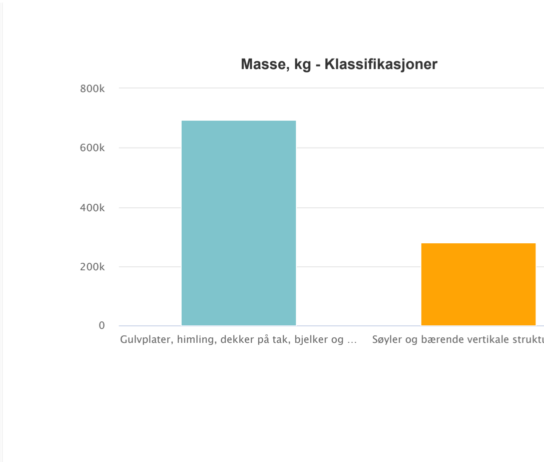
Oversikt over livssyklusen til Klimagassutslipp

Kake Linje **Kolumn** TreeMap









Vis datatabell: ☐ Klimagassutslipp - Livssyklus-stadier ☐ Klimagassutslipp - Klassifikasjoner ☐ Klimagassutslipp - Ressurs-typer ☐ Masse - Klassifikasjoner

Klimagassutslipp - Livssyklus-stadier

Enhet	Verdi	Enhet	Prosent %
A1-A3 Materialer	152 232,01	kg CO2e	97.81 %
A4 Transport	730,84	kg CO2e	0.47 %
C1-C4 Slutten på livet	2 675,43	kg CO2e	1.72 %

Klimagassutslipp - Klassifikasjoner

Enhet	Verdi	Enhet	Prosent %
Gulvplater, himling, dekker på tak, bjelker og tak (25, 26)	123 663,1	kg CO2e	79.46 %
Søyler og bærende vertikale strukturer (22)	31 975,17	kg CO2e	20.54 %

Klimagassutslipp - Ressurs-typer

Enhet	Verdi	Enhet	Prosent %
Betong	145 821,09	kg CO2e	93.69 %
og andre metaller	9 817,18	kg CO2e	6.31 %

Masse - Klassifikasjoner

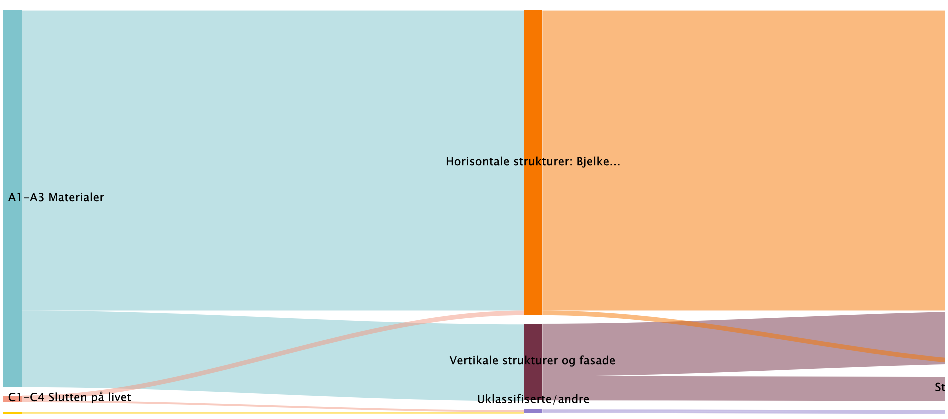
Enhet	Verdi	Enhet	Prosent %
Gulvplater, himling, dekker på tak, bjelker og tak (25, 26)	693 912,15	kg	71.26 %
Søyler og bærende vertikale strukturer (22)	279 811,75	kg	28.74 %

Boblediagram, total livssykluspåvirkning etter ressurstype og undertype, Klimagassutslipp

Hold musen over legendene eller boblene i diagrammet for å vise påvirkningene. Boblenes minste- og største størrelser er begrenset for lesbarhet

Konfigurer diagrammet ditt

Sankey-diagram, Klimagassutslipp



TreeMap , Klimagassutslipp

VEDLEGG C

Volumberegning

C. Volumberegning

C1. Volumberegning laminert tre

C2. Tabeller fra Revit-fil

VEDLEGG C1

Mengdeberegning av laminert tre i 2018

10455: Solgt produksjon av varer for store foretak i industri, etter prodcomkode 8 og statistikkvariabel	Solgt produksjon i 2018	Enhet
Høvellast av bartre	951161	m^3
Trelast av gran av arten 'Picea abies Karst.' eller edelgran (Abies alba mill.), tykkelse over 6 mm	354757	m^3
Skurlast og justert last av bartre	1327093	m^3
Tre i form av fliser eller spon, av bartrær	1401947	tonn
Laminerte bjelker og annet laminert tre	938488	tonn
Trapper av tre	1860	tonn

Omgjøring til antall kubikkmeter laminert tre	
ca. tyngdetetthet tre i tonn/kubikkmeter	0,4
ca. solgt produksjon av laminert tre i kubikkmeter	2346220

2,3 millioner kubikkmeter tre solgt i 2018 av norske produsenter

VEDLEGG C2

<Limtrebjelker>			
A	B	C	D
Type b x h	Overliggende etasje	Antall	Totalt volum
Limtrebjelke 215 x 585	2. etg	7	8.13 m^3
Limtrebjelke 215 x 585	3. etg	6	8.12 m^3
Limtrebjelke 215 x 585	4. etg	6	8.10 m^3
Limtrebjelke 215 x 495	Tak	6	6.57 m^3
Totalt: 25		25	30.91 m^3

<Limtresøyler>			
A	B	C	D
Type b x h	Beliggenhet	Antall	Totalt volum
Limtresøyle 215 x 315	1. etg	16	2.33 m^3
Limtresøyle 215 x 225	2. etg	16	1.63 m^3
Limtresøyle 215 x 225	3. etg	16	1.61 m^3
Limtresøyle 215 x 180	4. etg	14	1.16 m^3
Totalt: 62		62	6.73 m^3

<Tak og etasjeskiller i massivtre>					
A	B	C	D	E	F
Type	Beliggenhet	Areal	Structural Material	Tykkelse massivtre [mm]	Volum massivtre
Betong 200 mm	1. etg	307 m^2	<By Category>		0.00 m^3
Etasjeskiller massivtre	2. etg	307 m^2	Massivtre	180	55.21 m^3
Etasjeskiller massivtre	3. etg	307 m^2	Massivtre	180	55.21 m^3
Etasjeskiller massivtre	4. etg	226 m^2	Massivtre	180	40.69 m^3
Tak massivtre	4. etg	81 m^2	Massivtre	180	14.52 m^3
Tak massivtre	Tak	256 m^2	Massivtre	180	46.09 m^3
		1483 m^2			211.71 m^3

<Vegger i massivtre>					
A	B	C	D	E	F
Type	Beliggenhet	Structural Material	Tykkelse massivtre vegg [mm]	Areal	Volum massivtre vegg
Lydvegg	1. etg	Massivtre	200	331 m^2	66.14 m^3
Lydvegg	2. etg	Massivtre	200	58 m^2	11.54 m^3
Lydvegg	3. etg	Massivtre	200	57 m^2	11.40 m^3
Lydvegg	4. etg	Massivtre	200	44 m^2	8.73 m^3
Totalt: 19				489 m^2	97.81 m^3

VEDLEGG D

Personlig kommunikasjon med NODE

Nora Bjørvik <nbjorvik@gmail.com> 9. mars 2020 kl. 10:29
Til: Even Zachariassen Høyland <even.hoyland@node.no>

Hei! Hvor ligger grunnvannstanden på tomten?:)
[Sitert tekst skjult]

Even Zachariassen Høyland <even.hoyland@node.no> 9. mars 2020 kl. 13:38
Til: Nora Bjørvik <nbjorvik@gmail.com>

Hei!

Grunnvannstanden ligger under fundamentnivå. I tilegg legges det inn en drensledning som går rundt hele bygget (A, B og C) som sører for at det ikke kan samle seg overvann i byggegropen, se vedlagt sprengingsplan (foreløpig).

Med vennlig hilsen
Even Zachariassen Høyland
Sivilingeniør MSc
mob: (+47) 95 18 75 46

NODE rådgivende ingeniører AS
w w w . n o d e . n o
tel: (+47) 55 90 46 90
NODEbergen: C. Sundts gate 37, NO-5004 Bergen
NODEvoss: Bergsliplassen 2, NO-5700 Voss

[Sitert tekst skjult]

1811-BB01-U1-01-00A.pdf
821K

Nora Bjørvik <nbjorvik@gmail.com> 24. februar 2020 kl. 09:37
Til: Even Zachariassen Høyland <even.hoyland@node.no>

Hei! Vi har begynt å se på seismiske påvirkninger, og lurer derfor på hvilke grunnforhold det er?

fre. 8. nov. 2019 kl. 09:01 skrev Even Zachariassen Høyland <even.hoyland@node.no>:
[Sitert tekst skjult]

Even Zachariassen Høyland <even.hoyland@node.no> 24. februar 2020 kl. 10:38
Til: Nora Bjørvik <nbjorvik@gmail.com>

Hei!

Det er en utsprengt tomt og det skal fundamenteres på sprengstein i nærhet av fjell. Vi har benyttet grunntype A i våre beregninger.

Med vennlig hilsen
Even Zachariassen Høyland
Sivilingeniør MSc
mob: (+47) 95 18 75 46

NODE rådgivende ingeniører AS
w w w . n o d e . n o
tel: (+47) 55 90 46 90
NODEbergen: C. Sundts gate 37, NO-5004 Bergen
NODEvoss: Bergsliplassen 2, NO-5700 Voss

[Sitert tekst skjult]

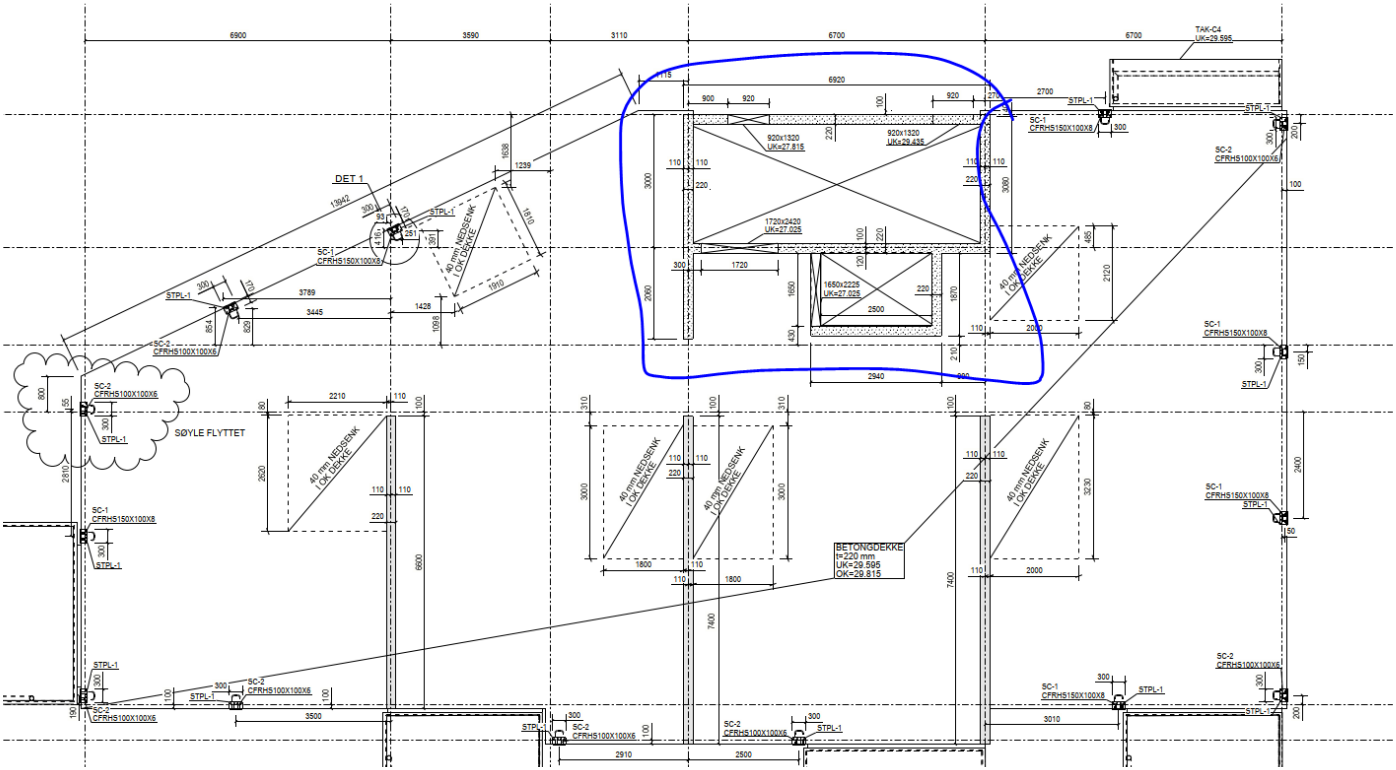
Nora Bjørvik <nbjorvik@gmail.com> 17. mars 2020 kl. 12:52
Til: Even Zachariassen Høyland <even.hoyland@node.no>

kjapt spørsmål: er det con-form som har levert dekkene og veggene?:)
[Sitert tekst skjult]

Even Zachariassen Høyland <even.hoyland@node.no> 17. mars 2
Til: Nora Bjørvik <nbjorvik@gmail.com>

Hei!

Con-form leverer de veggene som har fyll som ringet rundt under. NB: dette er vegger som blir støpt ut på stedet så i alle praktiske formål er dette en plasstøpt vegg. Dekker leveres av som plattendeker, men blir også støpt sammen på byggeplass så disse er også å betrakte som et plasstøpt dekke.



Med vennlig hilsen
Even Zachariassen Høyland
Sivilingeniør MSc
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NODEvoss: Bergsliplassen 2, NO-5700 Voss

[Sitert tekst skjult]

Nora Bjørvik <nbjorvik@gmail.com> 14. mai 2020 kl. 10:09
Til: Sigve Fossedal Olsen <sigve.fossedal.olsen@node.no>

Hei igjen!

Vi lurte bare på hvem som er leverandør av stålelementene i Bygg C? Er det Alsaker ute på Askøy?:)
[Sitert tekst skjult]

Nora Bjørvik <nbjorvik@gmail.com> 18. mai 2020 kl. 12:54
Til: Sigve Fossedal Olsen <sigve.fossedal.olsen@node.no>

Hei! Vi kommer til å levere oppgaven mandag om en uke. Ønsker dere å se gjennom den før det?:)
[Sitert tekst skjult]

Sigve Fossedal Olsen <sigve.fossedal.olsen@node.no> 18. mai 2020 kl. 13:51
Til: Nora Bjørvik <nbjorvik@gmail.com>

Hei!

Det er OK om dere sender en kopi av det ferdige resultatet. Om vi har eventuelle kommentarer de siste dagene av oppgaven, kan det skape usikkerhet og inødig "stress" som ikke er bra. Dere må stole på at det dere har gjort er bra, og fokusere på å finpusse til en ryddig og oversiktlig presentasjon.

Vedrørende stålleverandør: Det er Alsaker som leverer alt stål på Kronstadparken.

Med vennlig hilsen
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[Sitert tekst skjult]

VEDLEGG E

Lastberegninger

E. Lastberegninger

- E1. Snølast
- E2. Vindlast
  - *E2.1 Mathcad*
  - *E2.2 Lastberegninger OS-prog*
- E3. Seismisk påvirkning
  - *E3.1 Mathcad*
  - *E3.2 FEM-design*

VEDLEGG E1

SNØLAST

Referanser til standarder

NS-EN 1991-1-3: Laster på konstruksjoner - snølaster

NA.4.1
Tab NA.4.1(901)

$s_{k.0} := 2.0 \ \frac{kN}{m^2}$ $H_g := 150 \ m$ $\Delta s_k := 0.5 \ \frac{kN}{m^2}$ $s_{kmax} := 7.5 \ \frac{kN}{m^2}$

$H := 24 \ m$ $H < H_g$ $s_k := s_{k.0}$ $s_k = 2 \ \frac{kN}{m^2}$

Snølast på tak og takterrasse

Tab. 5.2 $0 \ ° \leq \alpha \leq 30 \ °$ $\mu_1 := 0.8$ (Takvinkel: 0°)

Tab. 5.1 $C_e := 1.0$ $C_t := 1.0$
5.2(8)
(5.1)

$S_1 := \mu_1 \cdot C_e \cdot C_t \cdot s_k$ $S_1 = 1.6 \ \frac{kN}{m^2}$

Snødriver flatt tak med parapet

6.2/Fig. 6.1 $\mu_{1s} := 0.8$ $h_p := 0.328 \ m$

$\mu_{2s} := \frac{(\gamma \cdot h_p)}{s_k}$ $\mu_{2s} = (9.466 \cdot 10^{-5}) \ \frac{m^2 \cdot s^2}{kg}$ $\mu_{2s} < 0.8$

Parapetet på 0.328m som går langs hele taket vil ha en svært liten innvirkning på opphopning av snø og vil ikke virke inn i på snølasten.

Fonning på takterrasse

5.3.6 $\alpha \leq 15 \ °$ $\mu_s := 0$

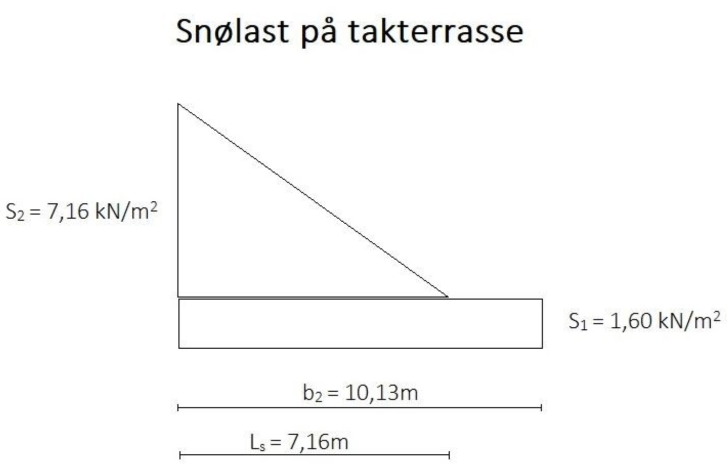
Fig 5.7 $b_1 := 16.41 \ m$ $b_2 := 10.13 \ m$ $\gamma := 2 \ \frac{kN}{m^3}$ $h := 3.58 \ m$

(5.8) $\mu_w := \frac{(b_1 + b_2)}{2 \cdot h}$ $\mu_w := \frac{(\gamma \cdot h)}{s_k}$ $\mu_w = 3.58$ $0.8 \leq \mu_w \leq 4$

(5.7) $\mu_2 := \mu_s + \mu_w$ $\mu_2 = 3.58$

$L_s := 2 \cdot h$ $\quad L_s = 7.16\ m$ $\quad 5\ m \le L_s \le 15\ m$

$S_2 := \mu_2 \cdot s_k$ $\quad S_2 = 7.16\ \frac{kN}{m^2}$



Beregningene forenkles ved å la trekantlasten fordele seg som en jevnt fordelt last over hele taket. Denne lasten legges sammen med den ordinære snølasten S_1 og gir en total, jevnt fordelt snølast.

$y := S_2 - S_1$ $\quad y = 5.56\ \frac{kN}{m^2}$

Sw er middelverdien til snølasten på grunn av fonning, omgjort fra trekantlast til en jevnt fordelt last:

$$S_w := \frac{\left(\frac{(y \cdot L_s)}{2}\right)}{b_2} \qquad S_w = 1.965\ \frac{kN}{m^2}$$

Den totale snølasten på terrassen:

$S_{total} := S_w + S_1$ $\quad S_{total} = 3.565\ \frac{kN}{m^2}$

VEDLEGG E2.1

VINDLAST

Referanser til standarder

[1] NS-EN 1991-1-4: Laster på konstruksjoner - vindlast

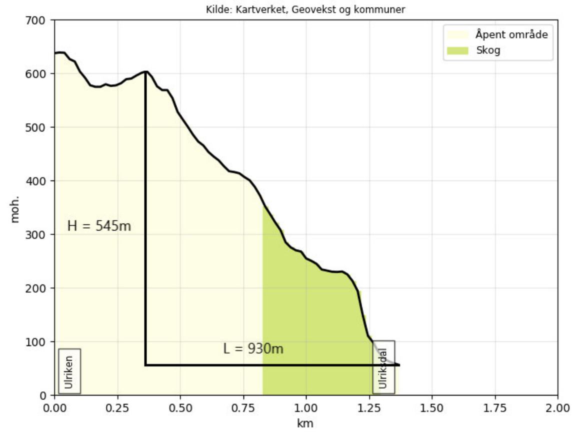
NA.4.3.3(901.4)

Byggested på lesiden av bratt terreng med fall større enn 30° i vindretningen

Ulriken

$$\alpha_{\text{Ulriken}} := \operatorname{atan}\left(\frac{545\ m}{930\ m}\right)$$

$$\alpha_{\text{Ulriken}} = 0.53$$



$$\alpha_U := \alpha_{\text{Ulriken}} \cdot \frac{360}{2\ \pi}$$

$$\alpha_U = 30.371$$

$$30\ ° < \alpha_U < 40\ °$$

$$H := 545\ m$$

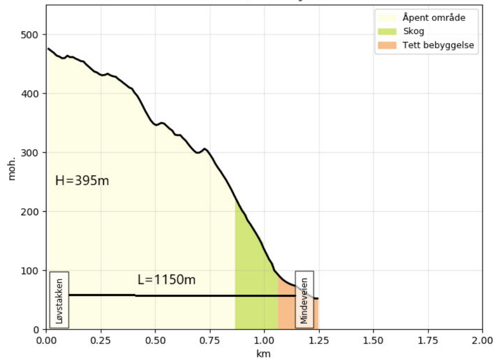
$8 \cdot H = 4.36\ km$ $\quad L_{byggsted} := 2.44\ km$ $\quad 8 \cdot H > L_{byggested}$

Dette gir: $C_0(z) := 0.9$ $\quad K_I := 1.75$ $\quad$ Terrengruhetskategori II

Løvstakken

$$\alpha_{Løvstakken} := \operatorname{atan}\left(\frac{475\ m - 60\ m}{1150\ m}\right)$$

$$\alpha_{Løvstakken} = 0.346$$



$$\alpha_L := \alpha_{Løvstakken} \cdot \frac{360}{2\ \pi}$$

$$\alpha_L = 19.843$$

$$\alpha_L < 30\ °$$

NA.4.3.3(901.4) Terrengruhetskategori II

Tab. NA.4.1 $k_r := 0.19$ $z_0 := 0.05\ m$

$z_{min} := 4\ m$ $z := 12.75\ m$ $z_{min} < z < z_{max}$

(4.4) $Cr(z) := k_r \cdot \ln\left(\frac{z}{z_0}\right)$ $Cr(z) = 1.053$

Tab. NA.4(901.1) $Vb_0 := 26\ \frac{m}{s}$

Tab. NA.4(901.2) $C_{dir} := 1.0$ $C_{season} := 1.0$ $C_{alt} := 1.0$ $C_{prob} := 1.0$
Tab. NA.4 (901.3)

$Vb := C_{dir} \cdot C_{season} \cdot C_{alt} \cdot C_{prob} \cdot Vb_0$ $Vb = 26\ \frac{m}{s}$

(4.3) $Vm(z) := C_0(z) \cdot Cr(z) \cdot Vb$ $Vm(z) = 24.636\ \frac{m}{s}$

NA 4.5 $\rho := 1.25\ \frac{kg}{m^3}$

$q_m(z) := \frac{1}{2} \cdot \rho \cdot Vm(z)^2$ $q_m(z) = 0.379\ \frac{kN}{m^2}$

$K_p := 3.5$

(4.7) $I_v(z) := \left(\frac{K_I}{C_0(z) \cdot \ln\left(\frac{z}{z_0}\right)}\right)$ $I_v(z) = 0.351$

(NA.4.8) $q_{po}(z) := \left(1 + 2 \cdot K_p \cdot I_v(z)\right) \cdot q_m(z)$ $q_{po}(z) = 1.311\ \frac{kN}{m^2}$

NA.4.3.2(2) V.6 Overgangsfaktor

Sone B: II Sone A: I

$X_b := 3.9\ km$ $\Delta n_{BA} := 2 - 1 = 1$

$$K_3 := 1.05 - \frac{3.9 - 2.5}{5 - 2.5} \cdot (1.05 - 1.00) \qquad K_3 = 1.022$$

$$q_{kast} := q_{po}(z) \cdot K_3 \qquad q_{kast} = 1.34 \ \frac{kN}{m^2}$$

7.2 Formfaktorer for bygninger

Benytter $Cpe_{,10}$ da dette gjelder dimensjonering av en bærekonstruksjon.

<u>Formfaktor vegger - vindfallsretning 0 grader (1)</u>

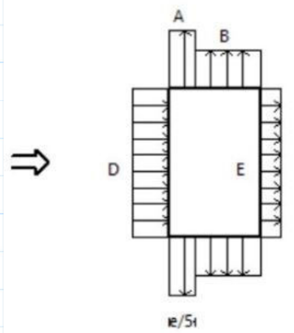
Tab. 7.1/Fig. 7.5 $\quad b_1 := 29490 \ mm \qquad d_1 := 18310 \ mm \qquad \frac{z}{d_1} = 0.696$

$$e_1 := min(b_1, 2 \cdot z) \qquad e_1 = 25.5 \ m$$

$$C_{A1} := -1.2 \qquad C_{B1} := -0.8 \qquad C_{C1} := 0$$

$$C_{D1} := 0.7 + \frac{0.8 - 0.7}{1 - 0.25} \cdot (0.696 - 0.25)$$

$$C_{D1} = 0.759$$



$$C_{E1} := -\left(0.3 + \frac{0.5 - 0.3}{1 - 0.25} \cdot (0.696 - 0.25)\right)$$

$$C_{E1} = -0.419$$

Gir vindlaster:

$$q_{A1} := q_{kast} \cdot C_{A1} \qquad q_{A1} = -1.608 \ \frac{kN}{m^2}$$

$$q_{B1} := q_{kast} \cdot C_{B1} \qquad q_{B1} = -1.072 \ \frac{kN}{m^2}$$

$$q_{C1} := q_{kast} \cdot C_{C1} \qquad q_{C1} = 0 \ \frac{kN}{m^2}$$

$$q_{D1} := q_{kast} \cdot C_{D1} \qquad q_{D1} = 1.018 \ \frac{kN}{m^2}$$

$$q_{E1} := q_{kast} \cdot C_{E1} \qquad q_{E1} = -0.561 \ \frac{kN}{m^2}$$

Formfaktor vegger - Vindfallsretning 90 grader (2)

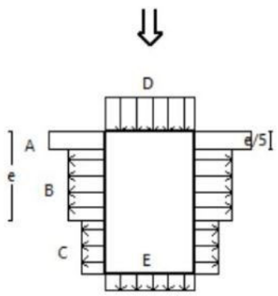
Tab. 7.1/Fig. 7.5

$$b_2 := 18310 \ mm \qquad d_2 := 29490 \ mm \qquad \frac{z}{d_2} = 0.432$$

$$e_2 := min(b_2, 2 \cdot z) \qquad e_2 = 18.31 \ m$$

$$C_{A2} := -1.2 \qquad C_{B2} := -0.8 \qquad C_{C2} := -0.5$$

$$C_{D2} := 0.7 + \frac{0.8 - 0.7}{1 - 0.25} \cdot (0.432 - 0.25)$$



$$C_{D2} = 0.724$$

$$C_{E2} := -\left(0.3 + \frac{0.5 - 0.3}{1 - 0.25} \cdot (0.432 - 0.25)\right)$$

$$C_{E2} = -0.349$$

Gir vindlaster:

$$q_{A2} := q_{kast} \cdot C_{A2} \qquad q_{A2} = -1.608 \ \frac{kN}{m^2}$$

$$q_{B2} := q_{kast} \cdot C_{B2} \qquad q_{B2} = -1.072 \ \frac{kN}{m^2}$$

$$q_{C2} := q_{kast} \cdot C_{C2} \qquad q_{C2} = -0.67 \ \frac{kN}{m^2}$$

$$q_{D2} := q_{kast} \cdot C_{D2} \qquad q_{D2} = 0.971 \ \frac{kN}{m^2}$$

$$q_{E2} := q_{kast} \cdot C_{E2} \qquad q_{E2} = -0.467 \ \frac{kN}{m^2}$$

Formfaktor flatt tak med parapet

Tab. 7.2/Fig. 7.6

$$h_p := 0.328 \ m \qquad h := 12.422 \ m$$

$$\frac{h_p}{h} = 0.026 \qquad 0.026 \approx 0.025$$

$C_F := -1.6 \qquad C_{I1} := 0.2$

$C_G := -1.1 \qquad C_{I2} := -0.2$

$C_H := -0.7$

Gir vindlaster:

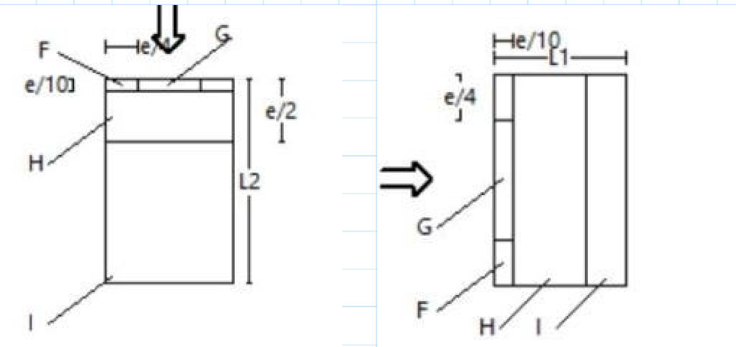
$q_F := q_{kast} \cdot C_F \qquad q_F = -2.144 \ \frac{kN}{m^2}$

$q_G := q_{kast} \cdot C_G \qquad q_G = -1.474 \ \frac{kN}{m^2}$

$q_H := q_{kast} \cdot C_H \qquad q_H = -0.938 \ \frac{kN}{m^2}$

$q_{I1} := q_{kast} \cdot C_{I1} \qquad q_{I1} = 0.268 \ \frac{kN}{m^2}$

$q_{I2} := q_{kast} \cdot C_{I2} \qquad q_{I2} = -0.268 \ \frac{kN}{m^2}$



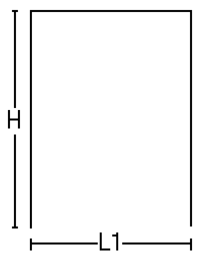
7.2.9 Innvendig vindtrykk

Etter punkt 7.2.9(6) Merknad 2 settes innvendig vindtrykk lik det ugunstigeste av +0,2 og -0,3.

Tittel			Side 1
Prosjekt	Ordre	Sign	Dato 18-03-2020

Dataprogram: LastBeregning versjon 6.2.5 Laget av Sletten Byggdata AS
Standard NS-EN 1991-1-4: Vindlaster
Data er lagret på fil:

1. Geometri



Vertikalsnitt

H	12422	mm
L1	18310	mm

Byggets lengde, L2: 29490 mm
Takvinkel : 0,00 (grader)

Parapet: hp/h=0,026

2. Vindhastighet

Fylke: Hordaland Kommune: Bergen Referansevindhastighet: 26 m/s
Byggested, høyde over havet (m): 24 Calt: 1
Returperiode (år):50 Cprob: 1
Årstidsfaktoren, Cseason: 1 hele året
Vindretning (region):Bruker retningsfaktoren C-ret: 1
Basisvindhastighet: 26 m/s
Høyde Z over grunnivået: 12,75 m

BYGGESTEDETS TERRENGDATA
Terrengruhetskategori II: Landbruksområde, område med spredte små bygninger eller trær.
Terrengruhetsfaktoren Kt: 0,19 Ruhetslengden Zo (m): 0,05 Zmin (m): 4 Vm (m/s): 27,37 Cr: 1,05

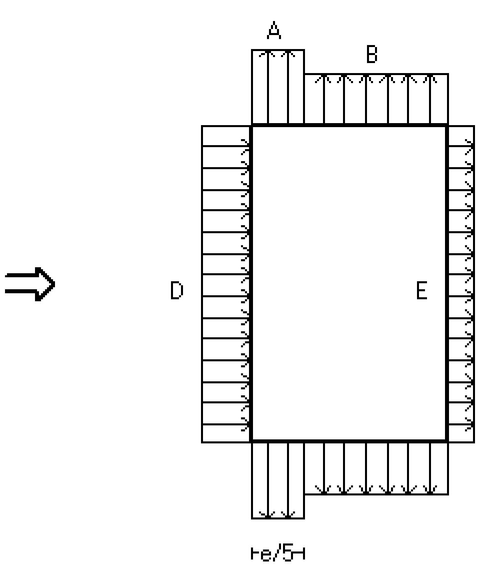
OVERGANGSONE
Terrengruhetskategori I: Kystnær, opprørt sjø. Åpne vidder og strandsoner uten trær eller busker.
Terrengruhetsfaktoren Kt: 0,17 Ruhetslengden Zo (m): 0,01 Zmin (m): 2 Vm (m/s) : 31,61 Cr: 1,22
Avstand mot vindretning fra byggested til grense for terrengkategoriendring Xb (m): 3900
Overgansonefaktor Cs(Xb): 1,04 Vm(z) : 28,4(lign NA.4(901.2/3))

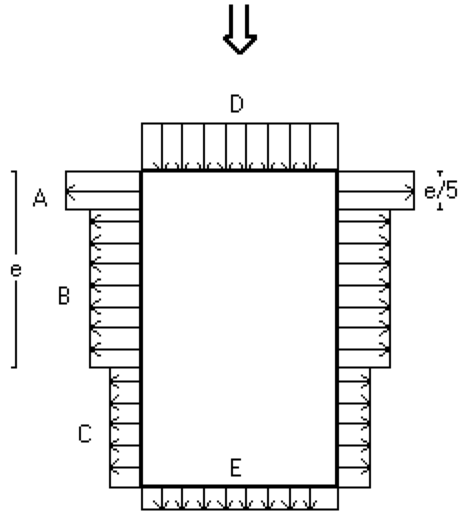
TOPOGRAFI: NA.4.3.3 (901.4) Byggested på lesiden av bratt terreng med fall større en 30 grader i vindretningen.
Terrengformfaktor Co(z): 0,9 Turbulensfaktor Ki: 1,75

Vkast: 44,25 m/s
Qkast: 1,224 kN/m2

3. Yttervegger

3.1 Utvendig vindlast





Vindretning 0 grader. *e=24844 mm*

Vindretning 90 grader. *e=18310 mm*

Vindinnfallsretning på 0 grader.

	A	B	C	D	E
Formfaktor Cpe,10	-1,20	-0,80		0,76	-0,41
Utvendig last (kN/m2)	-1,47	-0,98		0,93	-0,51
Formfaktor Cpe,1	-1,40	-1,10		1,00	-0,41
Utvendig last (kN/m2)	-1,71	-1,35		1,22	-0,51
Utstrekning (mm)	4969	13341		29490	29490

Vindinnfallsretning på 90 grader.

	A	B	C	D	E
Formfaktor Cpe,10	-1,20	-0,80	-0,50	0,72	-0,35
Utvendig last (kN/m2)	-1,47	-0,98	-0,61	0,88	-0,42
Formfaktor Cpe,1	-1,40	-1,10	-0,50	1,00	-0,35
Utvendig last (kN/m2)	-1,71	-1,35	-0,61	1,22	-0,42
Utstrekning (mm)	3662	14648	11180	18310	18310

Positiv verdi for last gir trykk. Negativ verdi hvis last er sug.

3.2 Innvendig vindlast

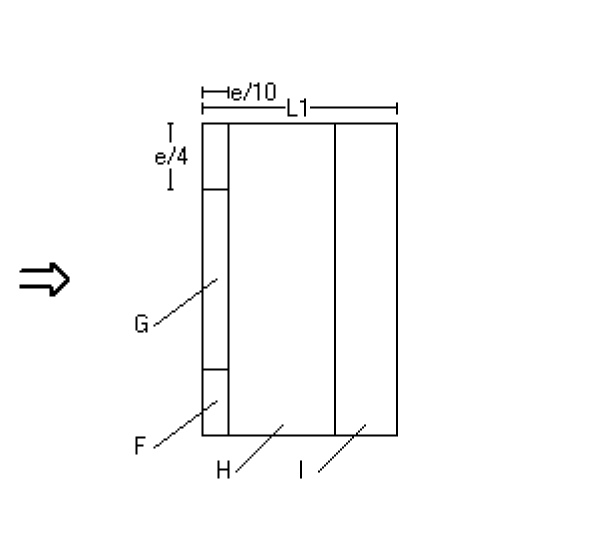
Bygning uten dominerende vindfasade

Beregn innvendig vindlast for u=0.2 overtrykk og u=-0.3 (undertrykk)

	Undertrykk	**Overtrykk**
Formfaktor	-0,30	0,20
Innvendig last (kN/m2)	-0,37	0,24

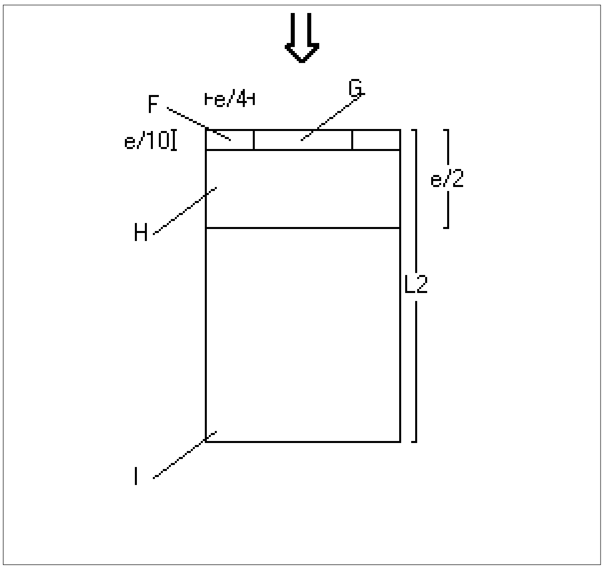
4 Overside av tak

Taktype: Flatt tak
L1=18310 mm L2=29490 mm
Cpe,10 Gjelder for hele bygget. (>=10m2)
Positiv verdi for last gir trykk. Negativ verdi hvis last er sug.



Utstrekning (mm)
e=24844
e/4=6211
e/10=2484

	Cpe,10	Last (kN/m2)	Hor.projeksjon (mm)
F	-1,59	-1,95	6211x2484
G	-1,09	-1,34	17068x2484
H	-0,70	-0,86	29490x9938
I	+/-0,20	+/-0,24	29490x5888



Utstrekning (mm)
e=18310
e/4=4578
e/10=1831

	Cpe,10	Last (kN/m2)	Hor.projeksjon (mm)
F	-1,59	-1,95	4578x1831
G	-1,09	-1,34	9155x1831
H	-0,70	-0,86	18310x7324
I	+/-0,20	+/-0,24	18310x20335

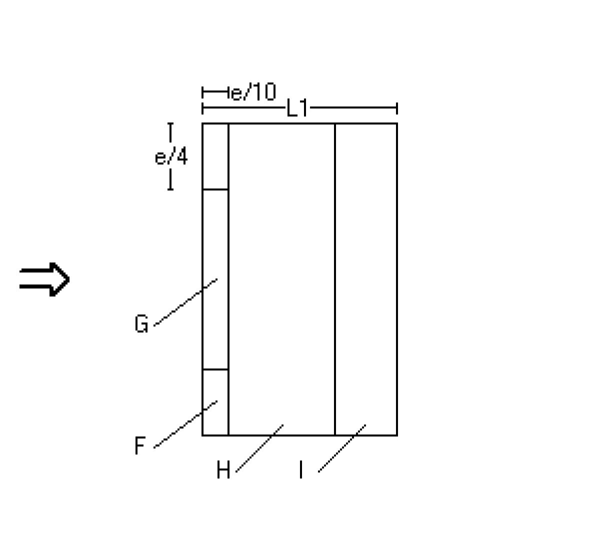
Taktype: Flatt tak

L1=18310 mm L2=29490 mm

Cpe,1 Gjelder for en lokal flate på 1m2. Benyttes ved dimensjonering av limfuger, spikring, båndstål o.l.

*Interpoleringsformel for belastet areal A mellom 1 og 10 m2 : Cpe = Cpe,1 + (Cpe,10 - Cpe,1) * log* $_{10}$*A*

Positiv verdi for last gir trykk. Negativ verdi hvis last er sug.



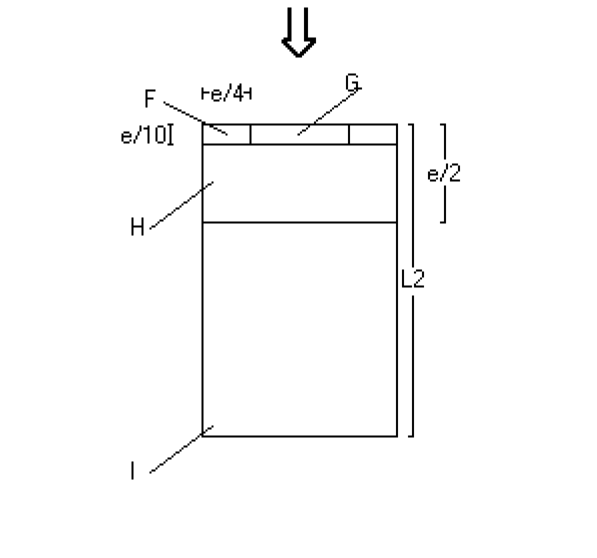
Utstrekning (mm)

e=24844

e/4=6211

e/10=2484

	Cpe,1	Last (kN/m2)	Hor.projeksjon(mm)
F	-2,19	-2,68	6211x2484
G	-1,79	-2,19	17068x2484
H	-1,20	-1,47	29490x9938
I	+/-0,20	+/-0,24	29490x5888



Utstrekning (mm)

e=18310

e/4=4578

e/10=1831

	Cpe,1	Last (kN/m2)	Hor.projeksjon(mm)
F	-2,19	-2,68	4578x1831
G	-1,79	-2,19	9155x1831
H	-1,20	-1,47	18310x7324
I	+/-0,20	+/-0,24	18310x20335

VEDLEGG E3.1

SEISMISK PÅVIRKNING

Referanse til standarder

[1] NS-EN 1998-1: Allmenne regler, seismiske laster og regler for bygninger

KRITERIE 1

[1] Fig. NA.3(901) $a_{g40Hz} := 0.85 \frac{m}{s^2}$ (Bergen)

[1] Tab NA.4(901) $\gamma_I := 1.0$ (Seismisk klasse II)

[1] Tab NA.3.3 $S := 1.0$ (Grunntype A)

[1] 3.2.2.2 **Dimensjonerende grunnakselrasjon**

$a_g := \gamma_I \cdot (0.8 \cdot a_{g40Hz})$ $a_g = 0.68 \frac{m}{s^2}$

$a_g \cdot S = 0.68 \frac{m}{s^2}$

$0.68 \frac{m}{s^2} > 0.49 \frac{m}{s^2}$ Ikke OK.

[1] NA.3.2.1(4) $0.68 \frac{m}{s^2} < 0.98 \frac{m}{s^2}$ Kan dimensjoneres etter bestemmelsene om lav seismisitet.

KRITERIE 4

[1] 4.3.3.2.2 $C_t := 0.050$ (Massivtre)

[1] NA.3.2.2.5 $\beta := 0.2$

[1] 5.3.3 $q := 1.5$

[1] Tab. NA.3.3 $T_B := 0.1\ s$ $T_C := 0.2\ s$ $T_D := 1.7\ s$

$H := 12.422\ m$

$T_1 := C_t \cdot H^{\frac{3}{4}}$ $T_1 := 0.331\ s$

[1] 3.2.2.5 $T_C < T_1 < T_D$

$$S_d\left(T_1\right) := a_g \cdot S \cdot \frac{2.5}{q} \cdot \left(\frac{T_C}{T_1}\right)$$

$$S_d\left(T_1\right) = 0.685 \ \frac{m}{s^2} \quad > \quad \beta \cdot a_g = 0.136 \ \frac{m}{s^2}$$

$$S_d\left(T_1\right) > 0.49 \ \frac{m}{s^2}$$ Ikke OK.

<u>KRITERIE 5</u>

Kontrollerer kapasiteten ved å sammenligne skjærkraften som er i overkant av den stive kjelleren fra jordskjelv med kraft fra vind og skjevstilling.

Betingelser

[1] 4.2.3.3 Kriterier for regularitet i oppriss

$L := 27.17\ m \qquad L_1 := L \qquad L_2 := 20.48\ m$

$$\frac{\left(L - L_2\right)}{L} = 0.246 \qquad \frac{\left(L - L_2\right)}{L} \le 0.30$$ OK.

$$\frac{\left(L_1 - L_2\right)}{L_1} = 0.246 \qquad \frac{\left(L_1 - L_2\right)}{L_1} \le 0.10$$ Ikke OK.

Går videre i beregningene som om kriterier for regularitet i oppriss er oppfylt.

Horisontalkraft ved overkant av stiv kjeller

<u>Masse til bygget (over kjeller)</u>

Kun overslagsberegninger for massen til bygget. Antall, lengder og dimensjoner er satt som en omtrent verdi. Dimensjoner er hentet fra Solibri.

Søyler

$$G_{s1} := 4.609\ \frac{kN}{m^3} \cdot 215\ mm \cdot 315\ mm \cdot 2.15\ m \qquad G_{s1} = 671.111\ N$$

$$G_{s2} := 4.609\ \frac{kN}{m^3} \cdot 215\ mm \cdot 225\ mm \cdot 2.15\ m \qquad G_{s2} = 479.365\ N$$

$$G_{s3} := 4.609 \frac{kN}{m^3} \cdot 215\ mm \cdot 225\ mm\ 2.15\ m \qquad G_{s3} = 479.365\ N$$

$$G_{s4} := 4.609 \frac{kN}{m^3} \cdot 215\ mm \cdot 180\ mm \cdot 2.15\ m \qquad G_{s4} = 383.492\ N$$

$$m_s := G_{s1} \cdot 16 + G_{s2} \cdot 16 + G_{s3} \cdot 16 + G_{s4} \cdot 14 \qquad m_s = 31.446\ kN$$

Bjelker

$$G_{b1} := 215\ mm \cdot 585\ mm \cdot 13.5\ m \cdot 430 \frac{kg}{m^3} \cdot 9.81 \frac{m}{s^2} \qquad G_{b1} = 7.163\ kN$$

$$G_{b2} := 215\ mm \cdot 585\ mm \cdot 13.5\ m \cdot 430 \frac{kg}{m^3} \cdot 9.81 \frac{m}{s^2} \qquad G_{b2} = 7.163\ kN$$

$$G_{b3} := 215\ mm \cdot 585\ mm \cdot 13.5\ m \cdot 430 \frac{kg}{m^3} \cdot 9.81 \frac{m}{s^2} \qquad G_{b3} = 7.163\ kN$$

$$G_{b4} := 215\ mm \cdot 495\ mm \cdot 13.5\ m \cdot 430 \frac{kg}{m^3} \cdot 9.81 \frac{m}{s^2} \qquad G_{b4} = 6.061\ kN$$

$$m_b := G_{b1} \cdot 6 + G_{b2} \cdot 6 + G_{b3} \cdot 6 + G_{b4} \cdot 6 \qquad m_b = 165.289\ kN$$

Tak og takterrasse

$$m_t := \left(254\ m^2 + 151\ m^2\right) \cdot 1.025 \frac{kN}{m^2} \qquad m_t = 415.125\ kN$$

Etasjeskillere

$$m_e := \left(192\ m^2 + 3 \cdot 310.5\ m^2\right) \cdot 2.595 \frac{kN}{m^2} \qquad m_e = \left(2.915 \cdot 10^3\right)\ kN$$

Bærende vegger

$$m_v := \left(216\ m^2\right) \cdot 1.147 \frac{kN}{m^2} \qquad m_v = 247.752\ kN$$

Heis- og trappesjakt av massivtre

$$m_h := \left(4 \cdot 78.3\ m^2\right) \cdot 1.2 \frac{kN}{m^2} \qquad m_h = 375.84\ kN$$

Sum masse

$$m := \frac{(m_s + m_b + m_t + m_e + m_v + m_h)}{9.81 \frac{m}{s^2}} \qquad m = (4.231 \cdot 10^5)\ kg$$

Jordskjelv vs. vind og skjevstilling

[1] 4.3.3.2.2 $T_1 \le 2 \cdot T_C$ $\lambda := 0.85$

Materialfaktorer $\gamma_{c.bruddgrense} := 1.15$ $\gamma_{cDCL} := 1.0$

Kraft fra vindpåkjenning $vind_x := 424\ kN$ $vind_y := 588\ kN$

Skjevstillingslast $skjev_x := 150\ kN$ $skjev_y := 155\ kN$

(Kraft fra vind og skjevstilling er hentet fra beregninger gjort i FEM-design)

Horisontalkraft grunnet jordskjelv

$$F_b := S_d(T_1) \cdot m \cdot \lambda \qquad F_b = 246.295\ kN$$

Påkjennng fra vind og skjevstilling sammenlignet med jordskjelv:

$$1.5 \cdot vind_x + 1.05 \cdot skjev_x \cdot \left(\frac{\gamma_{c.bruddgrense}}{\gamma_{cDCL}}\right) = 817.125\ kN \qquad > F_b = 246.295\ kN$$

$$1.5 \cdot vind_y + 1.05 \cdot skjev_y \cdot \left(\frac{\gamma_{c.bruddgrense}}{\gamma_{cDCL}}\right) = (1.069 \cdot 10^3)\ kN \qquad > F_b = 246.295\ kN$$

Videre kapasitetskontroll kan utelates, om man ser bort fra bibetingelsene.

2 - 7100 Global analyse

VEDLEGG E3.2

2.1 Globale kontroller

Equilibrium, Ultimate - Load case

Case	Component	Loads	Reactions	Error
[-]	[-]	kN(m)	kN(m)	[%]
Egenvekt	Fx'	0	-0	-
	Fy'	0	0	-
	Fz'	-5474	5474	0.00
	Mx'	-81012	81012	0.00
	My'	146934	-146934	0.00
	Mz'	0	0	-
Nyttelast	Fx'	0	-0	-
	Fy'	0	0	-
	Fz'	-2923	2923	0.00
	Mx'	-42851	42851	0.00
	My'	76353	-76353	0.00
	Mz'	0	0	-
Snølast	Fx'	0	-0	-
	Fy'	0	-0	-
	Fz'	-475	475	0.00
	Mx'	-7385	7385	0.00
	My'	13887	-13887	0.00
	Mz'	0	0	-
Påført egenlast	Fx'	0	-0	-
	Fy'	0	0	-
	Fz'	-2861	2861	0.00
	Mx'	-42397	42397	0.00
	My'	74033	-74033	0.00
	Mz'	0	0	-
Skjevstilling x	Fx'	150	-150	0.00
	Fy'	0	0	-
	Fz'	0	0	-
	Mx'	0	-0	-
	My'	1340	-1340	0.00
	Mz'	-2285	2285	0.00
Skjevstilling y	Fx'	0	-0	-
	Fy'	155	-155	0.00
	Fz'	0	-0	-
	Mx'	-1401	1401	0.00
	My'	0	0	-
	Mz'	4129	-4129	0.00
Vind X+	Fx'	424	-424	0.00
	Fy'	0	0	-
	Fz'	0	0	-
	Mx'	0	-0	-
	My'	3687	-3687	0.00
	Mz'	-7311	7311	0.00
Vind X-	Fx'	-280	280	0.00
	Fy'	0	-0	-
	Fz'	0	-0	-
	Mx'	0	0	-
	My'	-2473	2473	0.00
	Mz'	4420	-4420	0.00
Vind Y+	Fx'	0	-0	-
	Fy'	561	-561	0.00
	Fz'	0	-0	-
	Mx'	-4856	4856	0.00
	My'	0	-0	-
	Mz'	14493	-14493	0.00
Vind Y-	Fx'	0	0	-
	Fy'	-588	588	0.00
	Fz'	0	0	-
	Mx'	5081	-5081	0.00
	My'	0	0	-
	Mz'	-14982	14982	0.00

VEDLEGG F

Bjelker

VEDLEGG F1.1

BJELKE 4-4

Referanse til standarder

[1] NS-EN 1995-1-1: Allmenne regler og regler for bygninger
[2] NS-EN 1990: Grunnlag for prosjektering av konstruksjoner
[3] Norsk Limtreprodusenters Forening, *Limtreboka - Beregningseksempler* (2018)
[4] NS-EN 1995-1-2: Brannteknisk dimensjonering
[5] Byggeteknisk forskrift (TEK17)

FORUTSETNINGER OG ANTAKELSER

[1] NA.901 Klimaklasse: 1

[1] Tab. 2.1 Lastvarighetsklasse for snølast: Korttidslast
Lastvarighetsklasse for vindlasten: Øyeblikkslast

[1] NA.2.3 Partialfaktor for limtre: $\gamma_M := 1.15$

Lengde: $L := 7300\ mm$

Bjelketverrrsnitt: $b := 215\ mm$ $h := 495\ mm$

Lastbredde: $Lb := 3300\ mm$

Karakteristisk vindlast: $q_{kast} := 1.34\ \frac{kN}{m^2}$

[3] Tab.6 Limtre GL30c:

$\rho_m := 430\ \frac{kg}{m^3}$ $g := 9.81\ \frac{m}{s^2}$ $f_{m.k} := 30\ \frac{N}{mm^2}$

$E_{0.05} := 10800\ \frac{N}{mm^2}$ $f_{c.90k} := 2.5\ \frac{N}{mm^2}$ $f_{v.k} := 3.5\ \frac{N}{mm^2}$

$E_{mean} := 13000\ \frac{N}{mm^2}$

LASTER

Egenlast tak: $g_{d.k} := 1.025\ \frac{kN}{m^2} \cdot Lb$ $g_{d.k} = 3.383\ \frac{kN}{m}$

Egenlast bjelke: $g_{b.k} := b \cdot h \cdot \rho_m \cdot g$ $g_{b.k} = 0.449\ \frac{kN}{m}$

Permanent last: $g_k := g_{d.k} + g_{b.k}$ $g_k = 3.831\ \frac{kN}{m}$

Snølast: $s_k := 1.6\ \frac{kN}{m^2} \cdot Lb$ $s_k = 5.28\ \frac{kN}{m}$

Vindlast: $v_{t.k} := (0.2 + 0.3) \cdot q_{kast} \cdot Lb$ $v_{t.k} = 2.211\ \frac{kN}{m}$ (trykk)

$v_{s.k} := (-0.7 - 0.2) \cdot q_{kast} \cdot Lb$ $v_{s.k} = -3.98\ \frac{kN}{m}$ (sug)

BRUDDGRENSEKONTROLL

[2] Tab. NA.A1.1 $\psi_{0.s} := 0.7$ $\psi_{1.s} := 0.5$ $\psi_{2.s} := 0.2$ (Snølast)

$\psi_{0.v} := 0.6$ $\psi_{1.v} := 0.2$ $\psi_{2.v} := 0$ (Vindlast)

[1] (3.2) Høydefaktor: $k_h := min\left(\left(\frac{600\ mm}{h}\right)^{0.1}, 1.1\right)$ $k_h = 1.019$

Lastkombinasjoner

[1] Tab. 3.1 1. Kun egenlast $k_{mod1} := 0.6$

$p_{d1} := 1.35 \cdot g_k$ $p_{d1} = 5.172\ \frac{kN}{m}$

[1] Tab. 3.1 2. Egenlast og snølast $k_{mod2} := 0.9$

a) $p_{d2} := 1.35 \cdot g_k + 1.05 \cdot s_k$ $p_{d2} = 10.716\ \frac{kN}{m}$

b) $p_{d3} := 1.2 \cdot g_k + 1.5 \cdot s_k$ $p_{d3} = 12.518\ \frac{kN}{m}$

[1] Tab. 3.1 3. Egenlast, snølast og vindlast $k_{mod3} := 1.1$

a) $p_{d4} := 1.35 \cdot g_k + 1.05 \cdot s_k + 1.05 \cdot v_{t.k}$ $p_{d4} = 13.038 \frac{kN}{m}$

b) $p_{d5} := 1.2 \cdot g_k + 1.5 \cdot s_k + 1.05 \cdot v_{t.k}$ $p_{d5} = 14.839 \frac{kN}{m}$

Dimensjonerende fastheter

$f_{m.d} := f_{m.k} \cdot \frac{k_h \cdot k_{mod2}}{\gamma_M}$ $f_{m.d} = 23.934 \frac{N}{mm^2}$

$f_{c.90.d} := f_{c.90k} \cdot \frac{k_{mod2}}{\gamma_M}$ $f_{c.90.d} = 1.957 \frac{N}{mm^2}$

$f_{v.d} := f_{v.k} \cdot \frac{k_{mod2}}{\gamma_M}$ $f_{v.d} = 2.739 \frac{N}{mm^2}$

Dimensjonerende krefter

Benytter pd3 da pd5 er ca 18.5% høyere, samtidig som kapasiteten øker med 20%

$M_{Ed} := \frac{p_{d3} \cdot L^2}{8}$ $M_{Ed} = 83.384\ kN \cdot m$

$V_{Ed} := \frac{p_{d3} \cdot L}{2}$ $V_{Ed} = 45.69\ kN$

[1] 6.3.3 **Vipping**

Forutsetter at bjelken er gaffellagret (sikret mot rotasjon) om egen akse ved oppleggene. Videre antar en at takterrassen vil sikre mot forskyvninger på tvers. $k_{crit} := 1.0$. Om bjelken ikke var sikret mot vipping mellom oppleggene:

[1] Tab. 6.1 $l_{ef} := 0.9 \cdot 7300\ mm + 2 \cdot h$ $l_{ef} = 7.56\ m$

$\sigma_{m.crit} := \frac{0.78 \cdot b^2}{h \cdot l_{ef}} \cdot E_{0.05}$ $\sigma_{m.crit} = 104.056 \frac{N}{mm^2}$

$$\lambda_{relm} := \sqrt{\frac{f_{m.k}}{\sigma_{m.crit}}} \qquad \lambda_{relm} = 0.537$$

$$\lambda_{relm} \le 0.75 \qquad k_{crit} := 1.0$$

[1] 6.1.6 **Bøyekontroll**

$$W := \frac{\left(b \cdot h^2\right)}{6} \qquad W = \left(8.78 \cdot 10^6\right)\ mm^3$$

$$\sigma_{m.d} := \frac{M_{Ed}}{W} \qquad \sigma_{m.d} = 9.497\ \frac{N}{mm^2}$$

$$\frac{\sigma_{m.d}}{k_{crit} \cdot f_{m.d}} = 0.397$$ < 1.0 OK

[1] 6.1.7 **Skjærkontroll**

Sprekkfaktor for limtre: $k_{cr} := 0.67$

$$b_{ef} := k_{cr} \cdot b$$

$$\tau_d := \frac{\left(3 \cdot V_{Ed}\right)}{2 \cdot b_{ef} \cdot h} \qquad \tau_d = 0.961\ \frac{N}{mm^2}$$

$$\frac{\tau_d}{f_{v.d}} = 0.351$$ < 1.0 OK

[1] 6.1.5 **Trykk vinkelrett på fiberretning**

Søyledimensjon: $b_s := 215\ mm$ $h_s := 225\ mm$

$$F_{c.90.d} := V_{Ed}$$

$$A_{ef} := b_s \cdot h_s$$

$$\sigma_{c.90.d} := \frac{F_{c.90.d}}{A_{ef}} \qquad \sigma_{c.90.d} = 0.944\ \frac{N}{mm^2}$$

$k_{c.90} := 1.75$ (Hviler på enkeltopplegg, $l_1 \geq 2 \cdot h$, limtre og $l \leq 400\ mm$)

$\frac{\sigma_{c.90.d}}{k_{c.90} \cdot f_{c.90.d}} = 0.276$ < 1.0 OK

BRUKSGRENSEKONTROLL

Nedbøyning

[1] Tab 3.2 $k_{def} := 0.6$ (Klimaklasse 1) $I := \frac{1}{12} \cdot b \cdot h^3$

[1] Tab 7.2 Øyeblikkelig nedbøyning

$\delta_g := \frac{5}{384} \cdot \frac{g_k \cdot L^4}{E_{mean} \cdot I}$ $\delta_g = 5.015\ mm$

$\delta_s := \frac{5}{384} \cdot \frac{s_k \cdot L^4}{E_{mean} \cdot I}$ $\delta_s = 6.911\ mm$

$\delta_v := \frac{5}{384} \cdot \frac{v_{t.k} \cdot L^4}{E_{mean} \cdot I}$ $\delta_v = 2.894\ mm$

$w_{inst} := \delta_g + \delta_s + \delta_v$ $w_{inst} = 14.82\ mm$ < $\frac{L}{450} = 16.222\ mm$

Langtidsnedbøyning

$\delta_{gfin} := \delta_g \cdot (1 + k_{def})$ $\delta_{gfin} = 8.024\ mm$

$\delta_{sfin} := \delta_s \cdot (1 + \psi_{2.s} \cdot k_{def})$ $\delta_{sfin} = 7.74\ mm$

$\delta_{vfin} := \delta_v \cdot (\psi_{0.v} + \psi_{2.v} \cdot k_{def})$ $\delta_{vfin} = 1.736\ mm$

$w_{fin} := \delta_{gfin} + \delta_{sfin} + \delta_{vfin}$ $w_{fin} = 17.501\ mm$ < $\frac{L}{300} = 24.333\ mm$

BRANNDIMENSJONERING

[4] Tab 3.1 Dimensjonerende forkullingshastighet for limtre: $\beta_n := 0.7 \frac{mm}{min}$

[5] § 11-3 Brannklasse 2

[5] § 11-4 Brannmotstandstid R60: $t := 60\ min$

[4] 4.2.2(5) $k_{mod.fi} := 1.0$

[4] 2.3 (Merknad 2) $\gamma_{M.fi} := 1.0$

[4] 4.2.2 **Effektiv forkullingsdybde**

$k_0 := 1$ t > 20 min

$d_{char.n} := \beta_n \cdot t$

$d_0 := 7\ mm$

$d_{ef} := d_{char.n} + k_0 \cdot d_0$ $d_{ef} = 49\ mm$

Dimensjon effektivt resttverrsnitt

$b_{ef.fi} := b - 2 \cdot d_{ef}$ $b_{ef.fi} = 117\ mm$

$h_{ef.fi} := h - 1 \cdot d_{ef}$ $h_{ef.fi} = 446\ mm$

Dimensjonerende fastheter

$f_{m.d.fi} := 1.15 \cdot f_{m.k}$ $f_{m.d.fi} = 34.5 \frac{N}{mm^2}$

$f_{v.d.fi} := 1.15 \cdot f_{v.k}$ $f_{v.d.fi} = 4.025 \frac{N}{mm^2}$

Dimensjonerende krefter

(Verdier hentet fra Calculatis)

$M_{d.maks.fi} := 40.1\ kN \cdot m$

$V_{d.maks.fi} := 21.97\ kN$

[1] 6.3.3 **Vipping**

$\sigma_{m.crit.fi} := \frac{\left(0.78 \cdot b_{ef.fi}^{\ 2}\right)}{h_{ef.fi} \cdot l_{ef}} \cdot E_{0.05}$ $\sigma_{m.crit.fi} = 34.201\ \frac{N}{mm^2}$

$\lambda_{relm.fi} := \sqrt{\frac{f_{m.k}}{\sigma_{m.crit.fi}}}$ $\lambda_{relm.fi} = 0.937$ $0.75 < \lambda_{relm.fi} < 1.4$

$k_{crit.fi} := 1.56 - 0.75 \cdot \lambda_{relm.fi}$ $k_{crit.fi} = 0.858$

[1] 6.1.6 **Bøyekontroll**

$W_{fi} := \frac{\left(b_{ef.fi} \cdot h_{ef.fi}^{\ 2}\right)}{6}$ $W_{fi} = \left(3.879 \cdot 10^6\right)\ mm^3$

$\sigma_{m.d.fi} := \frac{M_{d.maks.fi}}{W_{fi}}$ $\sigma_{m.d.fi} = 10.338\ \frac{N}{mm^2}$

$\frac{\sigma_{m.d.fi}}{k_{crit.fi} \cdot f_{m.d.fi}} = 0.349$ < 1.0 OK

[1] 6.1.7 **Skjærkontroll**

$b_{ef} := k_{cr} \cdot b_{ef.fi}$

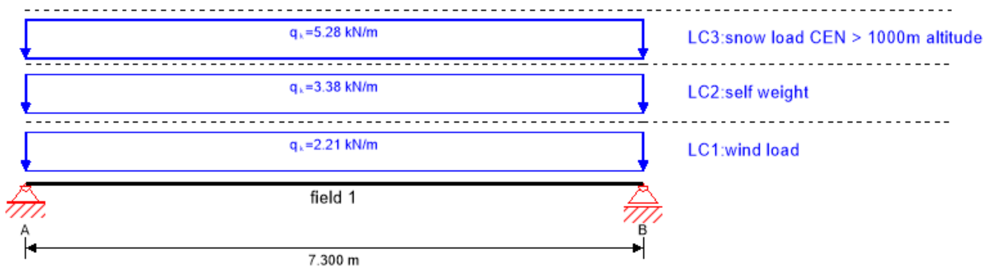
$\tau_{d.fi} := \frac{\left(3 \cdot V_{d.maks.fi}\right)}{2 \cdot b_{ef} \cdot h_{ef.fi}}$

$$\frac{\tau_{d.fi}}{f_{v.d.fi}} = 0.234$$ < 1 OK

Kan konkludere med at bjelken beholder tilstrekkelig bæreevne ved en 60 minutters brann.

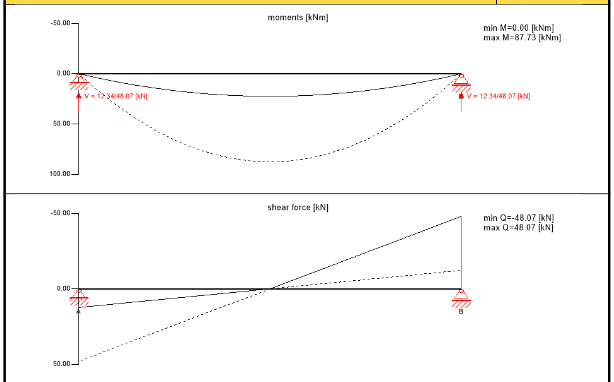
VEDLEGG F1.2

system



section: wooden beam 21.5/49.5; **material**: GL30c; **service class**: service class 1; **fire resistance class**: R 60

utilization — 90 %



flexural stress analysis — 43 %

$M_{y,d}$ =	87.73	kNm	$f_{m,k}$ =	30.00	N/mm²
$N_{t,d}$ =	0.00	kN	$f_{t,k}$ =	19.50	N/mm²
$\sigma_{t,d}$ =	0.00	N/mm²	$f_{t,d}$ =	15.26	N/mm²
$\sigma_{m,y,d}$ =	9.99	N/mm² <	$f_{m,y,d}$ =	23.48	N/mm² ✓

shear stress analysis — 32 %

V_d =	41.55	kN	$f_{v,k}$ =	3.50	N/mm²
$\tau_{v,d}$ =	0.59	N/mm² <	$f_{v,d}$ =	1.84	N/mm² ✓

lateral torsional buckling analysis — 43 %

$M_{y,d}$ =	87.73	kNm	$f_{m,k}$ =	30.00	N/mm²
$N_{c,d}$ =	0.00	kN	$f_{c,k}$ =	24.50	N/mm²
$\sigma_{c,d}$ =	0.00	N/mm²	$f_{c,d}$ =	19.17	N/mm²
$\sigma_{m,y,d}$ =	9.99	N/mm² <	$f_{m,y,d}$ =	23.48	N/mm² ✓

flexural stress analysis fire — 30 %

$M_{y,d}$ =	40.10	kNm	$f_{m,k}$ =	30.00	N/mm²
$N_{t,d}$ =	0.00	kN	$f_{t,k}$ =	19.50	N/mm²
$\sigma_{t,d}$ =	0.00	N/mm²	$f_{t,d}$ =	22.43	N/mm²
$\sigma_{m,y,d}$ =	10.34	N/mm² <	$f_{m,y,d}$ =	34.50	N/mm² ✓

shear stress analysis fire — 21 %

V_d =	19.29	kN	$f_{v,k}$ =	3.50	N/mm²
$\tau_{v,d}$ =	0.55	N/mm² <	$f_{v,d}$ =	2.70	N/mm² ✓

lateral torsional buckling analysis fire — 38 %

$M_{y,d}$ =	40.10	kNm	$f_{m,k}$ =	30.00	N/mm²
$N_{c,d}$ =	0.00	kN	$f_{c,k}$ =	24.50	N/mm²
$\sigma_{c,d}$ =	0.00	N/mm²	$f_{c,d}$ =	28.18	N/mm²
$\sigma_{m,y,d}$ =	10.34	N/mm² <	$f_{m,y,d}$ =	34.50	N/mm² ✓

w_{inst} = w[char]

field	K_{def}	limit	w_{limit}	$w_{calc.}$	ratio
		[-]	[mm]	[mm]	
1	0.6	L/500	14.6	13.1	90 %

w_{fin} = w[char] + w[q.p.]*kdef

field	K_{def}	limit	w_{limit}	$w_{calc.}$	ratio
		[-]	[mm]	[mm]	
1	0.6	L/300	24.3	16.6	68 %

$w_{net,fin}$ = w[q.p.] + w[q.p.]*kdef

field	K_{def}	limit	w_{limit}	$w_{calc.}$	ratio
		[-]	[mm]	[mm]	
1	0.6	L/350 L/350	20.9	9.3	45 %

support reaction

load case category	k_{mod}	A_V	B_V
		[kN]	
wind load	0.9	8.07	8.07
		0.00	0.00
self weight	0.6	12.34	12.34
		12.34	12.34
snow load CEN > 1000m altitude	0.8	19.27	19.27
		0.00	0.00

Disclaimer

The software was created to assist engineers in their daily business. The software is an engineering software that is dealing with a very complex matter of structural analysis and building physics analysis. Therefore, this software shall only be operated by skilled, experienced engineers, with a deep understanding of structural engineering and building physics related to timber structures. The user of the software is obliged to check all input values, no matter if they were given by the user or given by default by the software and all results for plausibility.
The use of the results of the software should not be relied upon as the basis for any decision or action. Any use of results of the software is only allowed, if the results have been verified and approved regarding completeness and correctness by a project structural/building physics engineer. The user has the possibility to make print-outs from the software. Any modification of those are not allowed.
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VEDLEGG F2.1

BJELKE 4-3

Referanse til standarder

[1] NS-EN 1995-1-1: Allmenne regler og regler for bygninger
[2] NS-EN 1990: Grunnlag for prosjektering av konstruksjoner
[3] Norsk Limtreprodusenters Forening, *Limtreboka - Beregningseksempler* (2018)
[4] NS-EN 1995-1-2: Brannteknisk dimensjonering
[5] Byggeteknisk forskrift (TEK17)

FORUTSETNINGER OG ANTAKELSER

[1] NA.901 Klimaklasse: 1

[1] Tab. 2.1 Lastvarighetsklasse for snølast: Korttidslast
Lastvarighetsklasse for vindlasten: Øyeblikkslast

[1] NA.2.3 Partialfaktor for limtre: $\gamma_M := 1.15$

Lengde: $L := 7300\ mm$

Bjelketverrrsnitt: $b := 215\ mm$ $h := 495\ mm$

Lastbredde: $Lb := 3100\ mm$

Karakteristisk vindlast: $q_{kast} := 1.34\ \frac{kN}{m^2}$

[3] Tab.6 Limtre GL30c:

$\rho_m := 430\ \frac{kg}{m^3}$ $g := 9.81\ \frac{m}{s^2}$ $f_{m.k} := 30\ \frac{N}{mm^2}$

$E_{0.05} := 10800\ \frac{N}{mm^2}$ $f_{c.90k} := 2.5\ \frac{N}{mm^2}$ $f_{v.k} := 3.5\ \frac{N}{mm^2}$

$E_{mean} := 13000\ \frac{N}{mm^2}$

LASTER

Egenlast tak: $g_{d.k} := 1.025\ \frac{kN}{m^2} \cdot Lb$ $g_{d.k} = 3.178\ \frac{kN}{m}$

Egenlast bjelke: $g_{b.k} := b \cdot h \cdot \rho_m \cdot g$ $g_{b.k} = 0.449\ \frac{kN}{m}$

Permanent last: $g_k := g_{d.k} + g_{b.k}$ $g_k = 3.626 \frac{kN}{m}$

Snølast: $s_k := 1.6 \frac{kN}{m^2} \cdot Lb$ $s_k = 4.96 \frac{kN}{m}$

Vindlast: $v_{t.k} := (0.2 + 0.3) \cdot q_{kast} \cdot Lb$ $v_{t.k} = 2.077 \frac{kN}{m}$ (trykk)

$v_{s.k} := (-0.7 - 0.2) \cdot q_{kast} \cdot Lb$ $v_{s.k} = -3.739 \frac{kN}{m}$ (sug)

BRUDDGRENSEKONTROLL

[2] Tab. NA.A1.1 $\psi_{0.s} := 0.7$ $\psi_{1.s} := 0.5$ $\psi_{2.s} := 0.2$ (Snølast)

$\psi_{0.v} := 0.6$ $\psi_{1.v} := 0.2$ $\psi_{2.v} := 0$ (Vindlast)

[1] (3.2) Høydefaktor: $k_h := min\left(\left(\frac{600\ mm}{h}\right)^{0.1}, 1.1\right)$ $k_h = 1.019$

Lastkombinasjoner

[1] Tab. 3.1 1. Kun egenlast $k_{mod1} := 0.6$

$p_{d1} := 1.35 \cdot g_k$ $p_{d1} = 4.896 \frac{kN}{m}$

[1] Tab. 3.1 2. Egenlast og snølast $k_{mod2} := 0.9$

a) $p_{d2} := 1.35 \cdot g_k + 1.05 \cdot s_k$ $p_{d2} = 10.104 \frac{kN}{m}$

b) $p_{d3} := 1.2 \cdot g_k + 1.5 \cdot s_k$ $p_{d3} = 11.792 \frac{kN}{m}$

[1] Tab. 3.1 3. Egenlast, snølast og vindlast $k_{mod3} := 1.1$

a) $p_{d4} := 1.35 \cdot g_k + 1.05 \cdot s_k + 1.05 \cdot v_{t.k}$ $p_{d4} = 12.285 \frac{kN}{m}$

b) $p_{d5} := 1.2 \cdot g_k + 1.5 \cdot s_k + 1.05 \cdot v_{t.k}$ $p_{d5} = 13.973 \frac{kN}{m}$

Dimensjonerende fastheter

$f_{m.d} := f_{m.k} \cdot \frac{k_h \cdot k_{mod2}}{\gamma_M}$ $f_{m.d} = 23.934 \frac{N}{mm^2}$

$f_{c.90.d} := f_{c.90k} \cdot \frac{k_{mod2}}{\gamma_M}$ $f_{c.90.d} = 1.957 \frac{N}{mm^2}$

$f_{v.d} := f_{v.k} \cdot \frac{k_{mod2}}{\gamma_M}$ $f_{v.d} = 2.739 \frac{N}{mm^2}$

Dimensjonerende krefter

Benytter pd3 da pd5 er ca 18.5% høyere, samtidig som kapasiteten øker med 20%

$M_{Ed} := \frac{p_{d3} \cdot L^2}{8}$ $M_{Ed} = 78.548 \ kN \cdot m$

$V_{Ed} := \frac{p_{d3} \cdot L}{2}$ $V_{Ed} = 43.04 \ kN$

[1] 6.3.3 **Vipping**

Forutsetter at bjelken er gaffellagret (sikret mot rotasjon) om egen akse ved oppleggene. Videre antar en at takterrassen vil sikre mot forskyvninger på tvers. $k_{crit} := 1.0$. Om bjelken ikke var sikret mot vipping mellom oppleggene:

[1] Tab. 6.1 $l_{ef} := 0.9 \cdot 7300 \ mm + 2 \cdot h$ $l_{ef} = 7.56 \ m$

$\sigma_{m.crit} := \frac{0.78 \cdot b^2}{h \cdot l_{ef}} \cdot E_{0.05}$ $\sigma_{m.crit} = 104.056 \frac{N}{mm^2}$

$\lambda_{relm} := \sqrt{\dfrac{f_{m.k}}{\sigma_{m.crit}}}$ $\lambda_{relm} = 0.537$

$\lambda_{relm} \le 0.75$ $k_{crit} := 1.0$

[1] 6.1.6 **Bøyekontroll**

$W := \dfrac{\left(b \cdot h^2\right)}{6}$ $W = \left(8.78 \cdot 10^6\right)\ mm^3$

$\sigma_{m.d} := \dfrac{M_{Ed}}{W}$ $\sigma_{m.d} = 8.946\ \dfrac{N}{mm^2}$

$\dfrac{\sigma_{m.d}}{k_{crit} \cdot f_{m.d}} = 0.374$ < 1.0 OK

[1] 6.1.7 **Skjærkontroll**

Sprekkfaktor for limtre: $k_{cr} := 0.67$

$b_{ef} := k_{cr} \cdot b$

$\tau_d := \dfrac{\left(3 \cdot V_{Ed}\right)}{2 \cdot b_{ef} \cdot h}$ $\tau_d = 0.905\ \dfrac{N}{mm^2}$

$\dfrac{\tau_d}{f_{v.d}} = 0.331$ < 1.0 OK

[1] 6.1.5 **Trykk vinkelrett på fiberretning**

Søyledimensjon: $b_s := 215\ mm$ $h_s := 225\ mm$

$F_{c.90.d} := V_{Ed}$

$A_{ef} := b_s \cdot h_s$

$\sigma_{c.90.d} := \dfrac{F_{c.90.d}}{A_{ef}}$ $\sigma_{c.90.d} = 0.89\ \dfrac{N}{mm^2}$

$k_{c.90} := 1.75$ (Hviler på enkeltopplegg, $l_1 \geq 2 \cdot h$, limtre og $l \leq 400\ mm$)

$\frac{\sigma_{c.90.d}}{k_{c.90} \cdot f_{c.90.d}} = 0.26$ < 1.0 OK

BRUKSGRENSEKONTROLL

Nedbøyning

[1] Tab 3.2 $k_{def} := 0.6$ (Klimaklasse 1) $I := \frac{1}{12} \cdot b \cdot h^3$

[1] Tab 7.2 Øyeblikkelig nedbøyning

$\delta_g := \frac{5}{384} \cdot \frac{g_k \cdot L^4}{E_{mean} \cdot I}$ $\delta_g = 4.747\ mm$

$\delta_s := \frac{5}{384} \cdot \frac{s_k \cdot L^4}{E_{mean} \cdot I}$ $\delta_s = 6.492\ mm$

$\delta_v := \frac{5}{384} \cdot \frac{v_{t.k} \cdot L^4}{E_{mean} \cdot I}$ $\delta_v = 2.719\ mm$

$w_{inst} := \delta_g + \delta_s + \delta_v$ $w_{inst} = 13.958\ mm$ < $\frac{L}{500} = 14.6\ mm$

Langtidsnedbøyning

$\delta_{gfin} := \delta_g \cdot (1 + k_{def})$ $\delta_{gfin} = 7.595\ mm$

$\delta_{sfin} := \delta_s \cdot (1 + \psi_{2.s} \cdot k_{def})$ $\delta_{sfin} = 7.271\ mm$

$\delta_{vfin} := \delta_v \cdot (\psi_{0.v} + \psi_{2.v} \cdot k_{def})$ $\delta_{vfin} = 1.631\ mm$

$w_{fin} := \delta_{gfin} + \delta_{sfin} + \delta_{vfin}$ $w_{fin} = 16.497\ mm$ < $\frac{L}{300} = 24.333\ mm$

BRANNDIMENSJONERING

[4] Tab 3.1 Dimensjonerende forkullingshastighet for limtre: $\beta_n := 0.7 \frac{mm}{min}$

[5] § 11-3 Brannklasse 2

[5] § 11-4 Brannmotstandstid R60: $t := 60\ min$

[4] 4.2.2(5) $k_{mod.fi} := 1.0$

[4] 2.3 (Merknad 2) $\gamma_{M.fi} := 1.0$

[4] 4.2.2 **Effektiv forkullingsdybde**

$k_0 := 1$ t > 20 min

$d_{char.n} := \beta_n \cdot t$

$d_0 := 7\ mm$

$d_{ef} := d_{char.n} + k_0 \cdot d_0$ $d_{ef} = 49\ mm$

Dimensjon effektivt resttverrsnitt

$b_{ef.fi} := b - 2 \cdot d_{ef}$ $b_{ef.fi} = 117\ mm$

$h_{ef.fi} := h - 1 \cdot d_{ef}$ $h_{ef.fi} = 446\ mm$

Dimensjonerende fastheter

$f_{m.d.fi} := 1.15 \cdot f_{m.k}$ $f_{m.d.fi} = 34.5 \frac{N}{mm^2}$

$f_{v.d.fi} := 1.15 \cdot f_{v.k}$ $f_{v.d.fi} = 4.025 \frac{N}{mm^2}$

Dimensjonerende krefter

(Verdier hentet fra Calculatis)

$M_{d.maks.fi} := 40.77\ kN \cdot m$

$V_{d.maks.fi} := 22.34\ kN$

[1] 6.3.3 **Vipping**

$\sigma_{m.crit.fi} := \frac{\left(0.78 \cdot b_{ef.fi}^{\ 2}\right)}{h_{ef.fi} \cdot l_{ef}} \cdot E_{0.05}$ $\sigma_{m.crit.fi} = 34.201\ \frac{N}{mm^2}$

$\lambda_{relm.fi} := \sqrt{\frac{f_{m.k}}{\sigma_{m.crit.fi}}}$ $\lambda_{relm.fi} = 0.937$ $0.75 < \lambda_{relm.fi} < 1.4$

$k_{crit.fi} := 1.56 - 0.75 \cdot \lambda_{relm.fi}$ $k_{crit.fi} = 0.858$

[1] 6.1.6 **Bøyekontroll**

$W_{fi} := \frac{\left(b_{ef.fi} \cdot h_{ef.fi}^{\ 2}\right)}{6}$ $W_{fi} = \left(3.879 \cdot 10^6\right)\ mm^3$

$\sigma_{m.d.fi} := \frac{M_{d.maks.fi}}{W_{fi}}$ $\sigma_{m.d.fi} = 10.511\ \frac{N}{mm^2}$

$\frac{\sigma_{m.d.fi}}{k_{crit.fi} \cdot f_{m.d.fi}} = 0.355$ < 1.0 OK

[1] 6.1.7 **Skjærkontroll**

$b_{ef} := k_{cr} \cdot b_{ef.fi}$

$\tau_{d.fi} := \frac{\left(3 \cdot V_{d.maks.fi}\right)}{2 \cdot b_{ef} \cdot h_{ef.fi}}$

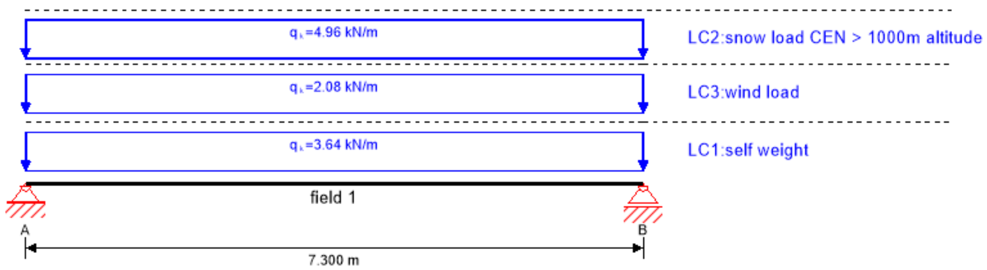
$$\frac{\tau_{d.fi}}{f_{v.d.fi}} = 0.238 \qquad < 1 \qquad \text{OK}$$

Kan konkludere med at bjelken beholder tilstrekkelig bæreevne ved en 60 minutters brann.

VEDLEGG F2.2

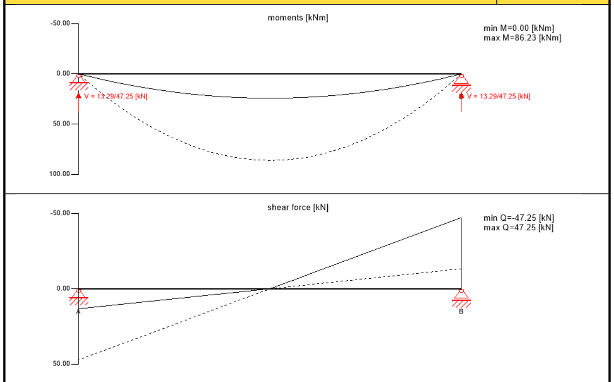
	project	BYGG C		page	1
Marthe Elise Leirvåg	element	Bjelke 4-3		date	24.05.2020

system



section: wooden beam 21.5/49.5; **material**: GL30c; **service class**: service class 2; **fire resistance class**: R 60

utilization — 88 %



flexural stress analysis — 42 %

$M_{y,d}$ =	86.23	kNm	$f_{m,k}$ =	30.00	N/mm²
$N_{t,d}$ =	0.00	kN	$f_{t,k}$ =	19.50	N/mm²
$\sigma_{t,d}$ =	0.00	N/mm²	$f_{t,d}$ =	15.26	N/mm²
$\sigma_{m,y,d}$ =	9.82	N/mm² <	$f_{m,y,d}$ =	23.48	N/mm² ✓

shear stress analysis — 31 %

V_d =	40.84	kN	$f_{v,k}$ =	3.50	N/mm²
$\tau_{v,d}$ =	0.58	N/mm² <	$f_{v,d}$ =	1.84	N/mm² ✓

lateral torsional buckling analysis — 42 %

$M_{y,d}$ =	86.23	kNm	$f_{m,k}$ =	30.00	N/mm²
$N_{c,d}$ =	0.00	kN	$f_{c,k}$ =	24.50	N/mm²
$\sigma_{c,d}$ =	0.00	N/mm²	$f_{c,d}$ =	19.17	N/mm²
$\sigma_{m,y,d}$ =	9.82	N/mm² <	$f_{m,y,d}$ =	23.48	N/mm² ✓

flexural stress analysis fire — 30 %

$M_{y,d}$ =	40.77	kNm	$f_{m,k}$ =	30.00	N/mm²
$N_{t,d}$ =	0.00	kN	$f_{t,k}$ =	19.50	N/mm²
$\sigma_{t,d}$ =	0.00	N/mm²	$f_{t,d}$ =	22.43	N/mm²
$\sigma_{m,y,d}$ =	10.51	N/mm² <	$f_{m,y,d}$ =	34.50	N/mm² ✓

shear stress analysis fire — 21 %

V_d =	19.61	kN	$f_{v,k}$ =	3.50	N/mm²
$\tau_{v,d}$ =	0.56	N/mm² <	$f_{v,d}$ =	2.70	N/mm² ✓

lateral torsional buckling analysis fire — 39 %

$M_{y,d}$ =	40.77	kNm	$f_{m,k}$ =	30.00	N/mm²
$N_{c,d}$ =	0.00	kN	$f_{c,k}$ =	24.50	N/mm²
$\sigma_{c,d}$ =	0.00	N/mm²	$f_{c,d}$ =	28.18	N/mm²
$\sigma_{m,y,d}$ =	10.51	N/mm² <	$f_{m,y,d}$ =	34.50	N/mm² ✓

w_{inst} = w[char]

field	K_{def}	limit	w_{limit}	$w_{calc.}$	ratio
		[-]	[mm]	[mm]	
1	0.8	L/500	14.6	12.9	88 %

w_{fin} = w[char] + w[q.p.]*kdef

field	K_{def}	limit	w_{limit}	$w_{calc.}$	ratio
		[-]	[mm]	[mm]	
1	0.8	L/300	24.3	17.7	73 %

$w_{net,fin}$ = w[q.p.] + w[q.p.]*kdef

field	K_{def}	limit	w_{limit}	$w_{calc.}$	ratio
		[-]	[mm]	[mm]	
1	0.8	L/350 L/350	20.9	10.9	52 %

support reaction

load case category	k_{mod}	A_V	B_V
		[kN]	
self weight	0.6	13.29	13.29
		13.29	13.29
snow load CEN > 1000m altitude	0.8	18.10	18.10
		0.00	0.00
wind load	0.9	7.59	7.59
		0.00	0.00

Disclaimer

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The use of the results of the software should not be relied upon as the basis for any decision or action. Any use of results of the software is only allowed, if the results have been verified and approved regarding completeness and correctness by a project structural/building physics engineer. The user has the possibility to make print-outs from the software. Any modification of those are not allowed.
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VEDLEGG F3.1

BJELKE 4-2

Referanse til standarder

[1] NS-EN 1995-1-1: Allmenne regler og regler for bygninger
[2] NS-EN 1990: Grunnlag for prosjektering av konstruksjoner
[3] Norsk Limtreprodusenters Forening, *Limtreboka - Beregningseksempler* (2018)
[4] NS-EN 1995-1-2: Brannteknisk dimensjonering
[5] Byggeteknisk forskrift (TEK17)
[6] SINTEF Byggforskserien, nr.421.051 *Statikkformler for bjelker* (2019)

FORUTSETNINGER OG ANTAKELSER

[1] NA.901 Klimaklasse: 1

[1] Tab. 2.1 Lastvarighetsklasse for snølast: Korttidslast
Lastvarighetsklasse for vindlasten: Øyeblikkslast

[1] NA.2.3 Partialfaktor for limtre: $\gamma_M := 1.15$

Lengde: $L := 13500\ mm$ $l_1 := 7300\ mm$ $l_2 := 5650\ mm$

Bjelketverrrsnitt: $b := 215\ mm$ $h := 495\ mm$

Lastbredde: $Lb := 3350\ mm$

Karakteristisk vindlast: $q_{kast} := 1.34\ \frac{kN}{m^2}$

[3] Tab. 6 Limtre GL30c:

$\rho_m := 430\ \frac{kg}{m^3}$ $g := 9.81\ \frac{m}{s^2}$ $f_{m.k} := 30\ \frac{N}{mm^2}$

$E_{0.05} := 10800\ \frac{N}{mm^2}$ $f_{c.90k} := 2.5\ \frac{N}{mm^2}$ $f_{v.k} := 3.5\ \frac{N}{mm^2}$

$E_{mean} := 13000\ \frac{N}{mm^2}$

LASTER

Egenlast tak: $g_{t.k} := 1.025\ \frac{kN}{m^2} \cdot Lb$ $g_{t.k} = 3.434\ \frac{kN}{m}$

Egenlast bjelke: $g_{b.k} := b \cdot h \cdot \rho_m \cdot g$ $g_{b.k} = 0.449\ \frac{kN}{m}$

Permanent last: $g_k := g_{t.k} + g_{b.k}$ $g_k = 3.883\ \frac{kN}{m}$

Snølast: $s_k := 1.6\ \frac{kN}{m^2} \cdot Lb$ $s_k = 5.36\ \frac{kN}{m}$

Vindlast: $v_{t.k} := (0.2 + 0.3) \cdot q_{kast} \cdot Lb$ $v_{t.k} = 2.245\ \frac{kN}{m}$ (trykk)

$v_{s.k} := (-0.7 - 0.2) \cdot q_{kast} \cdot Lb$ $v_{s.k} = -4.04\ \frac{kN}{m}$ (sug)

BRUDDGRENSEKONTROLL

[2] Tab. NA.A1.1

$\psi_{0.s} := 0.7$ $\psi_{1.s} := 0.5$ $\psi_{2.s} := 0.2$ (Snølast)

$\psi_{0.v} := 0.6$ $\psi_{1.v} := 0.2$ $\psi_{2.v} := 0$ (Vindlast)

[1] (3.2) Høydefaktor: $k_h := min\left(\left(\frac{600\ mm}{h}\right)^{0.1}, 1.1\right)$ $k_h = 1.019$

Lastkombinasjoner

[1] Tab. 3.1 1. Egenlast og snølast $k_{mod1} := 0.9$

$p_{d1} := 1.2 \cdot g_k + 1.5 \cdot s_k$ $p_{d1} = 12.699\ \frac{kN}{m}$

[1] Tab. 3.1 2. Alle laster $k_{mod2} := 1.1$

$p_{d2} := 1.2 \cdot g_k + 1.5 \cdot s_k + 1.5 \cdot \psi_{0.v} \cdot v_{t.k}$ $p_{d2} = 14.719\ \frac{kN}{m}$

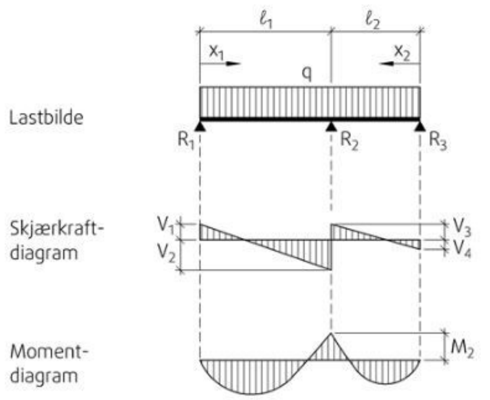
3. Vind på snøfritt tak (løftekrefter)

$p_{løft} := 1.0 \cdot g_k + 1.5 \cdot v_{s.k}$ $p_{løft} = -2.177\ \frac{kN}{m}$

Ved lastkombinasjon 2 vil dimensjonerende materialfastheter være ca. 20% høyere enn ved kombinasjon 1, grunnet høyere kmod. Samtidig er pd2 rundt 15% høyere enn pd1. Det vil gi relativt like verdier, men regner videre med lastkombinasjon 1 som dimensjonerende her.

[6] Nr. 81

Dimensjonerende krefter



$$M_2 := \frac{\left(p_{d1} \cdot l_2^{\ 3} + p_{d1} \cdot l_1^{\ 3}\right)}{8 \cdot \left(l_1 + l_2\right)}$$

$M_2 = 69.794\ kN \cdot m$

$R_1 := \frac{-M_2}{l_1} + \frac{p_{d1} \cdot l_1}{2}$ $\quad R_1 = 36.791\ kN$

$R_3 := \frac{-M_2}{l_2} + \frac{p_{d1} \cdot l_2}{2}$ $\quad R_3 = 23.522\ kN$

$R_2 := p_{d1} \cdot \left(l_1 + l_2\right) - R_1 - R_3$ $\quad R_2 = 104.141\ kN$

$V_1 := R_1$

$V_4 := R_3$

$V_2 := p_{d1} \cdot l_1 - R_1$ $\quad V_2 = 55.913\ kN$

$V_3 := p_{d1} \cdot l_2 - R_3$ $\quad V_3 = 48.228\ kN$

$x_1 := \frac{R_1}{p_{d1}}$ $\quad M_1 := R_1 \cdot x_1 - \frac{p_{d1} \cdot x_1^{\ 2}}{2}$ $\quad M_1 = 53.295\ kN \cdot m$

$x_2 := \frac{R_3}{p_{d1}}$ $\quad M_3 := R_1 \cdot x_2 - \frac{p_{d1} \cdot x_2^{\ 2}}{2}$ $\quad M_3 = 46.363\ kN \cdot m$

$$V_{Ed} := \max(V_1, V_2, V_3, V_4) \qquad V_{Ed} = 55.913\ kN$$

$$M_{Ed} := \max(M_1, M_2, M_3) \qquad M_{Ed} = 69.794\ kN \cdot m$$

Dimensjonerende fastheter

$$f_{m.d} := f_{m.k} \cdot \frac{k_h \cdot k_{mod1}}{\gamma_M} \qquad f_{m.d} = 23.934\ \frac{N}{mm^2}$$

$$f_{c.90.d} := f_{c.90k} \cdot \frac{k_{mod1}}{\gamma_M} \qquad f_{c.90.d} = 1.957\ \frac{N}{mm^2}$$

$$f_{v.d} := f_{v.k} \cdot \frac{k_{mod1}}{\gamma_M} \qquad f_{v.d} = 2.739\ \frac{N}{mm^2}$$

[1] 6.3.3 **Vipping**

Forutsetter at bjelken er gaffellagret (sikret mot rotasjon) om egen akse ved oppleggene. Videre antar en at taket vil sikre mot forskyvninger på tvers. En mulighet for vipping er over midtopplegg med trykk i underkant, regner konservativt den som en konsentrert last.

[1] Tab. 6.1 $l_{ef} := 0.8 \cdot 7300\ mm + 2 \cdot h \qquad l_{ef} = 6.83\ m$

$$\sigma_{m.crit} := \frac{\left(0.78 \cdot b^2\right)}{h \cdot l_{ef}} \cdot E_{0.05} \qquad \sigma_{m.crit} = 115.178\ \frac{N}{mm^2}$$

$$\lambda_{relm} := \sqrt{\frac{f_{m.k}}{\sigma_{m.crit}}} \qquad \lambda_{relm} = 0.51$$

$$\lambda_{relm} \le 0.75 \qquad k_{crit} := 1.0$$

[1] 6.1.6 **Bøyekontroll**

$$W := \frac{\left(b \cdot h^2\right)}{6} \qquad W = \left(8.78 \cdot 10^6\right)\ mm^3$$

$\sigma_{m.d} := \frac{M_{Ed}}{W}$ $\sigma_{m.d} = 7.949 \frac{N}{mm^2}$

$\frac{\sigma_{m.d}}{k_{crit} \cdot f_{m.d}} = 0.332$ < 1.0 OK

[1] 6.1.7 **Skjærkontroll**

Sprekkfaktor for limtre: $k_{cr} := 0.67$

$b_{ef} := k_{cr} \cdot b$

$\tau_d := \frac{(3 \cdot V_{Ed})}{2 \cdot b_{ef} \cdot h}$ $\tau_d = 1.176 \frac{N}{mm^2}$

$\frac{\tau_d}{f_{v.d}} = 0.429$ < 1.0 OK

[1] 6.1.5 **Trykk vinkelrett på fiberretning**

Trykk vinkelrett på fiberretningen blir ikke kontrollert i Calculatis

Søyledimensjon: $b_s := 215\ mm$ $h_s := 180\ mm$

Dimensjonerende opplagerkraft ved midtopplegg:

$F_{c.90.d} := R_2$

$A_{ef} := b_s \cdot (h_s + 60\ mm)$

$\sigma_{c.90.d} := \frac{F_{c.90.d}}{A_{ef}}$ $\sigma_{c.90.d} = 2.018 \frac{N}{mm^2}$

$k_{c.90} := 1.75$ (Hviler på enkeltopplegg, $l_1 \geq 2 \cdot h$, limtre og $l \leq 400\ mm$)

$$\frac{\sigma_{c.90.d}}{k_{c.90} \cdot f_{c.90.d}} = 0.589$$ < 1.0 OK

Dimensjonerende opplagerkraft ved endeopplegg:

$$F_{c.90.d.e} := R_1$$

$$A_{ef.e} := b_s \cdot (h_s + 30\ mm)$$

$$\sigma_{c.90.d.e} := \frac{F_{c.90.d.e}}{A_{ef.e}} \qquad \sigma_{c.90.d} = 2.018\ \frac{N}{mm^2}$$

$k_{c.90} := 1.75$ (Hviler på enkeltopplegg, $l_1 \geq 2 \cdot h$, limtre og $l \leq 400\ mm$)

$$\frac{\sigma_{c.90.d.e}}{k_{c.90} \cdot f_{c.90.d}} = 0.238$$ < 1.0 OK

Løftekrefter
Bjelken må festes slik at løftekreftene kan overføres til søylene og at kreftene føres videre ned til fundamentene.

$$q_{løft} := g_k \cdot 0.9 + v_{s.k} \cdot 1.5 \qquad q_{løft} = -2.566\ \frac{kN}{m}$$

BRUKSGRENSEKONTROLL

Nedbøyning

[1] Tab 3.2 $k_{def} := 0.6$ (Klimaklasse 1) $I := \frac{1}{12} \cdot b \cdot h^3$

Forenkler og benytter formel for en kontinuerlig bjelke med to like spenn med jevnt fordelt last. Setter lengden lik det lengste spennet. Kontrolleres i Calculatis, forventer da noe avvik.

[1] Tab 7.2 Øyeblikkelig nedbøyning

$\delta_g := \frac{1}{185} \cdot \frac{g_k \cdot l_1^{\ 4}}{E_{mean} \cdot I}$ $\delta_g = 2.11\ mm$

$\delta_s := \frac{1}{185} \cdot \frac{s_k \cdot l_1^{\ 4}}{E_{mean} \cdot I}$ $\delta_s = 2.913\ mm$

$\delta_v := \frac{1}{185} \cdot \frac{v_{t.k} \cdot l_1^{\ 4}}{E_{mean} \cdot I}$ $\delta_v = 1.22\ mm$

$w_{inst} := \delta_g + \delta_s + \delta_v$ $w_{inst} = 6.242\ mm$ < $\frac{l_1}{500} = 14.6\ mm$

Langtidsnedbøyning

$\delta_{gfin} := \delta_g \cdot (1 + k_{def})$ $\delta_{gfin} = 3.376\ mm$

$\delta_{sfin} := \delta_s \cdot (1 + \psi_{2.s} \cdot k_{def})$ $\delta_{sfin} = 3.262\ mm$

$\delta_{vfin} := \delta_v \cdot (\psi_{0.v} + \psi_{2.v} \cdot k_{def})$ $\delta_{vfin} = 0.732\ mm$

$w_{fin} := \delta_{gfin} + \delta_{sfin} + \delta_{vfin}$ $w_{fin} = 7.369\ mm$ < $\frac{l_1}{300} = 24.333\ mm$

BRANNDIMENSJONERING

[4] Tab 3.1 Dimensjonerende forkullingshastighet for limtre: $\beta_n := 0.7\ \frac{mm}{min}$

[5] § 11-3 Brannklasse 2

[5] § 11-4 Brannmotstandstid R60: $t := 60\ min$

[4] 4.2.2(5) $k_{mod.fi} := 1.0$

[4] 2.3 (Merknad 2) $\gamma_{M.fi} := 1.0$

[4] 4.2.2 **Effektiv forkullingsdybde**

$k_0 := 1$ t > 20 min

$d_{char.n} := \beta_n \cdot t$

$d_0 := 7\ mm$

$d_{ef} := d_{char.n} + k_0 \cdot d_0$ $d_{ef} = 49\ mm$

Dimensjon effektivt resttverrsnitt

$b_{ef.fi} := b - 2 \cdot d_{ef}$ $b_{ef.fi} = 117\ mm$

$h_{ef.fi} := h - 1 \cdot d_{ef}$ $h_{ef.fi} = 446\ mm$

Dimensjonerende fastheter

$f_{m.d.fi} := 1.15 \cdot f_{m.k}$ $f_{m.d.fi} = 34.5\ \frac{N}{mm^2}$

$f_{v.d.fi} := 1.15 \cdot f_{v.k}$ $f_{v.d.fi} = 4.025\ \frac{N}{mm^2}$

Dimensjonerende krefter

(Verdier hentet fra Calculatis, det er gjort overslagsberegning for kontroll)

$M_{d.maks.fi} := 36.08 \cdot kN \cdot m$

$V_{d.maks.fi} := 28.9\ kN$

[1] 6.3.3 **Vipping**

$\sigma_{m.crit.fi} := \frac{\left(0.78 \cdot b_{ef.fi}^{\ 2}\right)}{h_{ef.fi} \cdot l_{ef}} \cdot E_{0.05}$ $\sigma_{m.crit.fi} = 37.856\ \frac{N}{mm^2}$

$\lambda_{relm.fi} := \sqrt{\frac{f_{m.k}}{\sigma_{m.crit.fi}}}$ $\lambda_{relm.fi} = 0.89$ $0.75 < \lambda_{relm.fi} < 1.4$

$k_{crit.fi} := 1.56 - 0.75 \cdot \lambda_{relm.fi}$ $\qquad k_{crit.fi} = 0.892$

[1] 6.1.6 **Bøyekontroll**

$W_{fi} := \dfrac{\left(b_{ef.fi} \cdot h_{ef.fi}^{\ 2}\right)}{6}$ $\qquad W_{fi} = \left(3.879 \cdot 10^{6}\right)\ mm^{3}$

$\sigma_{m.d.fi} := \dfrac{M_{d.maks.fi}}{W_{fi}}$ $\qquad \sigma_{m.d.fi} = 9.302\ \dfrac{N}{mm^{2}}$

$\dfrac{\sigma_{m.d.fi}}{k_{crit.fi} \cdot f_{m.d.fi}} = 0.302$ < 1.0 OK

[1] 6.1.7 **Skjærkontroll**

$b_{ef} := k_{cr} \cdot b_{ef.fi}$

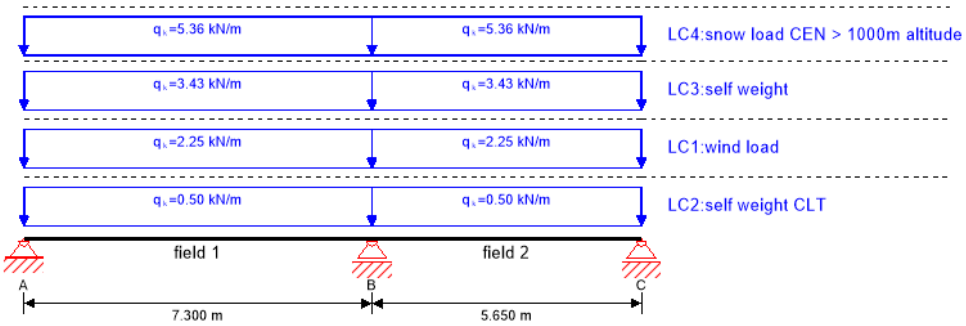
$\tau_{d.fi} := \dfrac{\left(3 \cdot V_{d.maks.fi}\right)}{2 \cdot b_{ef} \cdot h_{ef.fi}}$

$\dfrac{\tau_{d.fi}}{f_{v.d.fi}} = 0.308$ < 1 OK

Kan konkludere med at bjelken beholder tilstrekkelig bæreevne ved en 60 minutters brann.

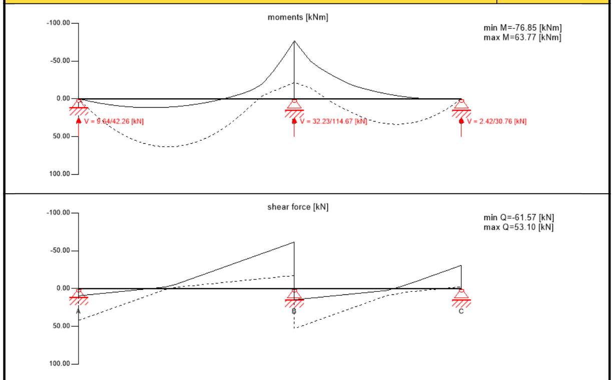
VEDLEGG F3.2

system



section: wooden beam 21.5/49.5; **material**: GL30c; **service class**: service class 1; **fire resistance class**: R 60

utilization 56 %



flexural stress analysis 37 %

$M_{y,d}$ =	-76.85	kNm	$f_{m,k}$ =	30.00	N/mm²
$N_{t,d}$ =	0.00	kN	$f_{t,k}$ =	19.50	N/mm²
$\sigma_{t,d}$ =	0.00	N/mm²	$f_{t,d}$ =	15.26	N/mm²
$\sigma_{m,y,d}$ =	8.75	N/mm² <	$f_{m,y,d}$ =	23.48	N/mm² ✓

shear stress analysis 42 %

V_d =	54.64	kN	$f_{v,k}$ =	3.50	N/mm²
$\tau_{v,d}$ =	0.77	N/mm² <	$f_{v,d}$ =	1.84	N/mm² ✓

lateral torsional buckling analysis 37 %

$M_{y,d}$ =	-76.85	kNm	$f_{m,k}$ =	30.00	N/mm²
$N_{c,d}$ =	0.00	kN	$f_{c,k}$ =	24.50	N/mm²
$\sigma_{c,d}$ =	0.00	N/mm²	$f_{c,d}$ =	19.17	N/mm²
$\sigma_{m,y,d}$ =	8.75	N/mm² <	$f_{m,y,d}$ =	23.48	N/mm² ✓

flexural stress analysis fire 27 %

$M_{y,d}$ =	-36.33	kNm	$f_{m,k}$ =	30.00	N/mm²
$N_{t,d}$ =	0.00	kN	$f_{t,k}$ =	19.50	N/mm²
$\sigma_{t,d}$ =	0.00	N/mm²	$f_{t,d}$ =	22.43	N/mm²
$\sigma_{m,y,d}$ =	9.37	N/mm² <	$f_{m,y,d}$ =	34.50	N/mm² ✓

shear stress analysis fire 28 %

V_d =	26.16	kN	$f_{v,k}$ =	3.50	N/mm²
$\tau_{v,d}$ =	0.75	N/mm² <	$f_{v,d}$ =	2.70	N/mm² ✓

lateral torsional buckling analysis fire 33 %

$M_{y,d}$ =	-36.33	kNm	$f_{m,k}$ =	30.00	N/mm²
$N_{c,d}$ =	0.00	kN	$f_{c,k}$ =	24.50	N/mm²
$\sigma_{c,d}$ =	0.00	N/mm²	$f_{c,d}$ =	28.18	N/mm²
$\sigma_{m,y,d}$ =	9.37	N/mm² <	$f_{m,y,d}$ =	34.50	N/mm² ✓

w_{inst} = w[char]

field	K_{def}	limit	w_{limit}	$w_{calc.}$	ratio
		[-]	[mm]	[mm]	
1	0.6	L/500	14.6	8.2	56 %
2	0.6	L/500	11.3	2.3	21 %

w_{fin} = w[char] + w[q.p.]*kdef

field	K_{def}	limit	w_{limit}	$w_{calc.}$	ratio
		[-]	[mm]	[mm]	
1	0.6	L/300	24.3	10.4	43 %
2	0.6	L/300	18.8	2.8	15 %

$w_{net,fin}$ = w[q.p.] + w[q.p.]*kdef

field	K_{def}	limit	w_{limit}	$w_{calc.}$	ratio
		[-]	[mm]	[mm]	
1	0.6	L/350 L/350	20.9	5.7	27 %
2	0.6	L/350 L/350	16.1	1.2	7 %

storaenso

support reaction				
load case category	k_{mod}	A_V	B_V	C_V
		[kN]		
wind load	0.9	6.50	18.41	4.16
		0.00	0.00	0.00
self weight CLT	0.6	1.45	4.10	0.93
		1.45	4.10	0.93
self weight	0.6	9.94	28.13	6.35
		9.94	28.13	6.35
snow load CEN > 1000m altitude	0.8	16.81	43.96	13.49
		-1.28	0.00	-3.56

Disclaimer

VEDLEGG F4.1

BJELKE 3-5

Referanse til standarder

[1] NS-EN 1995-1-1: Allmenne regler og regler for bygninger
[2] NS-EN 1990: Grunnlag for prosjektering av konstruksjoner
[3] Norsk Limtreprodusenters Forening, *Limtreboka - Beregningseksempler* (2018)
[4] NS-EN 1995-1-2: Brannteknisk dimensjonering
[5] Byggeteknisk forskrift (TEK17)
[6] SINTEF Byggforskserien, nr.421.051 *Statikkformler for bjelker* (2019)

FORUTSETNINGER OG ANTAKELSER

[1] NA.901 Klimaklasse: 1

[1] Tab. 2.1 Lastvarighetsklasse for nyttelastt: Halvårslast

[1] NA.2.3 Partialfaktor for limtre: $\gamma_M := 1.15$

Lengde: $L := 13500\ mm$ $l_1 := 6550\ mm$ $l_2 := 6700\ mm$

Bjelketverrrsnitt: $b := 215\ mm$ $h := 585\ mm$

Lastbredde: $Lb := 3300\ mm$

[3] Tab.6 Limtre GL30c:

$\rho_m := 430\ \frac{kg}{m^3}$ $g := 9.81\ \frac{m}{s^2}$ $f_{m.k} := 30\ \frac{N}{mm^2}$

$E_{0.05} := 10800\ \frac{N}{mm^2}$ $f_{c.90k} := 2.5\ \frac{N}{mm^2}$ $f_{v.k} := 3.5\ \frac{N}{mm^2}$

$E_{mean} := 13000\ \frac{N}{mm^2}$

LASTER

Egenlast dekke: $g_{d.k} := 2.595\ \frac{kN}{m^2} \cdot Lb$ $g_{d.k} = 8.564\ \frac{kN}{m}$

Egenlast bjelke: $g_{b.k} := b \cdot h \cdot \rho_m \cdot g$ $g_{b.k} = 0.531\ \frac{kN}{m}$

Permanent last: $g_k := g_{d.k} + g_{b.k}$ $g_k = 9.094\ \frac{kN}{m}$

Punktlast fra fasade: $g_f := 1\ \frac{kN}{m} \cdot Lb$ $g_f = 3.3\ kN$

Nyttelast: $q_k := 2.0 \frac{kN}{m^2} \cdot Lb$ $q_k = 6.6 \frac{kN}{m}$

BRUDDGRENSEKONTROLL

[2] Tab. NA.A1.1 $\psi_0 := 0.7$ $\psi_1 := 0.5$ $\psi_2 := 0.3$ (Nyttelast kategori A)

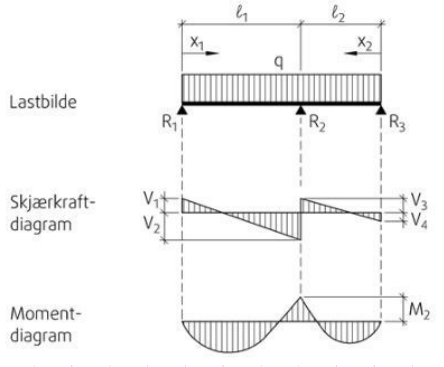
[1] (3.2) Høydefaktor: $k_h := min\left(\left(\frac{600\ mm}{h}\right)^{0.1}, 1.1\right)$ $k_h = 1.003$

Lastkombinasjoner

[1] Tab. 3.1 1. Egenlast og nyttelast $k_{mod} := 0.8$

$p_{d1} := 1.2 \cdot g_k + 1.5 \cdot q_k$ $p_{d1} = 20.813 \frac{kN}{m}$

[6] Nr. 81 **Dimensjonerende krefter**



$$M_2 := \frac{\left(p_{d1} \cdot l_2^{\ 3} + p_{d1} \cdot l_1^{\ 3}\right)}{8 \cdot \left(l_1 + l_2\right)}$$

$M_2 = 114.23\ kN \cdot m$

$R_1 := \frac{-M_2}{l_1} + \frac{p_{d1} \cdot l_1}{2}$ $R_1 = 50.722\ kN$

$R_3 := \frac{-M_2}{l_2} + \frac{p_{d1} \cdot l_2}{2}$ $R_3 = 52.674\ kN$

$V_1 := R_1$

$V_4 := R_3$

$V_2 := p_{d1} \cdot l_1 - R_1$ $V_2 = 85.602\ kN$

$V_3 := p_{d1} \cdot l_2 - R_3$ $V_3 = 86.772\ kN$

$R_2 := p_{d1} \cdot (l_1 + l_2) - R_1 - R_3$ $R_2 = 172.374\ kN$

$x_1 := \frac{R_1}{p_{d1}}$ $M_1 := R_1 \cdot x_1 - \frac{p_{d1} \cdot {x_1}^2}{2}$ $M_1 = 61.807\ kN \cdot m$

$x_2 := \frac{R_3}{p_{d1}}$ $M_3 := R_1 \cdot x_2 - \frac{p_{d1} \cdot {x_2}^2}{2}$ $M_3 = 61.716\ kN \cdot m$

$V_{Ed} := \max(V_1, V_2, V_3, V_4)$ $V_{Ed} = 86.772\ kN$

$M_{Ed} := \max(M_1, M_2, M_3)$ $M_{Ed} = 114.23\ kN \cdot m$

Dimensjonerende fastheter

$f_{m.d} := f_{m.k} \cdot \frac{k_h \cdot k_{mod}}{\gamma_M}$ $f_{m.d} = 20.922\ \frac{N}{mm^2}$

$f_{c.90.d} := f_{c.90k} \cdot \frac{k_{mod}}{\gamma_M}$ $f_{c.90.d} = 1.739\ \frac{N}{mm^2}$

$f_{v.d} := f_{v.k} \cdot \frac{k_{mod}}{\gamma_M}$ $f_{v.d} = 2.435\ \frac{N}{mm^2}$

[1] 6.3.3 **Vipping**

Forutsetter at bjelken er gaffellagret (sikret mot rotasjon) om egen akse ved oppleggene. Videre antar en at etasjeskilleren t vil sikre mot forskyvninger på tvers. En mulighet for vipping er over midtopplegg med trykk i underkant, regner konservativt den som en konsentrert last.

[1] Tab. 6.1 $l_{ef} := 0.8 \cdot 6750\ mm + 2 \cdot h$ $\quad l_{ef} = 6.57\ m$

$$\sigma_{m.crit} := \frac{\left(0.78 \cdot b^2\right)}{h \cdot l_{ef}} \cdot E_{0.05} \qquad \sigma_{m.crit} = 101.315\ \frac{N}{mm^2}$$

$$\lambda_{relm} := \sqrt{\frac{f_{m.k}}{\sigma_{m.crit}}} \qquad \lambda_{relm} = 0.544$$

$$\lambda_{relm} \le 0.75 \qquad k_{crit} := 1.0$$

[1] 6.1.6 **Bøyekontroll**

$$W := \frac{\left(b \cdot h^2\right)}{6} \qquad W = \left(1.226 \cdot 10^7\right)\ mm^3$$

$$\sigma_{m.d} := \frac{M_{Ed}}{W} \qquad \sigma_{m.d} = 9.315\ \frac{N}{mm^2}$$

$$\frac{\sigma_{m.d}}{k_{crit} \cdot f_{m.d}} = 0.445$$ < 1.0 OK

[1] 6.1.7 **Skjærkontroll**

Sprekkfaktor for limtre: $k_{cr} := 0.67$

$$b_{ef} := k_{cr} \cdot b$$

$$\tau_d := \frac{\left(3 \cdot V_{Ed}\right)}{2 \cdot b_{ef} \cdot h} \qquad \tau_d = 1.545\ \frac{N}{mm^2}$$

$$\frac{\tau_d}{f_{v.d}} = 0.634$$ < 1.0 OK

[1] 6.1.5 **Trykk vinkelrett på fiberretning**

Trykk vinkelrett på fiberretningen blir ikke kontrollert i Calculatis

Søyledimensjon: $b_s := 215\ mm$ $h_s := 225\ mm$

Dimensjonerende opplagerkraft ved midtopplegg:

$F_{c.90.d} := R_2$

$A_{ef} := b_s \cdot (h_s + 60\ mm)$

$\sigma_{c.90.d} := \frac{F_{c.90.d}}{A_{ef}}$ $\sigma_{c.90.d} = 2.813\ \frac{N}{mm^2}$

$k_{c.90} := 1.75$ (Hviler på enkeltopplegg, $l_1 \geq 2 \cdot h$, limtre og $l \leq 400\ mm$)

$\frac{\sigma_{c.90.d}}{k_{c.90} \cdot f_{c.90.d}} = 0.924$ < 1.0 OK

Dimensjonerende opplagerkraft ved endeopplegg:

$F_{c.90.d.e} := R_3$

$A_{ef.e} := b_s \cdot (h_s + 30\ mm)$

$\sigma_{c.90.d.e} := \frac{F_{c.90.d.e}}{A_{ef.e}}$ $\sigma_{c.90.d} = 2.813\ \frac{N}{mm^2}$

$k_{c.90} := 1.75$ (Hviler på enkeltopplegg, $l_1 \geq 2 \cdot h$, limtre og $l \leq 400\ mm$)

$\frac{\sigma_{c.90.d.e}}{k_{c.90} \cdot f_{c.90.d}} = 0.316$ < 1.0 OK

BRUKSGRENSEKONTROLL

Nedbøyning

[1] Tab 3.2 $k_{def} := 0.6$ (Klimaklasse 1) $I := \frac{1}{12} \cdot b \cdot h^3$

Forenkler og benytter formel for en kontinuerlig bjelke med to like spenn og jevnt fordelt last. Setter lengden lik det lengste spennet. Kontrolleres i Calculatis, forventer da noe avvik.

[1] Tab 7.2 Øyeblikkelig nedbøyning

$\delta_g := \frac{1}{185} \cdot \frac{g_k \cdot {l_2}^4}{E_{mean} \cdot I}$ $\delta_g = 2.124\ mm$

$\delta_q := \frac{1}{185} \cdot \frac{q_k \cdot {l_2}^4}{E_{mean} \cdot I}$ $\delta_q = 1.542\ mm$

$w_{inst} := \delta_g + \delta_q$ $w_{inst} = 3.666\ mm$ < $\frac{l_2}{500} = 13.4\ mm$

Langtidsnedbøyning

$\delta_{gfin} := \delta_g \cdot (1 + k_{def})$ $\delta_{gfin} = 3.399\ mm$

$\delta_{qfin} := \delta_q \cdot (1 + \psi_2 \cdot k_{def})$ $\delta_{qfin} = 1.819\ mm$

$w_{fin} := \delta_{gfin} + \delta_{qfin}$ $w_{fin} = 5.218\ mm$ < $\frac{l_2}{300} = 22.333\ mm$

BRANNDIMENSJONERING

[4] Tab 3.1 Dimensjonerende forkullingshastighet for limtre: $\beta_n := 0.7\ \frac{mm}{min}$

[5] § 11-3 Brannklasse 2

[5] § 11-4 Brannmotstandstid R60: $t := 60\ min$

[4] 4.2.2(5) $k_{mod.fi} := 1.0$

[4] 2.3 (Merknad 2) $\gamma_{M.fi} := 1.0$

[4] 4.2.2 **Effektiv forkullingsdybde**

$k_0 := 1$ t > 20 min

$d_{char.n} := \beta_n \cdot t$

$d_0 := 7\ mm$

$d_{ef} := d_{char.n} + k_0 \cdot d_0$ $\qquad d_{ef} = 49\ mm$

Dimensjon effektivt resttverrsnitt

$b_{ef.fi} := b - 2 \cdot d_{ef}$ $\qquad b_{ef.fi} = 117\ mm$

$h_{ef.fi} := h - 1 \cdot d_{ef}$ $\qquad h_{ef.fi} = 536\ mm$

Dimensjonerende fastheter

$f_{m.d.fi} := 1.15 \cdot f_{m.k}$ $\qquad f_{m.d.fi} = 34.5\ \dfrac{N}{mm^2}$

$f_{v.d.fi} := 1.15 \cdot f_{v.k}$ $\qquad f_{v.d.fi} = 4.025\ \dfrac{N}{mm^2}$

Dimensjonerende krefter

(Verdier hentet fra Calculatis)

$M_{d.maks.fi} := 64.41\ kN \cdot m$

$V_{d.maks.fi} := 48.3\ kN$

[1] 6.3.3 **Vipping**

$$\sigma_{m.crit.fi} := \frac{\left(0.78 \cdot b_{ef.fi}^{\ 2}\right)}{h_{ef.fi} \cdot l_{ef}} \cdot E_{0.05} \qquad \sigma_{m.crit.fi} = 32.746\ \frac{N}{mm^2}$$

$$\lambda_{relm.fi} := \sqrt{\frac{f_{m.k}}{\sigma_{m.crit.fi}}} \qquad \lambda_{relm.fi} = 0.957 \qquad 0.75 < \lambda_{relm.fi} < 1.4$$

$$k_{crit.fi} := 1.56 - 0.75 \cdot \lambda_{relm.fi} \qquad k_{crit.fi} = 0.842$$

[1] 6.1.6 **Bøyekontroll**

$$W_{fi} := \frac{\left(b_{ef.fi} \cdot h_{ef.fi}^{\ 2}\right)}{6} \qquad W_{fi} = \left(5.602 \cdot 10^{6}\right)\ mm^{3}$$

$$\sigma_{m.d.fi} := \frac{M_{d.maks.fi}}{W_{fi}} \qquad \sigma_{m.d.fi} = 11.497\ \frac{N}{mm^{2}}$$

$$\frac{\sigma_{m.d.fi}}{k_{crit.fi} \cdot f_{m.d.fi}} = 0.396$$ < 1.0 OK

[1] 6.1.7 **Skjærkontroll**

$$b_{ef} := k_{cr} \cdot b_{ef.fi}$$

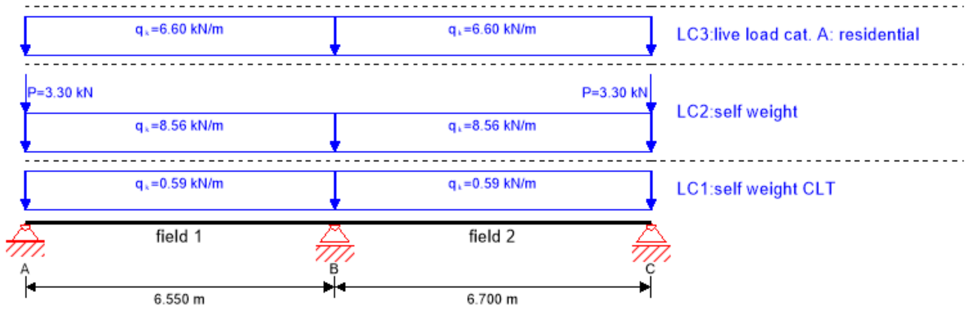
$$\tau_{d.fi} := \frac{\left(3 \cdot V_{d.maks.fi}\right)}{2 \cdot b_{ef} \cdot h_{ef.fi}}$$

$$\frac{\tau_{d.fi}}{f_{v.d.fi}} = 0.428$$ < 1 OK

Kan dermed konkludere med at bjelken beholder tilstrekkelig bæreevne ved en 60 minutters brann.

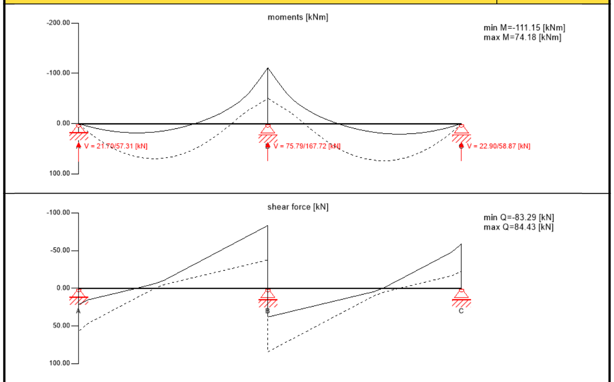
VEDLEGG F4.2

system



section: wooden beam 21.5/58.5; **material**: GL30c; **service class**: service class 1; **fire resistance class**: R 60

utilization 53 %



flexural stress analysis 43 %

$M_{y,d}$ =	-111.15	kNm	$f_{m,k}$ =	30.00	N/mm²
$N_{t,d}$ =	0.00	kN	$f_{t,k}$ =	19.50	N/mm²
$\sigma_{t,d}$ =	0.00	N/mm²	$f_{t,d}$ =	13.57	N/mm²
$\sigma_{m,y,d}$ =	9.06	N/mm² <	$f_{m,y,d}$ =	20.87	N/mm² ✓

shear stress analysis 53 %

V_d =	72.58	kN	$f_{v,k}$ =	3.50	N/mm²
$\tau_{v,d}$ =	0.87	N/mm² <	$f_{v,d}$ =	1.63	N/mm² ✓

lateral torsional buckling analysis 43 %

$M_{y,d}$ =	-111.15	kNm	$f_{m,k}$ =	30.00	N/mm²
$N_{c,d}$ =	0.00	kN	$f_{c,k}$ =	24.50	N/mm²
$\sigma_{c,d}$ =	0.00	N/mm²	$f_{c,d}$ =	17.04	N/mm²
$\sigma_{m,y,d}$ =	9.06	N/mm² <	$f_{m,y,d}$ =	20.87	N/mm² ✓

flexural stress analysis fire 35 %

$M_{y,d}$ =	-68.34	kNm	$f_{m,k}$ =	30.00	N/mm²
$N_{t,d}$ =	0.00	kN	$f_{t,k}$ =	19.50	N/mm²
$\sigma_{t,d}$ =	0.00	N/mm²	$f_{t,d}$ =	22.43	N/mm²
$\sigma_{m,y,d}$ =	12.20	N/mm² <	$f_{m,y,d}$ =	34.50	N/mm² ✓

shear stress analysis fire 40 %

V_d =	45.24	kN	$f_{v,k}$ =	3.50	N/mm²
$\tau_{v,d}$ =	1.08	N/mm² <	$f_{v,d}$ =	2.70	N/mm² ✓

lateral torsional buckling analysis fire 46 %

$M_{y,d}$ =	-68.34	kNm	$f_{m,k}$ =	30.00	N/mm²
$N_{c,d}$ =	0.00	kN	$f_{c,k}$ =	24.50	N/mm²
$\sigma_{c,d}$ =	0.00	N/mm²	$f_{c,d}$ =	28.18	N/mm²
$\sigma_{m,y,d}$ =	12.20	N/mm² <	$f_{m,y,d}$ =	34.50	N/mm² ✓

w_{inst} = w[char]

field	K_{def}	limit	w_{limit}	$w_{calc.}$	ratio
		[-]	[mm]	[mm]	
1	0.6	L/500	13.1	4.3	33 %
2	0.6	L/500	13.4	4.8	36 %

w_{fin} = w[char] + w[q.p.]*kdef

field	K_{def}	limit	w_{limit}	$w_{calc.}$	ratio
		[-]	[mm]	[mm]	
1	0.6	L/300	21.8	5.8	27 %
2	0.6	L/300	22.3	6.6	29 %

$w_{net,fin}$ = w[q.p.] + w[q.p.]*kdef

field	K_{def}	limit	w_{limit}	$w_{calc.}$	ratio
		[-]	[mm]	[mm]	
1	0.6	L/350 L/350	18.7	4.2	22 %
2	0.6	L/350 L/350	19.1	4.8	25 %

support reaction				
load case category	k_{mod}	A_V	B_V	C_V
			[kN]	
self weight CLT	0.6	1.44	4.90	1.50
		1.44	4.90	1.50
self weight	0.6	24.16	70.89	24.96
		24.16	70.89	24.96
live load cat. A: residential	0.8	18.94	54.66	19.31
		-2.86	0.00	-2.61

Disclaimer

VEDLEGG F5.1

BJELKE 3-4

Referanse til standarder

[1] NS-EN 1995-1-1: Allmenne regler og regler for bygninger
[2] NS-EN 1990: Grunnlag for prosjektering av konstruksjoner
[3] Norsk Limtreprodusenters Forening, *Limtreboka - Beregningseksempler* (2018)
[4] NS-EN 1995-1-2: Brannteknisk dimensjonering
[5] Byggeteknisk forskrift (TEK17)

FORUTSETNINGER OG ANTAKELSER

[1] NA.901 Klimaklasse: 1

[1] Tab. 2.1 Lastvarighetsklasse for snølast: Korttidslast
Lastvarighetsklasse for vindlasten: Øyeblikkslast

[1] NA.2.3 Partialfaktor for limtre: $\gamma_M := 1.15$

Lengde: $L := 7300\ mm$

Bjelketverrrsnitt: $b := 215\ mm$ $h := 585\ mm$

Lastbredde: $Lb := 3300\ mm$

[3] Tab.6 Limtre GL30c:

$\rho_m := 430\ \dfrac{kg}{m^3}$ $g := 9.81\ \dfrac{m}{s^2}$ $f_{m.k} := 30\ \dfrac{N}{mm^2}$

$E_{0.05} := 10800\ \dfrac{N}{mm^2}$ $f_{c.90k} := 2.5\ \dfrac{N}{mm^2}$ $f_{v.k} := 3.5\ \dfrac{N}{mm^2}$

$E_{mean} := 13000\ \dfrac{N}{mm^2}$

LASTER

Egenlast etasjeskiller: $g_{d.k} := 2.595\ \dfrac{kN}{m^2} \cdot Lb$ $g_{d.k} = 8.564\ \dfrac{kN}{m}$

Egenlast bjelke: $g_{b.k} := b \cdot h \cdot \rho_m \cdot g$ $g_{b.k} = 0.531\ \dfrac{kN}{m}$

Permanent last: $g_k := g_{d.k} + g_{b.k}$ $g_k = 9.094\ \dfrac{kN}{m}$

Nyttelast: $q_k := 2.0 \dfrac{kN}{m^2} \cdot Lb$ $q_k = 6.6 \dfrac{kN}{m}$

BRUDDGRENSEKONTROLL

[2] Tab. NA.A1.1 $\psi_0 := 0.7$ $\psi_1 := 0.5$ $\psi_2 := 0.3$ (Nyttelast kategori A)

[1] (3.2) Høydefaktor: $k_h := min\left(\left(\dfrac{600\ mm}{h}\right)^{0.1}, 1.1\right)$ $k_h = 1.003$

Lastkombinasjoner

[1] Tab. 3.1 1. Egenlast og nyttelast $k_{mod} := 0.8$

$p_{d1} := 1.2 \cdot g_k + 1.5 \cdot q_k$ $p_{d1} = 20.813 \dfrac{kN}{m}$

$p_{d2} := 1.3 \cdot g_k + 1.05 \cdot q_k$ $p_{d2} = 18.752 \dfrac{kN}{m}$

Dimensjonerende fastheter

$f_{m.d} := f_{m.k} \cdot \dfrac{k_h \cdot k_{mod}}{\gamma_M}$ $f_{m.d} = 20.922 \dfrac{N}{mm^2}$

$f_{c.90.d} := f_{c.90k} \cdot \dfrac{k_{mod}}{\gamma_M}$ $f_{c.90.d} = 1.739 \dfrac{N}{mm^2}$

$f_{v.d} := f_{v.k} \cdot \dfrac{k_{mod}}{\gamma_M}$ $f_{v.d} = 2.435 \dfrac{N}{mm^2}$

Dimensjonerende krefter

$M_{Ed} := \dfrac{p_{d1} \cdot L^2}{8}$ $M_{Ed} = 138.64\ kN \cdot m$

$V_{Ed} := \dfrac{p_{d1} \cdot L}{2}$ $V_{Ed} = 75.967\ kN$

[1] 6.3.3 **Vipping**

Forutsetter at bjelken er gaffellagret (sikret mot rotasjon) om egen akse ved oppleggene. Videre antar en at etasjeskilleren vil sikre mot forskyvninger på tvers. $k_{crit} := 1.0$. Om bjelken ikke var sikret mot vipping mellom oppleggene:

[1] Tab. 6.1 $l_{ef} := 0.9 \cdot 7300\ mm + 2 \cdot h$ $\qquad l_{ef} = 7.74\ m$

$$\sigma_{m.crit} := \frac{0.78 \cdot b^2}{h \cdot l_{ef}} \cdot E_{0.05} \qquad \sigma_{m.crit} = 86\ \frac{N}{mm^2}$$

$$\lambda_{relm} := \sqrt{\frac{f_{m.k}}{\sigma_{m.crit}}} \qquad \lambda_{relm} = 0.591$$

$$\lambda_{relm} \le 0.75 \qquad k_{crit} := 1.0$$

[1] 6.1.6 **Bøyekontroll**

$$W := \frac{\left(b \cdot h^2\right)}{6} \qquad W = \left(1.226 \cdot 10^7\right)\ mm^3$$

$$\sigma_{m.d} := \frac{M_{Ed}}{W} \qquad \sigma_{m.d} = 11.305\ \frac{N}{mm^2}$$

$\frac{\sigma_{m.d}}{k_{crit} \cdot f_{m.d}} = 0.54$ < 1.0 OK

[1] 6.1.7 **Skjærkontroll**

Sprekkfaktor for limtre: $k_{cr} := 0.67$

$$b_{ef} := k_{cr} \cdot b$$

$$\tau_d := \frac{\left(3 \cdot V_{Ed}\right)}{2 \cdot b_{ef} \cdot h} \qquad \tau_d = 1.352\ \frac{N}{mm^2}$$

$\frac{\tau_d}{f_{v.d}} = 0.555$ < 1.0 OK

[1] 6.1.5 **Trykk vinkelrett på fiberretning**

Søyledimensjon: $b_s := 215\ mm$ $h_s := 225\ mm$

$F_{c.90.d} := V_{Ed}$

$A_{ef} := b_s \cdot h_s$

$\sigma_{c.90.d} := \frac{F_{c.90.d}}{A_{ef}}$ $\sigma_{c.90.d} = 1.57\ \frac{N}{mm^2}$

$k_{c.90} := 1.75$ (Hviler på enkeltopplegg, $l_1 \geq 2 \cdot h$, limtre og $l \leq 400\ mm$)

$\frac{\sigma_{c.90.d}}{k_{c.90} \cdot f_{c.90.d}} = 0.516$ < 1.0 OK

BRUKSGRENSEKONTROLL

Nedbøyning

[1] Tab 3.2 $k_{def} := 0.6$ (Klimaklasse 1) $I := \frac{1}{12} \cdot b \cdot h^3$

[1] Tab 7.2 Øyeblikkelig nedbøyning

$\delta_g := \frac{5}{384} \cdot \frac{g_k \cdot L^4}{E_{mean} \cdot I}$ $\delta_g = 7.211\ mm$

$\delta_q := \frac{5}{384} \cdot \frac{q_k \cdot L^4}{E_{mean} \cdot I}$ $\delta_q = 5.234\ mm$

$w_{inst} := \delta_g + \delta_q$ $w_{inst} = 12.445\ mm$ < $\frac{L}{500} = 14.6\ mm$

Langtidsnedbøyning

$\delta_{gfin} := \delta_g \cdot (1 + k_{def})$ $\delta_{gfin} = 11.538\ mm$

$\delta_{qfin} := \delta_q \cdot (1 + \psi_2 \cdot k_{def})$ $\delta_{qfin} = 6.176\ mm$

$w_{fin} := \delta_{gfin} + \delta_{qfin}$ $w_{fin} = 17.714\ mm$ < $\frac{L}{300} = 24.333\ mm$

BRANNDIMENSJONERING

[4] Tab 3.1 Dimensjonerende forkullingshastighet for limtre: $\beta_n := 0.7\ \frac{mm}{min}$

[5] § 11-3 Brannklasse 2

[5] § 11-4 Brannmotstandstid R60: $t := 60\ min$

[4] 4.2.2(5) $k_{mod.fi} := 1.0$

[4] 2.3 (Merknad 2) $\gamma_{M.fi} := 1.0$

[4] 4.2.2 **Effektiv forkullingsdybde**

$k_0 := 1$ t > 20 min

$d_{char.n} := \beta_n \cdot t$

$d_0 := 7\ mm$

$d_{ef} := d_{char.n} + k_0 \cdot d_0$ $d_{ef} = 49\ mm$

Dimensjon effektivt resttverrsnitt

$b_{ef.fi} := b - 2 \cdot d_{ef}$ $b_{ef.fi} = 117\ mm$

$h_{ef.fi} := h - 1 \cdot d_{ef}$ $h_{ef.fi} = 536\ mm$

Dimensjonerende fastheter

$f_{m.d.fi} := 1.15 \cdot f_{m.k}$ $f_{m.d.fi} = 34.5\ \frac{N}{mm^2}$

$f_{v.d.fi} := 1.15 \cdot f_{v.k}$ $f_{v.d.fi} = 4.025\ \frac{N}{mm^2}$

Dimensjonerende krefter

(Verdier hentet fra Calculatis)

$M_{d.maks.fi} := 82.94\ kN \cdot m$

$V_{d.maks.fi} := 45.45\ kN$

[1] 6.3.3 **Vipping**

$\sigma_{m.crit.fi} := \frac{\left(0.78 \cdot b_{ef.fi}^{2}\right)}{h_{ef.fi} \cdot l_{ef}} \cdot E_{0.05}$ $\sigma_{m.crit.fi} = 27.796\ \frac{N}{mm^2}$

$\lambda_{relm.fi} := \sqrt{\frac{f_{m.k}}{\sigma_{m.crit.fi}}}$ $\lambda_{relm.fi} = 1.039$ $0.75 < \lambda_{relm.fi} < 1.4$

$k_{crit.fi} := 1.56 - 0.75 \cdot \lambda_{relm.fi}$ $k_{crit.fi} = 0.781$

[1] 6.1.6 **Bøyekontroll**

$W_{fi} := \frac{\left(b_{ef.fi} \cdot h_{ef.fi}^{2}\right)}{6}$ $W_{fi} = \left(5.602 \cdot 10^{6}\right)\ mm^3$

$\sigma_{m.d.fi} := \frac{M_{d.maks.fi}}{W_{fi}}$ $\sigma_{m.d.fi} = 14.805\ \frac{N}{mm^2}$

$\frac{\sigma_{m.d.fi}}{k_{crit.fi} \cdot f_{m.d.fi}} = 0.55$ < 1.0 OK

[1] 6.1.7 **Skjærkontroll**

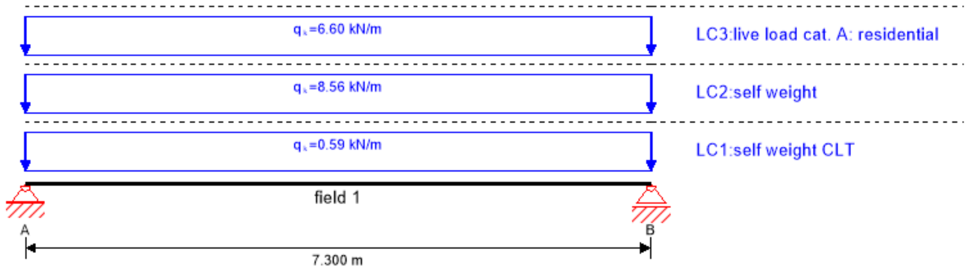
$b_{ef} := k_{cr} \cdot b_{ef.fi}$

$\tau_{d.fi} := \frac{\left(3 \cdot V_{d.maks.fi}\right)}{2 \cdot b_{ef} \cdot h_{ef.fi}}$

$$\frac{\tau_{d.fi}}{f_{v.d.fi}} = 0.403 \quad < 1 \qquad \text{OK}$$

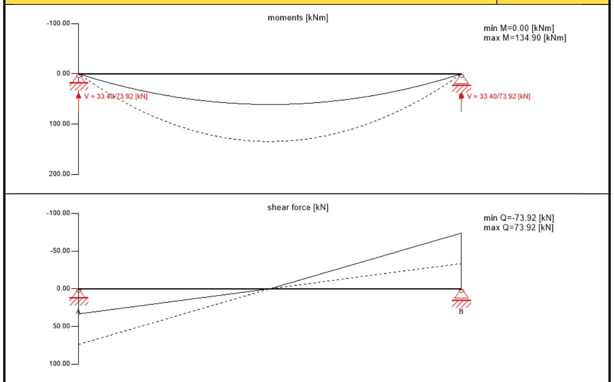
Kan konkludere med at bjelken beholder tilstrekkelig bæreevne ved en 60 minutters brann.

system



section: wooden beam 21.5/58.5; **material**: GL30c; **service class**: service class 1; **fire resistance class**: R 60

utilization 86 %



flexural stress analysis 53 %

$M_{y,d}$ =	134.90	kNm	$f_{m,k}$ =	30.00	N/mm²
$N_{t,d}$ =	0.00	kN	$f_{t,k}$ =	19.50	N/mm²
$\sigma_{t,d}$ =	0.00	N/mm²	$f_{t,d}$ =	13.57	N/mm²
$\sigma_{m,y,d}$ =	11.00	N/mm² <	$f_{m,y,d}$ =	20.87	N/mm² ✓

shear stress analysis 45 %

V_d =	62.07	kN	$f_{v,k}$ =	3.50	N/mm²
$\tau_{v,d}$ =	0.74	N/mm² <	$f_{v,d}$ =	1.63	N/mm² ✓

lateral torsional buckling analysis 53 %

$M_{y,d}$ =	134.90	kNm	$f_{m,k}$ =	30.00	N/mm²
$N_{c,d}$ =	0.00	kN	$f_{c,k}$ =	24.50	N/mm²
$\sigma_{c,d}$ =	0.00	N/mm²	$f_{c,d}$ =	17.04	N/mm²
$\sigma_{m,y,d}$ =	11.00	N/mm² <	$f_{m,y,d}$ =	20.87	N/mm² ✓

flexural stress analysis fire 43 %

$M_{y,d}$ =	82.94	kNm	$f_{m,k}$ =	30.00	N/mm²
$N_{t,d}$ =	0.00	kN	$f_{t,k}$ =	19.50	N/mm²
$\sigma_{t,d}$ =	0.00	N/mm²	$f_{t,d}$ =	22.43	N/mm²
$\sigma_{m,y,d}$ =	14.80	N/mm² <	$f_{m,y,d}$ =	34.50	N/mm² ✓

shear stress analysis fire 34 %

V_d =	38.77	kN	$f_{v,k}$ =	3.50	N/mm²
$\tau_{v,d}$ =	0.93	N/mm² <	$f_{v,d}$ =	2.70	N/mm² ✓

lateral torsional buckling analysis fire 61 %

$M_{y,d}$ =	82.94	kNm	$f_{m,k}$ =	30.00	N/mm²
$N_{c,d}$ =	0.00	kN	$f_{c,k}$ =	24.50	N/mm²
$\sigma_{c,d}$ =	0.00	N/mm²	$f_{c,d}$ =	28.18	N/mm²
$\sigma_{m,y,d}$ =	14.80	N/mm² <	$f_{m,y,d}$ =	34.50	N/mm² ✓

w_{inst} = w[char]

field	K_{def}	limit	w_{limit}	$w_{calc.}$	ratio
		[-]	[mm]	[mm]	
1	0.6	L/500	14.6	12.5	86 %

w_{fin} = w[char] + w[q.p.]*kdef

field	K_{def}	limit	w_{limit}	$w_{calc.}$	ratio
		[-]	[mm]	[mm]	
1	0.6	L/300	24.3	17.8	73 %

$w_{net,fin}$ = w[q.p.] + w[q.p.]*kdef

field	K_{def}	limit	w_{limit}	$w_{calc.}$	ratio
		[-]	[mm]	[mm]	
1	0.6	L/400 L/400	18.3	14.1	77 %

support reaction

load case category	k_{mod}	A_V	B_V
		[kN]	
self weight CLT	0.6	2.16	2.16
		2.16	2.16
self weight	0.6	31.24	31.24
		31.24	31.24
live load cat. A: residential	0.8	24.09	24.09
		0.00	0.00

Disclaimer

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VEDLEGG F6.1

BJELKE 3-3

Referanse til standarder

[1] NS-EN 1995-1-1: Allmenne regler og regler for bygninger
[2] NS-EN 1990: Grunnlag for prosjektering av konstruksjoner
[3] Norsk Limtreprodusenters Forening, *Limtreboka - Beregningseksempler* (2018)
[4] NS-EN 1995-1-2: Brannteknisk dimensjonering
[5] Byggeteknisk forskrift (TEK17)
[6] SINTEF Byggforskserien, nr.421.051 *Statikkformler for bjelker* (2019)

FORUTSETNINGER OG ANTAKELSER

[1] NA.901 Klimaklasse: 1

[1] Tab. 2.1 Lastvarighetsklasse for nyttelastt: Halvårslast

[1] NA.2.3 Partialfaktor for limtre: $\gamma_M := 1.15$

Lengde: $L := 13100\ mm$ $l_1 := 7300\ mm$ $l_2 := 5650\ mm$

Bjelketverrrsnitt: $b := 215\ mm$ $h := 585\ mm$

Lastbredde: $Lb := 3200\ mm$

[3] Tab.6 Limtre GL30c:

$\rho_m := 430\ \frac{kg}{m^3}$ $g := 9.81\ \frac{m}{s^2}$ $f_{m.k} := 30\ \frac{N}{mm^2}$

$E_{0.05} := 10800\ \frac{N}{mm^2}$ $f_{c.90k} := 2.5\ \frac{N}{mm^2}$ $f_{v.k} := 3.5\ \frac{N}{mm^2}$

$E_{mean} := 13000\ \frac{N}{mm^2}$

LASTER

Egenlast dekke: $g_{d.k} := 2.595\ \frac{kN}{m^2} \cdot Lb$ $g_{d.k} = 8.304\ \frac{kN}{m}$

Egenlast bjelke: $g_{b.k} := b \cdot h \cdot \rho_m \cdot g$ $g_{b.k} = 0.531\ \frac{kN}{m}$

Permanent last: $g_k := g_{d.k} + g_{b.k}$ $g_k = 8.835\ \frac{kN}{m}$

Punktlast fra fasade: $g_f := 1\,\frac{kN}{m} \cdot Lb$ $g_f = 3.2\ kN$

Nyttelast: $q_k := 2.0\,\frac{kN}{m^2} \cdot Lb$ $q_k = 6.4\,\frac{kN}{m}$

BRUDDGRENSEKONTROLL

[2] Tab. NA.A1.1 $\psi_0 := 0.7$ $\psi_1 := 0.5$ $\psi_2 := 0.3$ (Nyttelast kategori A)

[1] (3.2) Høydefaktor: $k_h := min\left(\left(\frac{600\ mm}{h}\right)^{0.1}, 1.1\right)$ $k_h = 1.003$

Lastkombinasjoner

[1] Tab. 3.1 1. Egenlast og nyttelast $k_{mod} := 0.8$

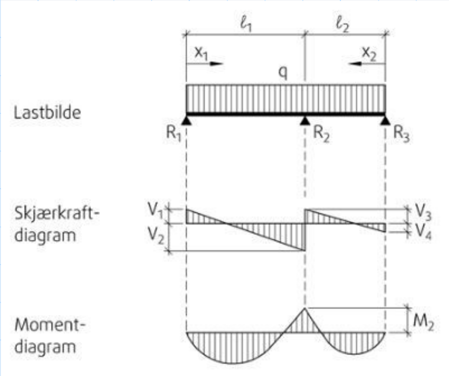
$p_{d1} := 1.2 \cdot g_k + 1.5 \cdot q_k$ $p_{d1} = 20.201\,\frac{kN}{m}$

$p_{d2} := 1.3 \cdot g_k + 1.05 \cdot q_k$ $p_{d2} = 18.205\,\frac{kN}{m}$

[6] Nr. 81 **Dimensjonerende krefter**

$$M_2 := \frac{\left(p_{d1} \cdot l_2^{\,3} + p_{d1} \cdot l_1^{\,3}\right)}{8 \cdot \left(l_1 + l_2\right)}$$

$M_2 = 111.026\ kN \cdot m$



$R_1 := \frac{-M_2}{l_1} + \frac{p_{d1} \cdot l_1}{2}$ $R_1 = 58.526\ kN$

$R_3 := \frac{-M_2}{l_2} + \frac{p_{d1} \cdot l_2}{2}$ $R_3 = 37.419\ kN$

$R_2 := p_{d1} \cdot (l_1 + l_2) - R_1 - R_3 \qquad R_2 = 165.664\ kN$

$V_1 := R_1$

$V_4 := R_3$

$V_2 := p_{d1} \cdot l_1 - R_1 \qquad V_2 = 88.944\ kN$

$V_3 := p_{d1} \cdot l_2 - R_3 \qquad V_3 = 76.72\ kN$

$x_1 := \dfrac{R_1}{p_{d1}} \qquad M_1 := R_1 \cdot x_1 - \dfrac{p_{d1} \cdot x_1^{\ 2}}{2} \qquad M_1 = 84.779\ kN \cdot m$

$x_2 := \dfrac{R_3}{p_{d1}} \qquad M_3 := R_1 \cdot x_2 - \dfrac{p_{d1} \cdot x_2^{\ 2}}{2} \qquad M_3 = 73.752\ kN \cdot m$

$V_{Ed} := \max\left(V_1, V_2, V_3, V_4\right) \qquad V_{Ed} = 88.944\ kN$

$M_{Ed} := \max\left(M_1, M_2, M_3\right) \qquad M_{Ed} = 111.026\ kN \cdot m$

Dimensjonerende fastheter

$f_{m.d} := f_{m.k} \cdot \dfrac{k_h \cdot k_{mod}}{\gamma_M} \qquad f_{m.d} = 20.922\ \dfrac{N}{mm^2}$

$f_{c.90.d} := f_{c.90k} \cdot \dfrac{k_{mod}}{\gamma_M} \qquad f_{c.90.d} = 1.739\ \dfrac{N}{mm^2}$

$f_{v.d} := f_{v.k} \cdot \dfrac{k_{mod}}{\gamma_M} \qquad f_{v.d} = 2.435\ \dfrac{N}{mm^2}$

[1] 6.3.3 **Vipping**

Forutsetter at bjelken er gaffellagret (sikret mot rotasjon) om egen akse ved oppleggene. Videre antar en at etasjeskilleren vil sikre mot forskyvninger på tvers. En mulighet for vipping er over midtopplegg med trykk i underkant, regner konservativt den som en konsentrert last.

[1] Tab. 6.1 $l_{ef} := 0.8 \cdot 6600\ mm + 2 \cdot h$ $\quad l_{ef} = 6.45\ m$

$\sigma_{m.crit} := \frac{(0.78 \cdot b^2)}{h \cdot l_{ef}} \cdot E_{0.05}$ $\quad \sigma_{m.crit} = 103.2\ \frac{N}{mm^2}$

$\lambda_{relm} := \sqrt{\frac{f_{m.k}}{\sigma_{m.crit}}}$ $\quad \lambda_{relm} = 0.539$

$\lambda_{relm} \le 0.75$ $\quad k_{crit} := 1.0$

[1] 6.1.6 **Bøyekontroll**

$W := \frac{(b \cdot h^2)}{6}$ $\quad W = (1.226 \cdot 10^7)\ mm^3$

$\sigma_{m.d} := \frac{M_{Ed}}{W}$ $\quad \sigma_{m.d} = 9.054\ \frac{N}{mm^2}$

$\frac{\sigma_{m.d}}{k_{crit} \cdot f_{m.d}} = 0.433$ < 1.0 OK

[1] 6.1.7 **Skjærkontroll**

Sprekkfaktor for limtre: $k_{cr} := 0.67$

$b_{ef} := k_{cr} \cdot b$

$\tau_d := \frac{(3 \cdot V_{Ed})}{2 \cdot b_{ef} \cdot h}$ $\quad \tau_d = 1.583\ \frac{N}{mm^2}$

$\frac{\tau_d}{f_{v.d}} = 0.65$ < 1.0 OK

[1] 6.1.5 **Trykk vinkelrett på fiberretning**

Søyledimensjon: $b_s := 215\ mm$ $\quad h_s := 225\ mm$

Dimensjonerende opplagerkraft ved midtopplegg:

$F_{c.90.d} := R_2$

$A_{ef} := b_s \cdot (h_s + 60\ mm)$

$\sigma_{c.90.d} := \frac{F_{c.90.d}}{A_{ef}}$ $\sigma_{c.90.d} = 2.704\ \frac{N}{mm^2}$

$k_{c.90} := 1.75$ (Hviler på enkeltopplegg, $l_1 \geq 2 \cdot h$, limtre og $l \leq 400\ mm$)

$\frac{\sigma_{c.90.d}}{k_{c.90} \cdot f_{c.90.d}} = 0.888$ < 1.0 OK

Dimensjonerende opplagerkraft ved endeopplegg:

$F_{c.90.d.e} := R_1$

$A_{ef.e} := b_s \cdot (h_s + 30\ mm)$

$\sigma_{c.90.d.e} := \frac{F_{c.90.d.e}}{A_{ef.e}}$ $\sigma_{c.90.d} = 2.704\ \frac{N}{mm^2}$

$k_{c.90} := 1.75$ (Hviler på enkeltopplegg, $l_1 \geq 2 \cdot h$, limtre og $l \leq 400\ mm$)

$\frac{\sigma_{c.90.d.e}}{k_{c.90} \cdot f_{c.90.d}} = 0.351$ < 1.0 OK

BRUKSGRENSEKONTROLL

Nedbøyning

[1] Tab 3.2 $k_{def} := 0.6$ (Klimaklasse 1) $I := \frac{1}{12} \cdot b \cdot h^3$

Forenkler og benytter formel for en kontinuerlig bjelke med to like spenn og jevnt fordelt last. Setter lengden lik det lengste spennet. Kontrolleres i Calculatis, forventer noe avvik.

[1] Tab 7.2 Øyeblikkelig nedbøyning

$\delta_g := \frac{1}{185} \cdot \frac{g_k \cdot l_1^{\ 4}}{E_{mean} \cdot I}$ $\delta_g = 2.908\ mm$

$\delta_q := \frac{1}{185} \cdot \frac{q_k \cdot l_1^{\ 4}}{E_{mean} \cdot I}$ $\delta_q = 2.107\ mm$

$w_{inst} := \delta_g + \delta_q$ $w_{inst} = 5.015\ mm$ < $\frac{l_1}{500} = 14.6\ mm$

Langtidsnedbøyning

$\delta_{gfin} := \delta_g \cdot (1 + k_{def})$ $\delta_{gfin} = 4.653\ mm$

$\delta_{qfin} := \delta_q \cdot (1 + \psi_2 \cdot k_{def})$ $\delta_{qfin} = 2.486\ mm$

$w_{fin} := \delta_{gfin} + \delta_{qfin}$ $w_{fin} = 7.139\ mm$ < $\frac{l_1}{300} = 24.333\ mm$

BRANNDIMENSJONERING

[4] Tab 3.1 Dimensjonerende forkullingshastighet for limtre: $\beta_n := 0.7\ \frac{mm}{min}$

[5] § 11-3 Brannklasse 2

[5] § 11-4 Brannmotstandstid R60: $t := 60\ min$

[4] 4.2.2(5) $k_{mod.fi} := 1.0$

[4] 2.3 (Merknad 2) $\gamma_{M.fi} := 1.0$

[4] 4.2.2 **Effektiv forkullingsdybde**

$k_0 := 1$ (t > 20 min)

$d_{char.n} := \beta_n \cdot t$

$d_0 := 7\ mm$

$d_{ef} := d_{char.n} + k_0 \cdot d_0$ $\quad d_{ef} = 49\ mm$

Dimensjon effektivt resttverrsnitt

$b_{ef.fi} := b - 2 \cdot d_{ef}$ $\quad b_{ef.fi} = 117\ mm$

$h_{ef.fi} := h - 1 \cdot d_{ef}$ $\quad h_{ef.fi} = 536\ mm$

Dimensjonerende fastheter

$f_{m.d.fi} := 1.15 \cdot f_{m.k}$ $\quad f_{m.d.fi} = 34.5\ \frac{N}{mm^2}$

$f_{v.d.fi} := 1.15 \cdot f_{v.k}$ $\quad f_{v.d.fi} = 4.025\ \frac{N}{mm^2}$

Dimensjonerende krefter

(Verdier hentet fra Calculatis)

$M_{d.maks.fi} := 66.45\ kN \cdot m$

$V_{d.maks.fi} := 53.2\ kN$

[1] 6.3.3 **Vipping**

$\sigma_{m.crit.fi} := \frac{\left(0.78 \cdot b_{ef.fi}^{\ 2}\right)}{h_{ef.fi} \cdot l_{ef}} \cdot E_{0.05}$ $\quad \sigma_{m.crit.fi} = 33.355\ \frac{N}{mm^2}$

$\lambda_{relm.fi} := \sqrt{\frac{f_{m.k}}{\sigma_{m.crit.fi}}}$ $\quad \lambda_{relm.fi} = 0.948$ $\quad 0.75 < \lambda_{relm.fi} < 1.4$

$k_{crit.fi} := 1.56 - 0.75 \cdot \lambda_{relm.fi}$ $\quad k_{crit.fi} = 0.849$

[1] 6.1.6 **Bøyekontroll**

$$W_{fi} := \frac{\left(b_{ef.fi} \cdot h_{ef.fi}^{2}\right)}{6} \qquad W_{fi} = \left(5.602 \cdot 10^{6}\right) mm^{3}$$

$$\sigma_{m.d.fi} := \frac{M_{d.maks.fi}}{W_{fi}} \qquad \sigma_{m.d.fi} = 11.861 \frac{N}{mm^{2}}$$

$$\frac{\sigma_{m.d.fi}}{k_{crit.fi} \cdot f_{m.d.fi}} = 0.405 \quad < 1.0$$ OK

[1] 6.1.7 **Skjærkontroll**

$$b_{ef} := k_{cr} \cdot b_{ef.fi}$$

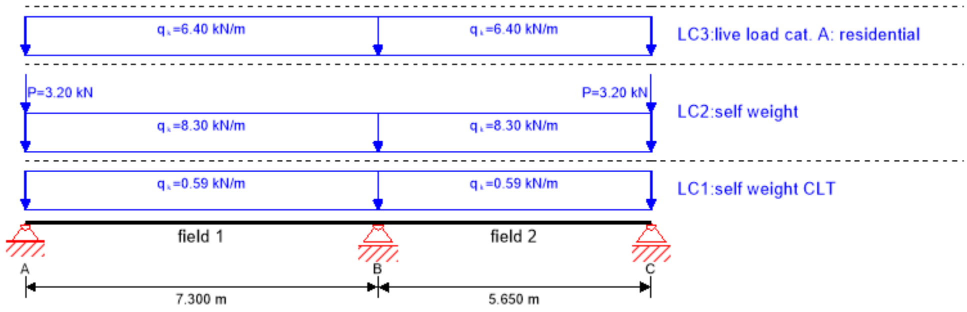
$$\tau_{d.fi} := \frac{\left(3 \cdot V_{d.maks.fi}\right)}{2 \cdot b_{ef} \cdot h_{ef.fi}}$$

$$\frac{\tau_{d.fi}}{f_{v.d.fi}} = 0.472 \quad < 1$$ OK

Kan konkludere med at bjelken beholder tilstrekkelig bæreevne ved en 60 minutters brann.

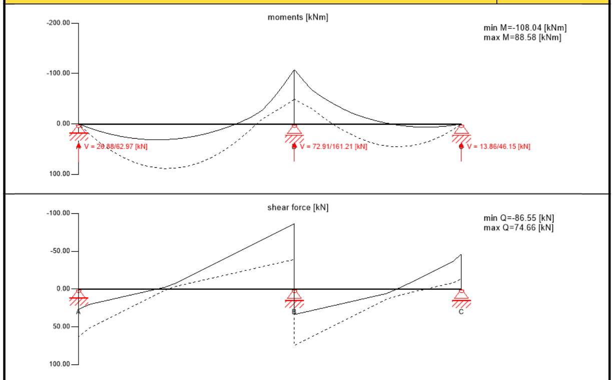
VEDLEGG F6.2

system



section: wooden beam 21.5/58.5; **material**: GL30c; **service class**: service class 1; **fire resistance class**: R 60

utilization 55 %



flexural stress analysis 42 %

$M_{y,d}$ =	- 108.04	kNm	$f_{m,k}$ =	30.00	N/mm²
$N_{t,d}$ =	0.00	kN	$f_{t,k}$ =	19.50	N/mm²
$\sigma_{t,d}$ =	0.00	N/mm²	$f_{t,d}$ =	13.57	N/mm²
$\sigma_{m,y,d}$ =	8.81	N/mm² <	$f_{m,y,d}$ =	20.87	N/mm² ✓

shear stress analysis 55 %

V_d =	75.05	kN	$f_{v,k}$ =	3.50	N/mm²
$\tau_{v,d}$ =	0.90	N/mm² <	$f_{v,d}$ =	1.63	N/mm² ✓

lateral torsional buckling analysis 42 %

$M_{y,d}$ =	- 108.04	kNm	$f_{m,k}$ =	30.00	N/mm²
$N_{c,d}$ =	0.00	kN	$f_{c,k}$ =	24.50	N/mm²
$\sigma_{c,d}$ =	0.00	N/mm²	$f_{c,d}$ =	17.04	N/mm²
$\sigma_{m,y,d}$ =	8.81	N/mm² <	$f_{m,y,d}$ =	20.87	N/mm² ✓

flexural stress analysis fire 34 %

$M_{y,d}$ =	-66.45	kNm	$f_{m,k}$ =	30.00	N/mm²
$N_{t,d}$ =	0.00	kN	$f_{t,k}$ =	19.50	N/mm²
$\sigma_{t,d}$ =	0.00	N/mm²	$f_{t,d}$ =	22.43	N/mm²
$\sigma_{m,y,d}$ =	11.86	N/mm² <	$f_{m,y,d}$ =	34.50	N/mm² ✓

shear stress analysis fire 41 %

V_d =	46.75	kN	$f_{v,k}$ =	3.50	N/mm²
$\tau_{v,d}$ =	1.12	N/mm² <	$f_{v,d}$ =	2.70	N/mm² ✓

lateral torsional buckling analysis fire 44 %

$M_{y,d}$ =	-66.45	kNm	$f_{m,k}$ =	30.00	N/mm²
$N_{c,d}$ =	0.00	kN	$f_{c,k}$ =	24.50	N/mm²
$\sigma_{c,d}$ =	0.00	N/mm²	$f_{c,d}$ =	28.18	N/mm²
$\sigma_{m,y,d}$ =	11.86	N/mm² <	$f_{m,y,d}$ =	34.50	N/mm² ✓

w_{inst} = w[char]

field	K_{def}	limit	w_{limit}	$w_{calc.}$	ratio
		[-]	[mm]	[mm]	
1	0.6	L/500	14.6	7.0	48 %
2	0.6	L/500	11.3	1.8	16 %

w_{fin} = w[char] + w[q.p.]*kdef

field	K_{def}	limit	w_{limit}	$w_{calc.}$	ratio
		[-]	[mm]	[mm]	
1	0.6	L/300	24.3	9.8	40 %
2	0.6	L/300	18.8	2.4	13 %

$w_{net,fin}$ = w[q.p.] + w[q.p.]*kdef

field	K_{def}	limit	w_{limit}	$w_{calc.}$	ratio
		[-]	[mm]	[mm]	
1	0.6	L/350 L/350	20.9	7.4	36 %
2	0.6	L/350 L/350	16.1	1.5	9 %

support reaction				
load case category	k_{mod}	A_V	B_V	C_V
		[kN]		
self weight CLT	0.6	1.71	4.85	1.09
		1.71	4.85	1.09
self weight	0.6	27.25	68.06	18.57
		27.25	68.06	18.57
live load cat. A: residential	0.8	20.07	52.48	16.11
		-1.53	0.00	-4.25

VEDLEGG F7.1

BJELKE 3-1

Referanse til standarder

[1] NS-EN 1995-1-1: Allmenne regler og regler for bygninger
[2] NS-EN 1990: Grunnlag for prosjektering av konstruksjoner
[3] Norsk Limtreprodusenters Forening, *Limtreboka - Beregningseksempler* (2018)
[4] NS-EN 1995-1-2: Brannteknisk dimensjonering
[5] Byggeteknisk forskrift (TEK17)

FORUTSETNINGER OG ANTAKELSER

[1] NA.901 Klimaklasse: 1

[1] Tab. 2.1 Lastvarighetsklasse for snølast: Korttidslast
Lastvarighetsklasse for vindlasten: Øyeblikkslast

[1] NA.2.3 Partialfaktor for limtre: $\gamma_M := 1.15$

Lengde: $L := 7600\ mm$

Bjelketverrrsnitt: $b := 215\ mm$ $h := 585\ mm$

Lastbredde: $Lb := 2400\ mm$

Karakteristisk vindlast: $q_{kast} := 1.34\ \frac{kN}{m^2}$

[3] Tab.6 Limtre GL30c:

$\rho_m := 430\ \frac{kg}{m^3}$ $g := 9.81\ \frac{m}{s^2}$ $f_{m.k} := 30\ \frac{N}{mm^2}$

$E_{0.05} := 10800\ \frac{N}{mm^2}$ $f_{c.90k} := 2.5\ \frac{N}{mm^2}$ $f_{v.k} := 3.5\ \frac{N}{mm^2}$

$E_{mean} := 13000\ \frac{N}{mm^2}$

LASTER

Egenlast takterrasse: $g_{d.k} := 1.025\ \frac{kN}{m^2} \cdot Lb$ $g_{d.k} = 2.46\ \frac{kN}{m}$

Egenlast bjelke: $g_{b.k} := b \cdot h \cdot \rho_m \cdot g$ $g_{b.k} = 0.531\ \frac{kN}{m}$

Permanent last: $g_k := g_{d.k} + g_{b.k}$ $g_k = 2.991\ \frac{kN}{m}$

Snølast: $s_k := 3.57\ \frac{kN}{m^2} \cdot Lb$ $s_k = 8.568\ \frac{kN}{m}$

Vindlast: $v_{t.k} := (0.2 + 0.3) \cdot q_{kast} \cdot Lb$ $v_{t.k} = 1.608\ \frac{kN}{m}$ (trykk)

$v_{s.k} := (-0.7 - 0.2) \cdot q_{kast} \cdot Lb$ $v_{s.k} = -2.894\ \frac{kN}{m}$ (sug)

BRUDDGRENSEKONTROLL

[2] Tab. NA.A1.1 $\psi_{0.s} := 0.7$ $\psi_{1.s} := 0.5$ $\psi_{2.s} := 0.2$ (Snølast)

$\psi_{0.v} := 0.6$ $\psi_{1.v} := 0.2$ $\psi_{2.v} := 0$ (Vindlast)

[1] (3.2) Høydefaktor: $k_h := min\left(\left(\frac{600\ mm}{h}\right)^{0.1}, 1.1\right)$ $k_h = 1.003$

Lastkombinasjoner

[1] Tab. 3.1 1. Kun egenlast $k_{mod1} := 0.6$

$p_{d1} := 1.35 \cdot g_k$ $p_{d1} = 4.037\ \frac{kN}{m}$

[1] Tab. 3.1 2. Egenlast og snølast $k_{mod2} := 0.9$

a) $p_{d2} := 1.35 \cdot g_k + 1.05 \cdot s_k$ $p_{d2} = 13.034\ \frac{kN}{m}$

b) $p_{d3} := 1.2 \cdot g_k + 1.5 \cdot s_k$ $p_{d3} = 16.441\ \frac{kN}{m}$

[1] Tab. 3.1 3. Egenlast, snølast og vindlast $k_{mod3} := 1.1$

a) $p_{d4} := 1.35 \cdot g_k + 1.05 \cdot s_k + 1.05 \cdot v_{t.k}$ $p_{d4} = 14.722 \frac{kN}{m}$

b) $p_{d5} := 1.2 \cdot g_k + 1.5 \cdot s_k + 1.05 \cdot v_{t.k}$ $p_{d5} = 18.129 \frac{kN}{m}$

Dimensjonerende fastheter

$f_{m.d} := f_{m.k} \cdot \frac{k_h \cdot k_{mod2}}{\gamma_M}$ $f_{m.d} = 23.538 \frac{N}{mm^2}$

$f_{c.90.d} := f_{c.90k} \cdot \frac{k_{mod2}}{\gamma_M}$ $f_{c.90.d} = 1.957 \frac{N}{mm^2}$

$f_{v.d} := f_{v.k} \cdot \frac{k_{mod2}}{\gamma_M}$ $f_{v.d} = 2.739 \frac{N}{mm^2}$

Dimensjonerende krefter
Benytter pd3 da pd5 kun er ca 10% høyere, samtidig som kapasiteten øker med 20%

$M_{Ed} := \frac{p_{d3} \cdot L^2}{8}$ $M_{Ed} = 118.702\ kN \cdot m$

$V_{Ed} := \frac{p_{d3} \cdot L}{2}$ $V_{Ed} = 62.475\ kN$

[1] 6.3.3 **Vipping**
Forutsetter at bjelken er gaffellagret (sikret mot rotasjon) om egen akse ved oppleggene. Videre antar en at takterrassen vil sikre mot forskyvninger på tvers. $k_{crit} := 1.0$. Om bjelken ikke var sikret mot vipping mellom oppleggene:

[1] Tab. 6.1 $l_{ef} := 0.9 \cdot 7600\ mm + 2 \cdot h$ $l_{ef} = 8.01\ m$

$\sigma_{m.crit} := \frac{0.78 \cdot b^2}{h \cdot l_{ef}} \cdot E_{0.05}$ $\sigma_{m.crit} = 83.101 \frac{N}{mm^2}$

$$\lambda_{relm} := \sqrt{\frac{f_{m.k}}{\sigma_{m.crit}}} \qquad \lambda_{relm} = 0.601$$

$$\lambda_{relm} \le 0.75 \qquad k_{crit} := 1.0$$

[1] 6.1.6 **Bøyekontroll**

$$W := \frac{\left(b \cdot h^2\right)}{6} \qquad W = \left(1.226 \cdot 10^7\right)\ mm^3$$

$$\sigma_{m.d} := \frac{M_{Ed}}{W} \qquad \sigma_{m.d} = 9.68\ \frac{N}{mm^2}$$

$\frac{\sigma_{m.d}}{k_{crit} \cdot f_{m.d}} = 0.411$ < 1.0 OK

[1] 6.1.7 **Skjærkontroll**

Sprekkfaktor for limtre: $k_{cr} := 0.67$

$$b_{ef} := k_{cr} \cdot b$$

$$\tau_d := \frac{\left(3 \cdot V_{Ed}\right)}{2 \cdot b_{ef} \cdot h} \qquad \tau_d = 1.112\ \frac{N}{mm^2}$$

$\frac{\tau_d}{f_{v.d}} = 0.406$ < 1.0 OK

[1] 6.1.5 **Trykk vinkelrett på fiberretning**

Søyledimensjon: $b_s := 215\ mm$ $h_s := 225\ mm$

$$F_{c.90.d} := V_{Ed}$$

$$A_{ef} := b_s \cdot h_s$$

$$\sigma_{c.90.d} := \frac{F_{c.90.d}}{A_{ef}} \qquad \sigma_{c.90.d} = 1.291\ \frac{N}{mm^2}$$

$k_{c.90} := 1.75$ (Hviler på enkeltopplegg, $l_1 \geq 2 \cdot h$, limtre og $l \leq 400\ mm$)

$\frac{\sigma_{c.90.d}}{k_{c.90} \cdot f_{c.90.d}} = 0.377$ < 1.0 OK

BRUKSGRENSEKONTROLL

Nedbøyning

[1] Tab 3.2 $k_{def} := 0.6$ (Klimaklasse 1) $I := \frac{1}{12} \cdot b \cdot h^3$

[1] Tab 7.2 Øyeblikkelig nedbøyning

$\delta_g := \frac{5}{384} \cdot \frac{g_k \cdot L^4}{E_{mean} \cdot I}$ $\delta_g = 2.786\ mm$

$\delta_s := \frac{5}{384} \cdot \frac{s_k \cdot L^4}{E_{mean} \cdot I}$ $\delta_s = 7.982\ mm$

$\delta_v := \frac{5}{384} \cdot \frac{v_{t.k} \cdot L^4}{E_{mean} \cdot I}$ $\delta_v = 1.498\ mm$

$w_{inst} := \delta_g + \delta_s + \delta_v$ $w_{inst} = 12.266\ mm$ < $\frac{L}{500} = 15.2\ mm$

Langtidsnedbøyning

$\delta_{gfin} := \delta_g \cdot (1 + k_{def})$ $\delta_{gfin} = 4.458\ mm$

$\delta_{sfin} := \delta_s \cdot (1 + \psi_{2.s} \cdot k_{def})$ $\delta_{sfin} = 8.94\ mm$

$\delta_{vfin} := \delta_v \cdot (\psi_{0.v} + \psi_{2.v} \cdot k_{def})$ $\delta_{vfin} = 0.899\ mm$

$w_{fin} := \delta_{gfin} + \delta_{sfin} + \delta_{vfin}$ $w_{fin} = 14.296\ mm$ < $\frac{L}{300} = 25.333\ mm$

BRANNDIMENSJONERING

[4] Tab 3.1 Dimensjonerende forkullingshastighet for limtre: $\beta_n := 0.7 \frac{mm}{min}$

[5] § 11-3 Brannklasse 2

[5] § 11-4 Brannmotstandstid R60: $t := 60\ min$

[4] 4.2.2(5) $k_{mod.fi} := 1.0$

[4] 2.3 (Merknad 2) $\gamma_{M.fi} := 1.0$

[4] 4.2.2 **Effektiv forkullingsdybde**

$k_0 := 1$ t > 20 min

$d_{char.n} := \beta_n \cdot t$

$d_0 := 7\ mm$

$d_{ef} := d_{char.n} + k_0 \cdot d_0$ $d_{ef} = 49\ mm$

Dimensjon effektivt resttverrsnitt

$b_{ef.fi} := b - 2 \cdot d_{ef}$ $b_{ef.fi} = 117\ mm$

$h_{ef.fi} := h - 1 \cdot d_{ef}$ $h_{ef.fi} = 536\ mm$

Dimensjonerende fastheter

$f_{m.d.fi} := 1.15 \cdot f_{m.k}$ $f_{m.d.fi} = 34.5 \frac{N}{mm^2}$

$f_{v.d.fi} := 1.15 \cdot f_{v.k}$ $f_{v.d.fi} = 4.025 \frac{N}{mm^2}$

Dimensjonerende krefter

(Verdier hentet fra Calculatis)

$M_{d.maks.fi} := 52.97\ kN \cdot m$

$V_{d.maks.fi} := 27.88\ kN$

[1] 6.3.3 **Vipping**

$\sigma_{m.crit.fi} := \frac{\left(0.78 \cdot b_{ef.fi}^{2}\right)}{h_{ef.fi} \cdot l_{ef}} \cdot E_{0.05}$ $\sigma_{m.crit.fi} = 26.859\ \frac{N}{mm^2}$

$\lambda_{relm.fi} := \sqrt{\frac{f_{m.k}}{\sigma_{m.crit.fi}}}$ $\lambda_{relm.fi} = 1.057$ $0.75 < \lambda_{relm.fi} < 1.4$

$k_{crit.fi} := 1.56 - 0.75 \cdot \lambda_{relm.fi}$ $k_{crit.fi} = 0.767$

[1] 6.1.6 **Bøyekontroll**

$W_{fi} := \frac{\left(b_{ef.fi} \cdot h_{ef.fi}^{2}\right)}{6}$ $W_{fi} = \left(5.602 \cdot 10^{6}\right)\ mm^3$

$\sigma_{m.d.fi} := \frac{M_{d.maks.fi}}{W_{fi}}$ $\sigma_{m.d.fi} = 9.455\ \frac{N}{mm^2}$

$\frac{\sigma_{m.d.fi}}{k_{crit.fi} \cdot f_{m.d.fi}} = 0.357$ < 1.0 OK

[1] 6.1.7 **Skjærkontroll**

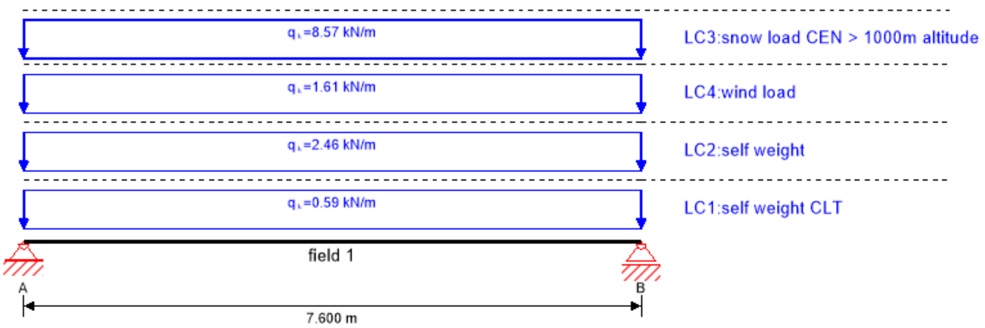
$b_{ef} := k_{cr} \cdot b_{ef.fi}$

$\tau_{d.fi} := \frac{\left(3 \cdot V_{d.maks.fi}\right)}{2 \cdot b_{ef} \cdot h_{ef.fi}}$

$$\frac{\tau_{d.fi}}{f_{v.d.fi}} = 0.247 \quad < 1 \quad \text{OK}$$

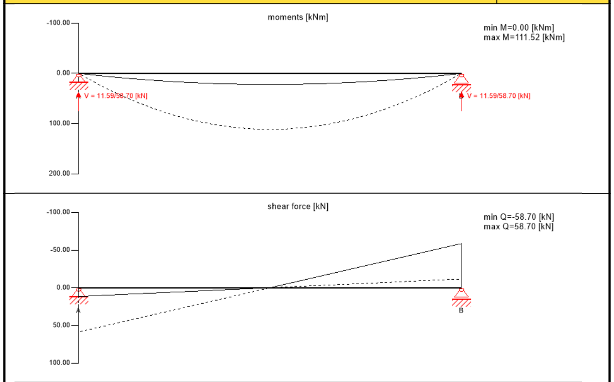
Kan konkludere med at bjelken beholder tilstrekkelig bæreevne ved en 60 minutters brann.

system



section: wooden beam 21.5/58.5; **material**: GL30c; **service class**: service class 1; **fire resistance class**: R 60

utilization 77 %



flexural stress analysis 44 %

$M_{y,d}$ =	111.52	kNm	$f_{m,k}$ =	30.00	N/mm²
$N_{t,d}$ =	0.00	kN	$f_{t,k}$ =	19.50	N/mm²
$\sigma_{t,d}$ =	0.00	N/mm²	$f_{t,d}$ =	13.57	N/mm²
$\sigma_{m,y,d}$ =	9.09	N/mm² <	$f_{m,y,d}$ =	20.87	N/mm² ✓

shear stress analysis 36 %

V_d =	49.66	kN	$f_{v,k}$ =	3.50	N/mm²
$\tau_{v,d}$ =	0.59	N/mm² <	$f_{v,d}$ =	1.63	N/mm² ✓

lateral torsional buckling analysis 44 %

$M_{y,d}$ =	111.52	kNm	$f_{m,k}$ =	30.00	N/mm²
$N_{c,d}$ =	0.00	kN	$f_{c,k}$ =	24.50	N/mm²
$\sigma_{c,d}$ =	0.00	N/mm²	$f_{c,d}$ =	17.04	N/mm²
$\sigma_{m,y,d}$ =	9.09	N/mm² <	$f_{m,y,d}$ =	20.87	N/mm² ✓

flexural stress analysis fire 27 %

$M_{y,d}$ =	52.97	kNm	$f_{m,k}$ =	30.00	N/mm²
$N_{t,d}$ =	0.00	kN	$f_{t,k}$ =	19.50	N/mm²
$\sigma_{t,d}$ =	0.00	N/mm²	$f_{t,d}$ =	22.43	N/mm²
$\sigma_{m,y,d}$ =	9.45	N/mm² <	$f_{m,y,d}$ =	34.50	N/mm² ✓

shear stress analysis fire 21 %

V_d =	23.95	kN	$f_{v,k}$ =	3.50	N/mm²
$\tau_{v,d}$ =	0.57	N/mm² <	$f_{v,d}$ =	2.70	N/mm² ✓

lateral torsional buckling analysis fire 40 %

$M_{y,d}$ =	52.97	kNm	$f_{m,k}$ =	30.00	N/mm²
$N_{c,d}$ =	0.00	kN	$f_{c,k}$ =	24.50	N/mm²
$\sigma_{c,d}$ =	0.00	N/mm²	$f_{c,d}$ =	28.18	N/mm²
$\sigma_{m,y,d}$ =	9.45	N/mm² <	$f_{m,y,d}$ =	34.50	N/mm² ✓

w_{inst} = w[char]

field	K_{def}	limit	w_{limit}	$w_{calc.}$	ratio
		[-]	[mm]	[mm]	
1	0.6	L/500	15.2	11.7	77 %

w_{fin} = w[char] + w[q.p.]*kdef

field	K_{def}	limit	w_{limit}	$w_{calc.}$	ratio
		[-]	[mm]	[mm]	
1	0.6	L/300	25.3	14.4	57 %

$w_{net,fin}$ = w[q.p.] + w[q.p.]*kdef

field	K_{def}	limit	w_{limit}	$w_{calc.}$	ratio
		[-]	[mm]	[mm]	
1	0.6	L/350 L/350	21.7	7.1	33 %

storaenso

support reaction			
load case category	k_{mod}	A_V	B_V
		[kN]	
self weight CLT	0.6	2.25	2.25
		2.25	2.25
self weight	0.6	9.35	9.35
		9.35	9.35
snow load CEN > 1000m altitude	0.8	32.57	32.57
		0.00	0.00
wind load	0.9	6.12	6.12
		0.00	0.00

Disclaimer

VEDLEGG F8.1

BJELKE 2-2

Referanse til standarder

[1] NS-EN 1995-1-1: Allmenne regler og regler for bygninger
[2] NS-EN 1990: Grunnlag for prosjektering av konstruksjoner
[3] Norsk Limtreprodusenters Forening, *Limtreboka - Beregningseksempler* (2018)
[4] NS-EN 1995-1-2: Brannteknisk dimensjonering
[5] Byggeteknisk forskrift (TEK17)
[6] SINTEF Byggforskserien, nr.421.051 *Statikkformler for bjelker* (2019)

FORUTSETNINGER OG ANTAKELSER

[1] NA.901 Klimaklasse: 1

[1] Tab. 2.1 Lastvarighetsklasse for snølast: Korttidslast
Lastvarighetsklasse for vindlasten: Øyeblikkslast

[1] NA.2.3 Partialfaktor for limtre: $\gamma_M := 1.15$

Lengde: $L := 9600\ mm$ $l_1 := 6500\ mm$ $l_2 := 2850\ mm$

Bjelketverrrsnitt: $b := 215\ mm$ $h := 585\ mm$

Lastbredde: $Lb := 3400\ mm$

[3] Tab.6 Limtre GL30c:

$\rho_m := 430\ \frac{kg}{m^3}$ $g := 9.81\ \frac{m}{s^2}$ $f_{m.k} := 30\ \frac{N}{mm^2}$

$E_{0.05} := 10800\ \frac{N}{mm^2}$ $f_{c.90k} := 2.5\ \frac{N}{mm^2}$ $f_{v.k} := 3.5\ \frac{N}{mm^2}$

$E_{mean} := 13000\ \frac{N}{mm^2}$

LASTER

Egenlast etasjeskiller: $g_{t.k} := 2.595\ \frac{kN}{m^2} \cdot Lb$ $g_{t.k} = 8.823\ \frac{kN}{m}$

Egenlast bjelke: $g_{b.k} := b \cdot h \cdot \rho_m \cdot g$ $g_{b.k} = 0.531\ \frac{kN}{m}$

Permanent last: $g_k := g_{t.k} + g_{b.k}$ $g_k = 9.354\ \frac{kN}{m}$

Nyttelast: $q_k := 2 \frac{kN}{m^2} \cdot Lb$ $q_k = 6.8 \frac{kN}{m}$

Punktlast fra fasade: $F := 1 \frac{kN}{m} \cdot Lb$ $F = 3.4\ kN$

BRUDDGRENSEKONTROLL

[2] Tab. NA.A1.1 $\psi_0 := 0.7$ $\psi_1 := 0.5$ $\psi_2 := 0.3$ (Nyttelast kategori A)

[1] (3.2) Høydefaktor: $k_h := min\left(\left(\frac{600\ mm}{h}\right)^{0.1}, 1.1\right)$ $k_h = 1.003$

Lastkombinasjoner

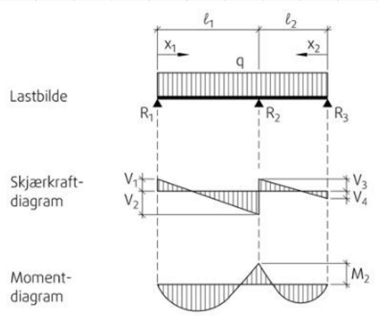
[1] Tab. 3.1 1. Egenlast og nyttelast $k_{mod1} := 0.8$

$p_{d1} := 1.2 \cdot g_k + 1.5 \cdot q_k$ $p_{d1} = 21.424 \frac{kN}{m}$

[6] Nr. 81 **Dimensjonerende krefter**

$$M_2 := \frac{\left(p_{d1} \cdot l_2^{\ 3} + p_{d1} \cdot l_1^{\ 3}\right)}{8 \cdot \left(l_1 + l_2\right)}$$

$M_2 = 85.289\ kN \cdot m$



$R_1 := \frac{-M_2}{l_1} + \frac{p_{d1} \cdot l_1}{2}$ $R_1 = 56.508\ kN$

$R_3 := \frac{-M_2}{l_2} + \frac{p_{d1} \cdot l_2}{2}$ $R_3 = 0.604\ kN$

$R_2 := p_{d1} \cdot \left(l_1 + l_2\right) - R_1 - R_3$ $R_2 = 143.206\ kN$

$V_1 := R_1$

$V_4 := R_3$

$V_2 := p_{d1} \cdot l_1 - R_1$ $\quad V_2 = 82.75\ kN$

$V_3 := p_{d1} \cdot l_2 - R_3$ $\quad V_3 = 60.455\ kN$

$x_1 := \frac{R_1}{p_{d1}}$ $\quad M_1 := R_1 \cdot x_1 - \frac{p_{d1} \cdot x_1^{\ 2}}{2}$ $\quad M_1 = 74.521\ kN \cdot m$

$x_2 := \frac{R_3}{p_{d1}}$ $\quad M_3 := R_1 \cdot x_2 - \frac{p_{d1} \cdot x_2^{\ 2}}{2}$ $\quad M_3 = 1.584\ kN \cdot m$

$V_{Ed} := \max\left(V_1, V_2, V_3, V_4\right)$ $\quad V_{Ed} = 82.75\ kN$

$M_{Ed} := \max\left(M_1, M_2, M_3\right)$ $\quad M_{Ed} = 85.289\ kN \cdot m$

Dimensjonerende fastheter

$f_{m.d} := f_{m.k} \cdot \frac{k_h \cdot k_{mod1}}{\gamma_M}$ $\quad f_{m.d} = 20.922\ \frac{N}{mm^2}$

$f_{c.90.d} := f_{c.90k} \cdot \frac{k_{mod1}}{\gamma_M}$ $\quad f_{c.90.d} = 1.739\ \frac{N}{mm^2}$

$f_{v.d} := f_{v.k} \cdot \frac{k_{mod1}}{\gamma_M}$ $\quad f_{v.d} = 2.435\ \frac{N}{mm^2}$

[1] 6.3.3 **Vipping**

Forutsetter at bjelken er gaffellagret (sikret mot rotasjon) om egen akse ved oppleggene. Videre antar en at etasjeskilleren vil sikre mot forskyvninger på tvers. En mulighet for vipping er over midtopplegg med trykk i underkant, regner konservativt den som en konsentrert last.

[1] Tab. 6.1 $\quad l_{ef} := 0.8 \cdot 6500\ mm + 2 \cdot h$ $\quad l_{ef} = 6.37\ m$

$\sigma_{m.crit} := \frac{(0.78 \cdot b^2)}{h \cdot l_{ef}} \cdot E_{0.05}$ $\sigma_{m.crit} = 104.496 \frac{N}{mm^2}$

$\lambda_{relm} := \sqrt{\frac{f_{m.k}}{\sigma_{m.crit}}}$ $\lambda_{relm} = 0.536$

$\lambda_{relm} \leq 0.75$ $k_{crit} := 1.0$

[1] 6.1.6 **Bøyekontroll**

$W := \frac{(b \cdot h^2)}{6}$ $W = (1.226 \cdot 10^7)\ mm^3$

$\sigma_{m.d} := \frac{M_{Ed}}{W}$ $\sigma_{m.d} = 6.955 \frac{N}{mm^2}$

$\frac{\sigma_{m.d}}{k_{crit} \cdot f_{m.d}} = 0.332$ < 1.0 OK

[1] 6.1.7 **Skjærkontroll**

Sprekkfaktor for limtre: $k_{cr} := 0.67$

$b_{ef} := k_{cr} \cdot b$

$\tau_d := \frac{(3 \cdot V_{Ed})}{2 \cdot b_{ef} \cdot h}$ $\tau_d = 1.473 \frac{N}{mm^2}$

$\frac{\tau_d}{f_{v.d}} = 0.605$ < 1.0 OK

[1] 6.1.5 **Trykk vinkelrett på fiberretning**

Trykk vinkelrett på fiberretningen blir ikke kontrollert i Calculatis

Søyledimensjon: $b_s := 215\ mm$ $h_s := 225\ mm$

Dimensjonerende opplagerkraft ved midtopplegg:

$F_{c.90.d} := R_2$

$A_{ef} := b_s \cdot (h_s + 60\ mm)$

$\sigma_{c.90.d} := \frac{F_{c.90.d}}{A_{ef}}$ $\sigma_{c.90.d} = 2.337\ \frac{N}{mm^2}$

$k_{c.90} := 1.75$ (Hviler på enkeltopplegg, $l_1 \geq 2 \cdot h$, limtre og $l \leq 400\ mm$)

$\frac{\sigma_{c.90.d}}{k_{c.90} \cdot f_{c.90.d}} = 0.768$ < 1.0 OK

Dimensjonerende opplagerkraft ved endeopplegg:

$F_{c.90.d.e} := R_1$

$A_{ef.e} := b_s \cdot (h_s + 30\ mm)$

$\sigma_{c.90.d.e} := \frac{F_{c.90.d.e}}{A_{ef.e}}$ $\sigma_{c.90.d} = 2.337\ \frac{N}{mm^2}$

$k_{c.90} := 1.75$ (Hviler på enkeltopplegg, $l_1 \geq 2 \cdot h$, limtre og $l \leq 400\ mm$)

$\frac{\sigma_{c.90.d.e}}{k_{c.90} \cdot f_{c.90.d}} = 0.339$ < 1.0 OK

BRUKSGRENSEKONTROLL

Nedbøyning

[1] Tab 3.2 $k_{def} := 0.6$ (Klimaklasse 1) $I := \frac{1}{12} \cdot b \cdot h^3$

Forenkler og benytter formel for en kontinuerlig bjelke med to like spenn og jevnt fordelt last. Setter lengden lik det lengste spennet. Kontrolleres i Calculatis., forventer da noe avvik.

[1] Tab 7.2 Øyeblikkelig nedbøyning

$\delta_g := \frac{1}{185} \cdot \frac{g_k \cdot {l_1}^4}{E_{mean} \cdot I}$ $\delta_g = 1.935\ mm$

$\delta_q := \frac{1}{185} \cdot \frac{q_k \cdot {l_1}^4}{E_{mean} \cdot I}$ $\delta_q = 1.407\ mm$

$w_{inst} := \delta_g + \delta_q$ $w_{inst} = 3.343\ mm$ < $\frac{l_1}{500} = 13\ mm$

Langtidsnedbøyning

$\delta_{gfin} := \delta_g \cdot (1 + k_{def})$ $\delta_{gfin} = 3.097\ mm$

$\delta_{qfin} := \delta_q \cdot (1 + \psi_2 \cdot k_{def})$ $\delta_{qfin} = 1.66\ mm$

$w_{fin} := \delta_{gfin} + \delta_{qfin}$ $w_{fin} = 4.757\ mm$ < $\frac{l_1}{300} = 21.667\ mm$

BRANNDIMENSJONERING

[4] Tab 3.1 Dimensjonerende forkullingshastighet for limtre: $\beta_n := 0.7\ \frac{mm}{min}$

[5] § 11-3 Brannklasse 2

[5] § 11-4 Brannmotstandstid R60: $t := 60\ min$

[4] 4.2.2(5) $k_{mod.fi} := 1.0$

[4] 2.3 (Merknad 2) $\gamma_{M.fi} := 1.0$

[4] 4.2.2 **Effektiv forkullingsdybde**

$k_0 := 1$ (t > 20 min)

$d_{char.n} := \beta_n \cdot t$

$d_0 := 7\ mm$

$d_{ef} := d_{char.n} + k_0 \cdot d_0$ $\qquad d_{ef} = 49\ mm$

Dimensjon effektivt resttverrsnitt

$b_{ef.fi} := b - 2 \cdot d_{ef}$ $\qquad b_{ef.fi} = 117\ mm$

$h_{ef.fi} := h - 1 \cdot d_{ef}$ $\qquad h_{ef.fi} = 536\ mm$

Dimensjonerende fastheter

$f_{m.d.fi} := 1.15 \cdot f_{m.k}$ $\qquad f_{m.d.fi} = 34.5\ \frac{N}{mm^2}$

$f_{v.d.fi} := 1.15 \cdot f_{v.k}$ $\qquad f_{v.d.fi} = 4.025\ \frac{N}{mm^2}$

Dimensjonerende krefter

(Verdier hentet fra Calculatis)

$M_{d.maks.fi} := 51\ kN \cdot m$

$V_{d.maks.fi} := 49.5\ kN$

[1] 6.3.3 **Vipping**

$\sigma_{m.crit.fi} := \frac{\left(0.78 \cdot b_{ef.fi}^{\ 2}\right)}{h_{ef.fi} \cdot l_{ef}} \cdot E_{0.05}$ $\qquad \sigma_{m.crit.fi} = 33.774\ \frac{N}{mm^2}$

$\lambda_{relm.fi} := \sqrt{\frac{f_{m.k}}{\sigma_{m.crit.fi}}}$ $\qquad \lambda_{relm.fi} = 0.942$ $\qquad 0.75 < \lambda_{relm.fi} < 1.4$

$k_{crit.fi} := 1.56 - 0.75 \cdot \lambda_{relm.fi}$ $\qquad k_{crit.fi} = 0.853$

[1] 6.1.6 **Bøyekontroll**

$$W_{fi} := \frac{\left(b_{ef.fi} \cdot h_{ef.fi}^{2}\right)}{6} \qquad W_{fi} = \left(5.602 \cdot 10^{6}\right) mm^{3}$$

$$\sigma_{m.d.fi} := \frac{M_{d.maks.fi}}{W_{fi}} \qquad \sigma_{m.d.fi} = 9.103 \ \frac{N}{mm^{2}}$$

$$\frac{\sigma_{m.d.fi}}{k_{crit.fi} \cdot f_{m.d.fi}} = 0.309$$ < 1.0 OK

[1] 6.1.7 **Skjærkontroll**

$$b_{ef} := k_{cr} \cdot b_{ef.fi}$$

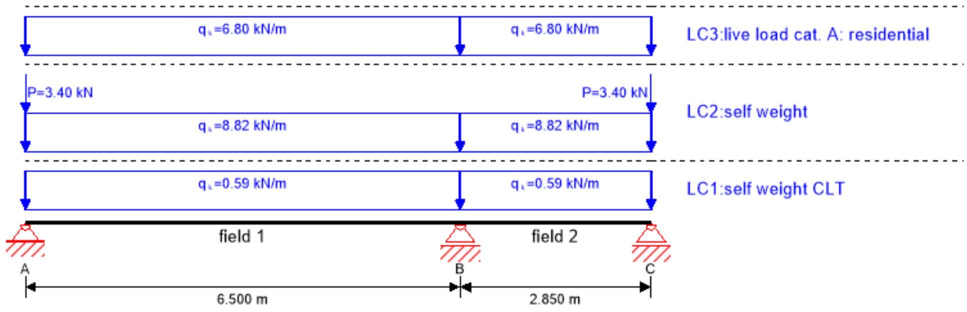
$$\tau_{d.fi} := \frac{\left(3 \cdot V_{d.maks.fi}\right)}{2 \cdot b_{ef} \cdot h_{ef.fi}}$$

$$\frac{\tau_{d.fi}}{f_{v.d.fi}} = 0.439$$ < 1 OK

Kan dermed konkludere med at bjelken beholder tilstrekkelig bæreevne ved en 60 minutters brann.

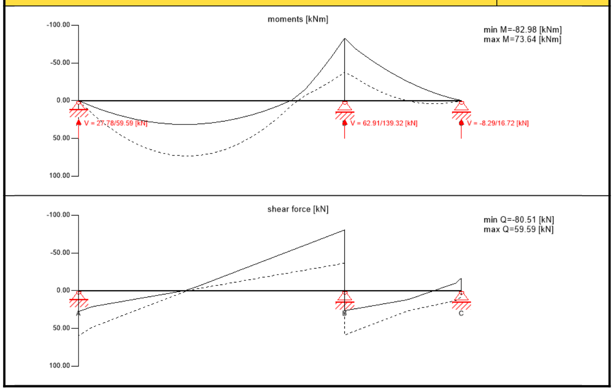
VEDLEGG F8.2

system



section: wooden beam 21.5/58.5; **material**: GL30c; **service class**: service class 1; **fire resistance class**: R 60

utilization 50 %



flexural stress analysis 32 %

$M_{y,d}$ =	-82.98	kNm	$f_{m,k}$ =	30.00	N/mm²
$N_{t,d}$ =	0.00	kN	$f_{t,k}$ =	19.50	N/mm²
$\sigma_{t,d}$ =	0.00	N/mm²	$f_{t,d}$ =	13.57	N/mm²
$\sigma_{m,y,d}$ =	6.77	N/mm² <	$f_{m,y,d}$ =	20.87	N/mm² ✓

shear stress analysis 50 %

V_d =	68.31	kN	$f_{v,k}$ =	3.50	N/mm²
$\tau_{v,d}$ =	0.81	N/mm² <	$f_{v,d}$ =	1.63	N/mm² ✓

lateral torsional buckling analysis 32 %

$M_{y,d}$ =	-82.98	kNm	$f_{m,k}$ =	30.00	N/mm²
$N_{c,d}$ =	0.00	kN	$f_{c,k}$ =	24.50	N/mm²
$\sigma_{c,d}$ =	0.00	N/mm²	$f_{c,d}$ =	17.04	N/mm²
$\sigma_{m,y,d}$ =	6.77	N/mm² <	$f_{m,y,d}$ =	20.87	N/mm² ✓

flexural stress analysis fire 26 %

$M_{y,d}$ =	-51.00	kNm	$f_{m,k}$ =	30.00	N/mm²
$N_{t,d}$ =	0.00	kN	$f_{t,k}$ =	19.50	N/mm²
$\sigma_{t,d}$ =	0.00	N/mm²	$f_{t,d}$ =	22.43	N/mm²
$\sigma_{m,y,d}$ =	9.10	N/mm² <	$f_{m,y,d}$ =	34.50	N/mm² ✓

shear stress analysis fire 38 %

V_d =	42.62	kN	$f_{v,k}$ =	3.50	N/mm²
$\tau_{v,d}$ =	1.02	N/mm² <	$f_{v,d}$ =	2.70	N/mm² ✓

lateral torsional buckling analysis fire 34 %

$M_{y,d}$ =	-51.00	kNm	$f_{m,k}$ =	30.00	N/mm²
$N_{c,d}$ =	0.00	kN	$f_{c,k}$ =	24.50	N/mm²
$\sigma_{c,d}$ =	0.00	N/mm²	$f_{c,d}$ =	28.18	N/mm²
$\sigma_{m,y,d}$ =	9.10	N/mm² <	$f_{m,y,d}$ =	34.50	N/mm² ✓

w_{inst} = w[char]

field	K_{def}	limit	w_{limit}	$w_{calc.}$	ratio
		[-]	[mm]	[mm]	
1	0.6	L/500	13.0	4.6	35 %
2	0.6	L/500	5.7	0.0	0 %

w_{fin} = w[char] + w[q.p.]*kdef

field	K_{def}	limit	w_{limit}	$w_{calc.}$	ratio
		[-]	[mm]	[mm]	
1	0.6	L/300	21.7	6.5	30 %
2	0.6	L/300	9.5	0.0	0 %

$w_{net,fin}$ = w[q.p.] + w[q.p.]*kdef

field	K_{def}	limit	w_{limit}	$w_{calc.}$	ratio
		[-]	[mm]	[mm]	
1	0.6	L/400 L/400	16.3	5.1	32 %
2	0.6	L/400 L/400	7.1	0.0	0 %

support reaction				
load case category	k_{mod}	A_V	B_V	C_V
		[kN]		
self weight CLT	0.6	1.56	3.95	0.02
		1.56	3.95	0.02
self weight	0.6	26.66	58.96	3.65
		26.66	58.96	3.65
live load cat. A: residential	0.8	18.26	45.45	8.95
		-0.32	0.00	-8.76

Disclaimer

VEDLEGG F9

AVSTIVNING AV SØYLETOPP OG GAFFELLAGRING AV BJELKENS OPPLEGG

Referanse til standarder

[1] NS-EN 1995 Prosjektering av trekonstruksjoner
[2] Norsk Limtreprodusenters Forening, *Limtreboka - Beregningseksempler* (2018)

<u>FORUTSETNINGER</u>

Bjelketverrsnitt $h := 585\ mm$ $b := 215\ mm$

Lengde (søyle + bjelke) $L := 2700\ mm$

(Verdier hentet fra Calculatis:)

Aksialkraft $N_d := 244.6\ kN$

Maks moment over søyle $M_d := 111.2\ kN \cdot m$

[1] Tab.9.2 Korreksjonsfaktor for avstvningsmotstand $k_{f.2} := 80$

Limtre styrkeklasse GL30c

$E_{0.05} := 10800\ \frac{N}{mm^2}$ $f_{m.k} := 30\ \frac{N}{mm^2}$

[1] 6.3.3 k_{crit} for den ikke-avstivende bjelken

$l_{ef} := 0.9 \cdot 13500\ mm + 2 \cdot h$

$\sigma_{m.crit} := \frac{\left(0.78 \cdot b^2\right)}{h \cdot l_{ef}} \cdot E_{0.05}$ $\sigma_{m.crit} = 49.973\ \frac{N}{mm^2}$

$\lambda_{relm} := \sqrt{\frac{f_{m.k}}{\sigma_{m.crit}}}$ $\lambda_{relm} = 0.775$

$k_{crit} := 1.56 - 0.75 \cdot \lambda_{relm}$ $k_{crit} = 0.979$

[1] 9.2.5.2 SIDEAVSTIVEDE STAVER UNDER TRYKK

Bidrag aksialkraft

[1] (9.35) $F_{h.Nd} := \frac{N_d}{k_{f.2}}$ $F_{h.Nd} = 3.058\ kN$

Bidrag bøyemoment

$N_{Md} := (1 - k_{crit}) \cdot \frac{M_d}{h}$ $N_{Md} = 4.012\ kN$

$F_{h.Md} := \frac{N_{Md}}{k_{f.2}}$ $F_{h.Md} = 0.05\ kN$

Totalt

$F_{hd} := F_{h.Nd} + F_{h.Md}$ $F_{hd} = 3.108\ kN$

Moment i "ledd-punktet"

$M_{hd} := F_{hd} \cdot \left(\frac{2.1\ m}{L}\right) \cdot h$ $M_{hd} = 1.414\ kN \cdot m$

Bidrag fra skjevstilling

[1] 10.2 $\frac{L}{500} = 5.4\ mm$

$e := 10\ mm$ (Antar avviket noe høyere tss)

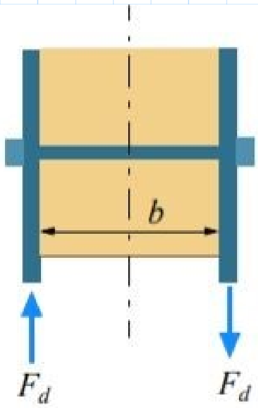
$M_{ed} := N_d \cdot e$ $M_{ed} = 2.446\ kN \cdot m$

Moment fra stabilisering og skjevstilling

$M_d := M_{hd} + M_{ed}$ $M_d = 3.86\ kN \cdot m$

Dette momentet tas opp som et kraftpar i laskeforbindelsen. Kraft i hver lask blir:

$F_d := \frac{M_d}{b}$ $F_d = 17.953\ kN$



[1] 8.2.3 BOLTEFORBINDELSE

Velger å benytte 20 mm tykke stål-lasker og Ø12 mm bolter av kvalitet 8.8

$d := 12\ mm$ $f_{uk} := 800\ \frac{N}{mm^2}$ $\rho_k := 430\ \frac{kg}{m^3}$

$k_{mod} := 0.9$

$\gamma_M := 1.3$ (Forbindelse)

$t_1 := 20\ mm$ (Velger 20 mm tykke stålplater)

Tykke eller tynn stålplate?

$0.5 \cdot d = 6\ mm$ < 20 mm

$d = 12\ mm$ < 20 mm

--> Stålplatene er tykke

Boltens flytemoment

$M_{y.rk} := 0.3 \cdot f_{uk} \cdot d^{2.6}$ $M_{y.rk} = 324.28\ kN \cdot mm$

Kapasiteten til en bolt

Det er en en-snitts forbindelse

I søylen er kraftretningen lik fiberretningen, a=0 grader

[1] (8.32) $f_{h.0.k} := 0.082 \cdot (1 - 0.01 \cdot d) \cdot \rho_k$ $f_{h.0.k} := 29.62\ \frac{N}{mm^2}$

[1] (8.10d) $F_{v.rk.0} := f_{h.0.k} \cdot t_1 \cdot d \cdot \left(\sqrt{2 + \frac{4 \cdot M_{y.rk}}{f_{h.0.k} \cdot d \cdot t_1^{\ 2}}} - 1 \right)$ $F_{v.rk.0} = 22.618\ kN$

$F_{v.Rk.0} := 1.25 \cdot F_{v.rk.0}$ $F_{v.Rk.0} = 28.273\ kN$

$F_{v.Rd.0} := F_{v.Rk.0} \cdot \frac{k_{mod}}{\gamma_M}$ $F_{v.Rd.0} = 19.573\ kN$

Antall bolter:

$n := \frac{F_d}{F_{v.Rd.0}}$ $n = 0.917$

I bjelken er kraftretningen vinkelrett på fiberretningen, a=90 grader

$a := 90$

[1] (8.33) $k_{90} := 1.35 + 0.015 \cdot 16$ $k_{90} = 1.59$

[1] (8.31) $f_{h.90.k} := \frac{f_{h.0.k}}{k_{90} \cdot \left(\sin(a)^2\right) + \cos(a)^2}$ $f_{h.90.k} = 20.128\ \frac{N}{mm^2}$

[1] (8.10d) $F_{v.rk.90} := f_{h.90.k} \cdot t_1 \cdot d \cdot \left(\sqrt{2 + 4 \cdot \frac{M_{y.rk}}{f_{h.90.k} \cdot d \cdot t_1^{\ 2}}} - 1 \right)$ $F_{v.rk.90} = 18.348\ kN$

$F_{v.Rk.90} := 1.25 \cdot F_{v.rk.90}$ $F_{v.Rk.90} = 22.935\ kN$

$F_{v.Rd.90} := F_{v.Rk.90} \cdot \frac{k_{mod}}{\gamma_M}$ $F_{v.Rd.90} = 15.878\ kN$

Antall bolter:

$n := \frac{F_d}{F_{v.Rd.90}}$ $n = 1.131$

Benytter to bolter på en rekke på både søyle og bjelke for opprettholde symmetri.

VEDLEGG G

Søyler

G. Søyler

- G1. Søyle 4-8
  - *G1.1 Mathcad*
  - *G1.2 Excel*
  - *G1.3 Calculatis*
- G2. Søyle 4-7
  - *G2.1 Mathcad*
  - *G2.2 Excel*
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- G3. Søyle 4-6
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- G4. Søyle 3-10
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- G5. Søyle 3-9
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- G6. Søyle 2-10
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- G8. Søyle 2-6
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  - *G8.3 Calculatis*
- G9. Søyle 1-10
  - *G9.1 Mathcad*
  - *G9.2 Excel*
  - *G9.3 Calculatis*
- G10. Søyle 1-9

G10.1 Mathcad

G10.2 Excel

G10.3 Calculatis

VEDLEGG G1.1

SØYLE 4-8

Referanse til standarder

[1] NS-EN 1995-1-1: Allmenne regler og regler for bygninger
[2] NS-EN 1990: Grunnlag for prosjektering av konstruksjoner
[3] NS-EN 1992-1-2: Brannteknisk dimensjonering
[4] Byggeteknisk forskrift (TEK17)
[5] Norsk Limtreprodusenters Forening, *Limtreboka - Beregningseksempler* (2018)

FORUTSETNINGER OG ANTAKELSER

[1] NA.901 — Klimaklasse: 1
[1] Tab. 2.1 — Lastvarighetsklasse for snølast: Korttidslast
Lastvarighetsklasse for vindlasten: Øyeblikkslast

[1] NA.2.3 — Partialfaktor for limtre: $\gamma_M := 1.15$

Antar søyletverrrsnitt: $b := 215\ mm$ $h := 180\ mm$

Lastbredde: $Lb := 6640\ mm$

Lengde: $L := 2100\ mm$

[5] Tab. 6 — Limtre GL30c:

$f_{m.k} := 30\ \frac{N}{mm^2}$ $f_{c.90.k} := 2.5\ \frac{N}{mm^2}$ $f_{v.k} := 3.5\ \frac{N}{mm^2}$

$f_{c.0.k} := 24.5\ \frac{N}{mm^2}$ $E_{0.05} := 10800\ \frac{N}{mm^2}$ $\rho_m := 430\ \frac{kg}{m^3}$

$g := 9.81\ \frac{m}{s^2}$

LASTER

(Verdier hente fra Calculatis)

Egenvekt tak, bjelke og fasade: $G_b := 17.42\ kN$

Egenvekt søyle: $G_s := 4.609\ \frac{kN}{m^3} \cdot b \cdot h \cdot L$ $G_s = 374.573\ N$

Permanent last: $G_k := G_b + G_s$ $G_k = 17.795\ kN$

Snølast: $S_k := 21.3\ kN$

Vindlast (fra tak): $Q_k := 8.9\ kN$

Vindlast på vegg:

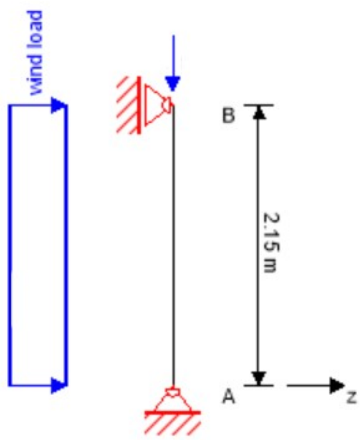
$q_{kast} := 1.34\ \frac{kN}{m^2}$ $C_A := 1.2$ $C_D := 0.724$

$q_{vind1} := q_{kast} \cdot (C_A + 0.2) \cdot Lb$ $q_{vind1} = 12.457\ \frac{kN}{m}$

$q_{vind2} := q_{kast} \cdot (C_D + 0.3) \cdot Lb$ $q_{vind2} = 9.111\ \frac{kN}{m}$

$q_k := \max(q_{vind1}, q_{vind2})$ $q_k = 12.457\ \frac{kN}{m}$

LASTKOMBINASJONER



1. Kun egenlast $k_{mod} := 0.6$

$G_{f1} := 1.35 \cdot G_k$ $G_{f1} = 24.023\ kN$

2. Alle laster $k_{mod} := 1.1$

a) Egenlast dominerende:

$G_{f2} := 1.35 \cdot G_k$ $G_{f2} = 24.023\ kN$

$S_{f2} := 1.05 \cdot S_k$ $S_{f2} = 22.365\ kN$

$Q_{f2} := 1.05 \cdot Q_k$ $Q_{f2} = 9.345\ kN$

$q_{f2} := 1.05 \cdot q_k$ $q_{f2} = 13.079\ \frac{kN}{m}$

b) Snølast dominerende:

$G_{f3} := 1.2 \cdot G_k$ $\quad G_{f3} = 21.353\ kN$

$S_{f3} := 1.5 \cdot S_k$ $\quad S_{f3} = 31.95\ kN$

$Q_{f3} := 1.05 \cdot Q_k$ $\quad Q_{f3} = 9.345\ kN$

$q_{f3} := 1.05 \cdot q_k$ $\quad q_{f3} = 13.079\ \frac{kN}{m}$

c) Vindlast dominerende:

$G_{f4} := 1.2 \cdot G_k$ $\quad G_{f4} = 21.353\ kN$

$S_{f4} := 1.05 \cdot S_k$ $\quad S_{f4} = 22.365\ kN$

$Q_{f4} := 1.5 \cdot Q_k$ $\quad Q_{f4} = 13.35\ kN$

$q_{f4} := 1.5 \cdot q_k$ $\quad q_{f4} = 18.685\ \frac{kN}{m}$

3. Kun egenlast og snølast $\quad k_{mod} := 0.9$

a) Egenlast dominerende:

$G_{f5} := 1.35 \cdot G_k$ $\quad G_{f5} = 24.023\ kN$

$S_{f5} := 1.05 \cdot S_k$ $\quad S_{f5} = 22.365\ kN$

b) Snølast dominerende:

$G_{f6} := 1.2 \cdot G_k$ $\quad G_{f6} = 21.353\ kN$

$S_{f6} := 1.5 \cdot S_k$ $\quad S_{f6} = 31.95\ kN$

[1] (3.2) Høydefaktor: $k_h := min\left(\left(\frac{600\ mm}{h}\right)^{0.1}, 1.1\right)$ $\quad k_h = 1.1$

KNEKKING

y-y: Knekklengde: $l_{ky} := 2150\ mm + 450\ mm$ (søyle + bjelkehøyde)

Treghetsradius: $i_y := 0.29 \cdot h$ $i_y = 52.2\ mm$

Slankhet: $\lambda_y := \frac{l_{ky}}{i_y}$ $\lambda_y = 49.808$

[1] (6.21) $\lambda_{rel.y} := \frac{\lambda_y}{\pi} \cdot \sqrt{\frac{f_{c.0.k}}{E_{0.05}}}$ $\lambda_{rel.y} = 0.755$

[1] (6.29) $\beta_c := 0.1$ (limtre)

[1] (6.27) $k_y := 0.5 \cdot \left(1 + \beta_c \cdot \left(\lambda_{rel.y} - 0.3\right) + \lambda_{rel.y}^{\ 2}\right)$ $k_y = 0.808$

$k_{cy} := \frac{1}{k_y + \sqrt{k_y^{\ 2} - \lambda_{rel.y}^{\ 2}}}$ $k_{cy} = 0.913$

z-z: Knekklengde: $l_{kz} := 2150\ mm + 450\ mm$

Treghetsradius: $i_z := 0.29 \cdot b$ $i_z = 62.35\ mm$

Slankhet: $\lambda_z := \frac{l_{kz}}{i_z}$ $\lambda_z = 41.7$

[1] (6.21) $\lambda_{rel.z} := \frac{\lambda_z}{\pi} \cdot \sqrt{\frac{f_{c.0.k}}{E_{0.05}}}$ $\lambda_{rel.z} = 0.632$

[1] (6.29) $\beta_c := 0.1$ (limtre)

[1] (6.27) $k_z := 0.5 \cdot \left(1 + \beta_c \cdot \left(\lambda_{rel.z} - 0.3\right) + \lambda_{rel.z}^{\ 2}\right)$ $k_z = 0.716$

$k_{cz} := \frac{1}{k_z + \sqrt{k_z^{\ 2} - \lambda_{rel.z}^{\ 2}}}$ $k_{cz} = 0.949$

BRANNDIMENSJONERING

[3] Tab 3.1 Dimensjonerende forkullingshastighet for limtre: $\beta_n := 0.7 \frac{mm}{min}$

[4] § 11-3 Brannklasse 2

[4] § 11-4 Brannmotstandstid R60: $t := 60\ min$

[2] Tab. NA.A1.1

$\psi_{0.s} := 0.7$ $\psi_{1.s} := 0.5$ $\psi_{2.s} := 0.2$

$\psi_{0.v} := 0.6$ $\psi_{1.v} := 0.2$ $\psi_{2.v} := 0$

$N_{fi} := G_k + \psi_{1.v} \cdot Q_k + \psi_{2.s} \cdot S_k$ $N_{fi} = 23.835\ kN$

[3] 4.2.2 **Effektiv forkullingsdybde**

$t > 20\ min$ $k_0 := 1.0$

$d_{char.n} := \beta_n \cdot t$

$d_0 := 7\ mm$

$d_{ef} := d_{char.n} + k_0 \cdot d_0$ $d_{ef} = 49\ mm$

Dimensjon effektivt resttverrsnitt

$b_{ef.fi} := b - 2 \cdot d_{ef}$ $b_{ef.fi} = 117\ mm$

$h_{ef.fi} := h - 2 \cdot d_{ef}$ $h_{ef.fi} = 82\ mm$

Dimensjonerende fasthet

$f_{c.0.d.fi} := 1.15 \cdot f_{c.0.k}$ $f_{c.0.d.fi} = 28.175 \frac{N}{mm^2}$

$f_{m.d.fi} := 1.15 \cdot f_{m.k}$ $f_{m.d.fi} = 34.5 \frac{N}{mm^2}$

Knekking - brann

y-y: Treghetsradius: $i_{y.fi} := 0.29 \cdot h_{ef.fi}$ $i_{y.fi} = 23.78\ mm$

Slankhet: $\lambda_{y.fi} := \dfrac{l_{ky}}{i_{y.fi}}$ $\lambda_{y.fi} = 109.336$

[1] (6.21) $\lambda_{rel.y.fi} := \dfrac{\lambda_{y.fi}}{\pi} \cdot \sqrt{\dfrac{f_{c.0.k}}{E_{0.05}}}$ $\lambda_{rel.y.fi} = 1.658$

[1] (6.29) $\beta_c := 0.1$ (limtre)

[1] (6.27) $k_{y.fi} := 0.5 \cdot \left(1 + \beta_c \cdot \left(\lambda_{rel.y.fi} - 0.3\right) + \lambda_{rel.y.fi}^{\ 2}\right)$ $k_{y.fi} = 1.942$

$k_{cy.fi} := \dfrac{1}{k_{y.fi} + \sqrt{k_{y.fi}^{\ 2} - \lambda_{rel.y.fi}^{\ 2}}}$ $k_{cy.fi} = 0.339$

z-z: Treghetsradius: $i_{z.fi} := 0.29 \cdot b_{ef.fi}$ $i_{z.fi} = 33.93\ mm$

Slankhet: $\lambda_{z.fi} := \dfrac{l_{kz}}{i_{z.fi}}$ $\lambda_{z.fi} = 76.628$

[1] (6.21) $\lambda_{rel.z.fi} := \dfrac{\lambda_{z.fi}}{\pi} \cdot \sqrt{\dfrac{f_{c.0.k}}{E_{0.05}}}$ $\lambda_{rel.z.fi} = 1.162$

[1] (6.29) $\beta_c := 0.1$ (limtre)

[1] (6.27) $k_{z.fi} := 0.5 \cdot \left(1 + \beta_c \cdot \left(\lambda_{rel.z.fi} - 0.3\right) + \lambda_{rel.z.fi}^{\ 2}\right)$ $k_{z.fi} = 1.218$

$k_{cz.fi} := \dfrac{1}{k_{z.fi} + \sqrt{k_{z.fi}^{\ 2} - \lambda_{rel.z.fi}^{\ 2}}}$ $k_{cz.fi} = 0.632$

[1] 6.1.6 **Bøyespenning**

$$W := \frac{1}{6} \cdot b_{ef.fi} \cdot h_{ef.fi}^{2}$$

$$M_{y.fi} := \frac{q_k \cdot \psi_{1.v} \cdot L^2}{8} \qquad M_{y.fi} = 1.373 \ kN \cdot m$$

$$\sigma_{m.y.fi} := \frac{M_{y.fi}}{W} \qquad \sigma_{m.y.fi} = 10.474 \ \frac{N}{mm^2}$$

$$\frac{\sigma_{m.y.fi}}{f_{m.d.fi}} = 0.304 \quad < 1.0 \quad \text{OK}$$

[1] 6.1.4 **Aksialspenning**

$$A_{fi} := b_{ef.fi} \cdot h_{ef.fi}$$

$$\sigma_{c.0.d.fi} := \frac{N_{fi}}{A_{fi}} \qquad \sigma_{c.0.d.fi} = 2.484 \ \frac{N}{mm^2}$$

$$\frac{\sigma_{c.0.d.fi}}{k_{cz.fi} \cdot f_{c.0.d.fi}} = 0.14 \quad < 1.0 \quad \text{OK}$$

Kombinert virkning

[1] 6.1.6 (2) $k_m := 0.7$

[1] (6.23) $$\frac{\sigma_{c.0.d.fi}}{k_{cy.fi} \cdot f_{c.0.d.fi}} + \frac{\sigma_{m.y.fi}}{f_{m.d.fi}} = 0.564 \quad < 1.0 \quad \text{OK}$$

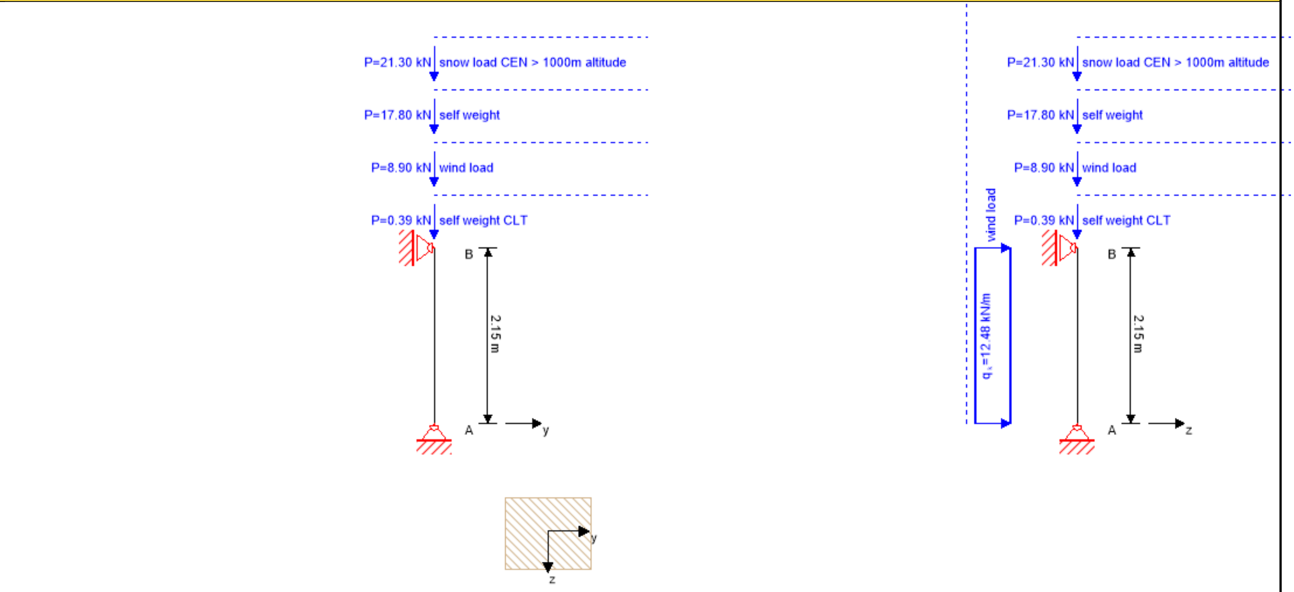
[1] (6.24) $$\frac{\sigma_{c.0.d.fi}}{k_{cz.fi} \cdot f_{c.0.d.fi}} + k_m \cdot \frac{\sigma_{m.y.fi}}{f_{m.d.fi}} = 0.352 \quad < 1.0 \quad \text{OK}$$

VEDLEGG G1.2

Søyle 4-8							
L	2,15		kcz	0,913		kh	1,1
b	215		kcy	0,949		W	1161000
h	180		fcok	24,5		km	0,7
A	38700		Ym	1,15			
			fmk	30			

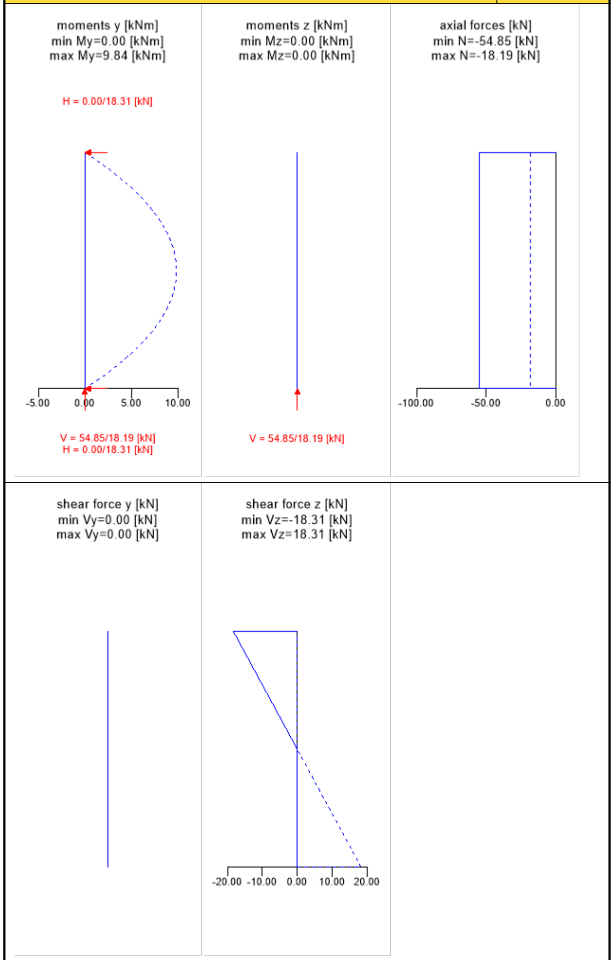
Kombinasjon	1	2a	2b	2c	3a	3b
G	17,80	17,80	17,80	17,80	17,80	17,80
γG	1,35	1,35	1,20	1,20	1,35	1,20
S		21,30	21,30	21,30	21,30	21,30
γS		1,05	1,50	1,05	1,05	1,50
Q		8,90	8,90	8,90		
γQ		1,05	1,05	1,50		
q		12,46	12,46	12,46		
γq		1,05	1,05	1,50		
Nf	24,03	68,82	62,66	57,08	46,40	53,31
qf	0,00	13,08	13,08	18,69	0,00	0,00
Mf	0,00	7,56	7,56	10,80	0,00	0,00
σN	0,62	1,78	1,62	1,47	1,20	1,38
σM	0,00	6,51	6,51	9,30	0,00	0,00
kmod	0,6	1,1	1,1	1,1	0,9	0,9
fcod	12,78	23,43	23,43	23,43	19,17	19,17
fmyd	17,22	31,57	31,57	31,57	25,83	21,09
UF (6.23)	0,05	0,29	0,28	**0,36**	0,07	0,08
UF (6.24)	0,05	0,23	0,22	0,28	0,07	0,07

system



section: wooden beam 21.5/18; **material**: GL30c; **service class**: service class 1; **fire resistance class**: R 60

utilization	51 %



flexural stress analysis						37 %
$M_{y,d}$ =	9.84	kNm	$f_{m,k}$ =	30.00	N/mm²	
$N_{c,d}$ =	-54.85	kN	$f_{c,k}$ =	24.50	N/mm²	
$\sigma_{c,d}$ =	1.42	N/mm²	$f_{c,d}$ =	19.17	N/mm²	
$\sigma_{m,y,d}$ =	8.48	N/mm²<	$f_{m,y,d}$ =	23.48	N/mm²	✓

shear stress analysis Y						0 %
V_d =	0.00	kN	$f_{v,k}$ =	3.50	N/mm²	
$\tau_{v,d}$ =	0.00	N/mm²<	$f_{v,d}$ =	1.84	N/mm²	✓

shear stress analysis Z						39 %
V_d =	18.31	kN	$f_{v,k}$ =	3.50	N/mm²	
$\tau_{v,d}$ =	0.71	N/mm²<	$f_{v,d}$ =	1.84	N/mm²	✓

shear stress analysis combined						15 %
$V_{y,d}$ =	0.00	kN	$V_{z,d}$ =	18.31	kN	
$\tau_{v,y,d}$ =	0.00	N/mm²	$\tau_{v,z,d}$ =	0.71	N/mm²	
			ratio =	15 %		✓

lateral torsional buckling analysis						9 %
$M_{y,d}$ =	0.00	kNm	$f_{m,k}$ =	30.00	N/mm²	
$N_{c,d}$ =	-58.71	kN	$f_{c,k}$ =	24.50	N/mm²	
$\sigma_{c,d}$ =	1.52	N/mm²	$f_{c,d}$ =	19.17	N/mm²	
$\sigma_{m,y,d}$ =	0.00	N/mm²<	$f_{m,y,d}$ =	23.48	N/mm²	✓

buckling analysis						44 %
$M_{y,d}$ =	9.84	kNm	$f_{m,k}$ =	30.00	N/mm²	
$N_{c,d}$ =	-54.85	kN	$f_{c,k}$ =	24.50	N/mm²	
$\sigma_{c,d}$ =	1.42	N/mm²	$f_{c,d}$ =	19.17	N/mm²	
$\sigma_{m,y,d}$ =	8.48	N/mm²	$f_{m,y,d}$ =	23.48	N/mm²	
$\sigma_{m,z,d}$ =	0.00	N/mm²<	$f_{m,z,d}$ =	23.48	N/mm²	✓

flexural stress analysis fire						33 %
$M_{y,d}$ =	1.44	kNm	$f_{m,k}$ =	30.00	N/mm²	
$N_{c,d}$ =	-24.23	kN	$f_{c,k}$ =	24.50	N/mm²	
$\sigma_{c,d}$ =	2.53	N/mm²	$f_{c,d}$ =	28.18	N/mm²	
$\sigma_{m,y,d}$ =	11.00	N/mm²<	$f_{m,y,d}$ =	34.50	N/mm²	✓

shear stress analysis Y fire						0 %
V_d =	0.00	kN	$f_{v,k}$ =	3.50	N/mm²	
$\tau_{v,d}$ =	0.00	N/mm²<	$f_{v,d}$ =	2.70	N/mm²	✓

shear stress analysis Z fire						16 %
V_d =	2.68	kN	$f_{v,k}$ =	3.50	N/mm²	
$\tau_{v,d}$ =	0.42	N/mm²<	$f_{v,d}$ =	2.70	N/mm²	✓

shear stress analysis combined fire						2 %
$V_{y,d}$ =	0.00	kN	$V_{z,d}$ =	2.68	kN	
$\tau_{v,y,d}$ =	0.00	N/mm²	$\tau_{v,z,d}$ =	0.42	N/mm²	
			ratio =	2 %		✓

lateral torsional buckling analysis fire						32 %
$M_{y,d}$ =	0.00	kNm	$f_{m,k}$ =	30.00	N/mm²	
$N_{c,d}$ =	-28.84	kN	$f_{c,k}$ =	24.50	N/mm²	
$\sigma_{c,d}$ =	3.01	N/mm²	$f_{c,d}$ =	28.18	N/mm²	
$\sigma_{m,y,d}$ =	0.00	N/mm²<	$f_{m,y,d}$ =	34.50	N/mm²	✓

buckling analysis fire						51 %
$M_{y,d}$ =	1.44	kNm	$f_{m,k}$ =	30.00	N/mm²	
$N_{c,d}$ =	-24.23	kN	$f_{c,k}$ =	24.50	N/mm²	
$\sigma_{c,d}$ =	2.53	N/mm²	$f_{c,d}$ =	28.18	N/mm²	
$\sigma_{m,y,d}$ =	11.00	N/mm²	$f_{m,y,d}$ =	34.50	N/mm²	
$\sigma_{m,z,d}$ =	0.00	N/mm²<	$f_{m,z,d}$ =	34.50	N/mm²	✓

support reaction

load case category	k_{mod}	A_y	A_z	B_x	B_y	B_z
		[kN]	[kN]	[kN]	[kN]	[kN]
self weight CLT	0.6	0.00	0.00	0.39	0.00	0.00
		0.00	0.00	0.39	0.00	0.00
wind load	0.9	0.00	13.42	0.00	0.00	13.42
		0.00	0.00	8.90	0.00	0.00
self weight	0.6	0.00	0.00	17.80	0.00	0.00
		0.00	0.00	17.80	0.00	0.00
snow load CEN > 1000m altitude	0.8	0.00	0.00	0.00	0.00	0.00
		0.00	0.00	21.30	0.00	0.00

Disclaimer

The software was created to assist engineers in their daily business. The software is an engineering software that is dealing with a very complex matter of structural analysis and building physics analysis. Therefore, this software shall only be operated by skilled, experienced engineers, with a deep understanding of structural engineering and building physics related to timber structures. The user of the software is obliged to check all input values, no matter if they were given by the user or given by default by the software and all results for plausibility.
The use of the results of the software should not be relied upon as the basis for any decision or action. Any use of results of the software is only allowed, if the results have been verified and approved regarding completeness and correctness by a project structural/building physics engineer. The user has the possibility to make print-outs from the software. Any modification of those are not allowed.
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VEDLEGG G2.1

SØYLE 4-7

Referanse til standarder

[1] NS-EN 1995-1-1: Allmenne regler og regler for bygninger
[2] NS-EN 1990: Grunnlag for prosjektering av konstruksjoner
[3] NS-EN 1992-1-2: Brannteknisk dimensjonering
[4] Byggeteknisk forskrift (TEK17)
[5] Norsk Limtreprodusenters Forening, *Limtreboka - Beregningseksempler* (2018)

FORUTSETNINGER OG ANTAKELSER

[1] NA.901 Klimaklasse: 1
[1] Tab. 2.1 Lastvarighetsklasse for snølast: Korttidslast
Lastvarighetsklasse for vindlasten: Øyeblikkslast

[1] NA.2.3 Partialfaktor for limtre: $\gamma_M := 1.15$

Antar søyletverrrsnitt: $b := 215\ mm$ $h := 180\ mm$

Lengde: $L := 2150\ mm$

[5] Tab. 6 Limtre GL30c:

$f_{m.k} := 30\ \frac{N}{mm^2}$ $f_{c.90} := 2.5\ \frac{N}{mm^2}$ $f_{v.k} := 3.5\ \frac{N}{mm^2}$

$f_{c.0.k} := 24.5\ \frac{N}{mm^2}$ $E_{0.05} := 10800\ \frac{N}{mm^2}$ $\rho_m := 430\ \frac{kg}{m^3}$

$g := 9.81\ \frac{m}{s^2}$

LASTER

(Verdier hentet fra Calculatis)

Egenvekt tak og bjelke: $G_{tb} := 31.76\ kN$

Egenvekt søyle: $G_s := 4.609\ \frac{kN}{m^3} \cdot b \cdot h \cdot L$ $G_s = 383.492\ N$

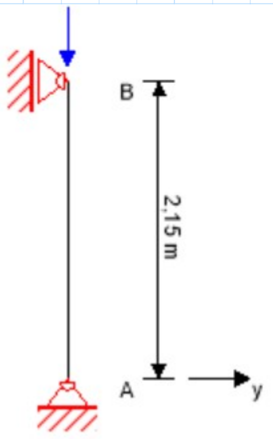
Permanent last: $G_k := G_{tb} + G_s$ $G_k = 32.143\ kN$

Snølast: $S_k := 43.73\ kN$

Vindlast: $Q_k := 18.3\ kN$

LASTKOMBINASJONER

1. Kun egenlast $k_{mod} := 0.6$



$G_{f1} := 1.35 \cdot G_k$ $G_{f1} = 43.394\ kN$

2. Alle laster $k_{mod} := 1.1$

a) Egenlast dominerende:

$G_{f2} := 1.35 \cdot G_k$ $G_{f2} = 43.394\ kN$

$S_{f2} := 1.05 \cdot S_k$ $S_{f2} = 45.917\ kN$

$Q_{f2} := 1.05 \cdot Q_k$ $Q_{f2} = 19.215\ kN$

b) Snølast dominerende:

$G_{f3} := 1.2 \cdot G_k$ $G_{f3} = 38.572\ kN$

$S_{f3} := 1.5 \cdot S_k$ $S_{f3} = 65.595\ kN$

$Q_{f3} := 1.05 \cdot Q_k$ $Q_{f3} = 19.215\ kN$

c) Vindlast dominerende:

$G_{f4} := 1.2 \cdot G_k$ $G_{f4} = 38.572\ kN$

$S_{f4} := 1.05 \cdot S_k$ $S_{f4} = 45.917\ kN$

$Q_{f4} := 1.5 \cdot Q_k$ $Q_{f4} = 27.45\ kN$

3. Kun egenlast og snølast $k_{mod} := 0.9$

a) Egenlast dominerende:

$G_{f5} := 1.35 \cdot G_k$ $G_{f5} = 43.394\ kN$

$S_{f5} := 1.05 \cdot S_k$ $S_{f5} = 45.917\ kN$

b) Snølast dominerende:

$G_{f6} := 1.2 \cdot G_k$ $G_{f6} = 38.572\ kN$

$S_{f6} := 1.5 \cdot S_k$ $S_{f6} = 65.595\ kN$

KNEKKING

y-y: Knekklengde: $l_{ky} := 2150\ mm + 450\ mm$ (Søyle + bjelkehøyde)

Treghetsradius: $i_y := 0.29 \cdot h$ $i_y = 52.2\ mm$

Slankhet: $\lambda_y := \frac{l_{ky}}{i_y}$ $\lambda_y = 49.808$

[1] (6.21) $\lambda_{rel.y} := \frac{\lambda_y}{\pi} \cdot \sqrt{\frac{f_{c.0.k}}{E_{0.05}}}$ $\lambda_{rel.y} = 0.755$

[1] (6.29) $\beta_c := 0.1$ (limtre)

[1] (6.27) $k_y := 0.5 \cdot \left(1 + \beta_c \cdot \left(\lambda_{rel.y} - 0.3\right) + \lambda_{rel.y}^{\ 2}\right)$ $k_y = 0.808$

$k_{cy} := \frac{1}{k_y + \sqrt{k_y^{\ 2} - \lambda_{rel.y}^{\ 2}}}$ $k_{cy} = 0.913$

z-z: Knekklengde: $l_{kz} := 2150\ mm + 450\ mm$ (Søyle + bjelkehøyde)

Treghetsradius: $i_z := 0.29 \cdot b$ $i_z = 62.35\ mm$

Slankhet: $\lambda_z := \dfrac{l_{kz}}{i_z}$ $\lambda_z = 41.7$

[1] (6.21) $\lambda_{rel.z} := \dfrac{\lambda_z}{\pi} \cdot \sqrt{\dfrac{f_{c.0.k}}{E_{0.05}}}$ $\lambda_{rel.z} = 0.632$

[1] (6.29) $\beta_c := 0.1$ (limtre)

[1] (6.27) $k_z := 0.5 \cdot \left(1 + \beta_c \cdot \left(\lambda_{rel.z} - 0.3\right) + \lambda_{rel.z}^{\ 2}\right)$ $k_z = 0.716$

$k_{cz} := \dfrac{1}{k_z + \sqrt{k_z^{\ 2} - \lambda_{rel.z}^{\ 2}}}$ $k_{cz} = 0.949$

BRANNDIMENSJONERING

[3] Tab 3.1 Dimensjonerende forkullingshastighet for limtre: $\beta_n := 0.7\ \dfrac{mm}{min}$

[4] § 11-3 Brannklasse 2

[4] § 11-4 Brannmotstandstid R60: $t := 60\ min$

[2] Tab. NA.A1.1

$\psi_{0.s} := 0.7$ $\psi_{1.s} := 0.5$ $\psi_{2.s} := 0.2$

$\psi_{0.v} := 0.6$ $\psi_{1.v} := 0.2$ $\psi_{2.v} := 0$

$\psi_{0.n} := 0.7$ $\psi_{1.n} := 0.5$ $\psi_{2.n} := 0.3$

$N_{fi} := G_k + \psi_{1.s} \cdot S_k + \psi_{2.v} \cdot Q_k$ $N_{fi} = 54.008\ kN$

[3] 4.2.2 **Effektiv forkullingsdybde**

$t > 20\ min$ $k_0 := 1.0$

$d_{char.n} := \beta_n \cdot t$

$d_0 := 7\ mm$

$d_{ef} := d_{char.n} + k_0 \cdot d_0$ $\qquad d_{ef} = 49\ mm$

Dimensjon effektivt resttverrsnitt

$b_{ef.fi} := b - 2 \cdot d_{ef}$ $\qquad b_{ef.fi} = 117\ mm$

$h_{ef.fi} := h - 2 \cdot d_{ef}$ $\qquad h_{ef.fi} = 82\ mm$

Dimensjonerende fasthet

$f_{c.0.d.fi} := 1.15 \cdot f_{c.0.k}$ $\qquad f_{c.0.d.fi} = 28.175\ \frac{N}{mm^2}$

$f_{m.d.fi} := 1.15 \cdot f_{m.k}$ $\qquad f_{m.d.fi} = 34.5\ \frac{N}{mm^2}$

Knekking - brann

y-y: Treghetsradius: $i_{y.fi} := 0.29 \cdot h_{ef.fi}$ $\qquad i_{y.fi} = 23.78\ mm$

Slankhet: $\lambda_{y.fi} := \frac{l_{ky}}{i_{y.fi}}$ $\qquad \lambda_{y.fi} = 109.336$

[1] (6.21) $\lambda_{rel.y.fi} := \frac{\lambda_{y.fi}}{\pi} \cdot \sqrt{\frac{f_{c.0.k}}{E_{0.05}}}$ $\qquad \lambda_{rel.y.fi} = 1.658$

[1] (6.29) $\beta_c := 0.1$ (limtre)

[1] (6.27) $k_{y.fi} := 0.5 \cdot \left(1 + \beta_c \cdot \left(\lambda_{rel.y.fi} - 0.3\right) + \lambda_{rel.y.fi}^2\right)$ $\qquad k_{y.fi} = 1.942$

$k_{cy.fi} := \frac{1}{k_{y.fi} + \sqrt{k_{y.fi}^2 - \lambda_{rel.y.fi}^2}}$ $\qquad k_{cy.fi} = 0.339$

z-z: Treghetsradius: $i_{z.fi} := 0.29 \cdot b_{ef.fi}$ $i_{z.fi} = 33.93\ mm$

Slankhet: $\lambda_{z.fi} := \frac{l_{kz}}{i_{z.fi}}$ $\lambda_{z.fi} = 76.628$

[1] (6.21) $\lambda_{rel.z.fi} := \frac{\lambda_{z.fi}}{\pi} \cdot \sqrt{\frac{f_{c.0.k}}{E_{0.05}}}$ $\lambda_{rel.z.fi} = 1.162$

[1] (6.29) $\beta_c := 0.1$ (limtre)

[1] (6.27) $k_{z.fi} := 0.5 \cdot \left(1 + \beta_c \cdot \left(\lambda_{rel.z.fi} - 0.3\right) + \lambda_{rel.z.fi}^{\ 2}\right)$ $k_{z.fi} = 1.218$

$k_{cz.fi} := \frac{1}{k_{z.fi} + \sqrt{k_{z.fi}^{\ 2} - \lambda_{rel.z.fi}^{\ 2}}}$ $k_{cz.fi} = 0.632$

[1] 6.1.4 **Aksialspenning**

$A_{fi} := b_{ef.fi} \cdot h_{ef.fi}$

$\sigma_{c.0.d.fi} := \frac{N_{fi}}{A_{fi}}$ $\sigma_{c.0.d.fi} = 5.629\ \frac{N}{mm^2}$

$\frac{\sigma_{c.0.d.fi}}{k_{cy.fi} \cdot f_{c.0.d.fi}} = 0.59$ < 1.0 OK

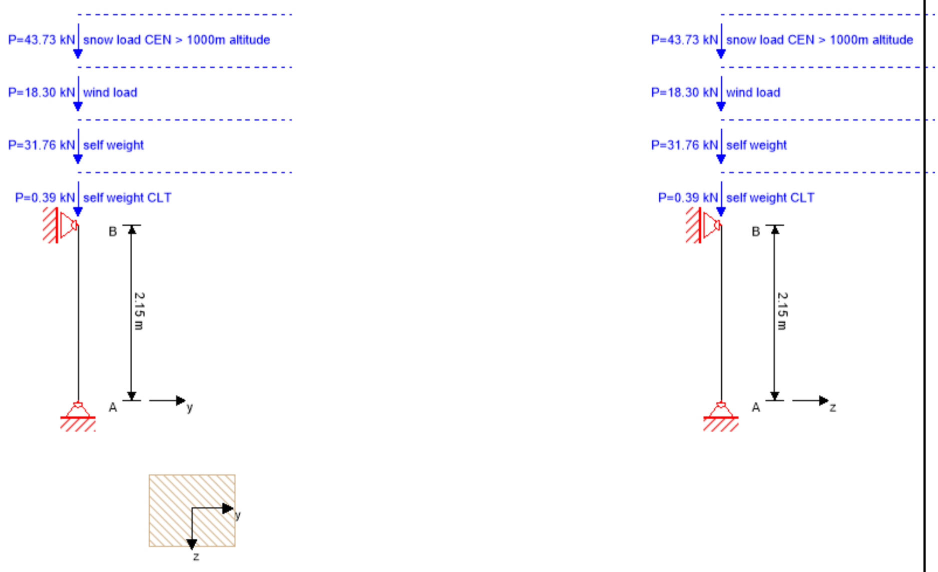
VEDLEGG G2.2

Søyle 4-7				
L	2,15		kcz	0,949
b	215		kcy	0,913
h	180		fcok	24,5
A	38700		Ym	1,15
W	1161000		fmk	30

Kombinasjon	1	2a	2b	2c	3a	3b
G	32,14	32,14	32,14	32,14	32,14	32,14
γG	1,35	1,35	1,20	1,20	1,35	1,20
S		43,73	43,73	43,73	43,73	43,73
γS		1,05	1,50	1,05	1,05	1,50
Q		18,30	18,30	18,30		
γQ		1,05	1,05	1,50		
Nf	43,39	108,52	123,38	111,93	89,31	104,16
σN	1,12	2,80	3,19	2,89	2,31	2,69
kmod	0,6	1,1	1,1	1,1	0,9	0,9
fcod	12,78	23,43	23,43	23,43	19,17	19,17
UF (6.23)	0,10	0,13	0,15	0,14	0,13	**0,15**
UF (6.24)	0,09	0,13	0,14	0,13	0,13	**0,15**

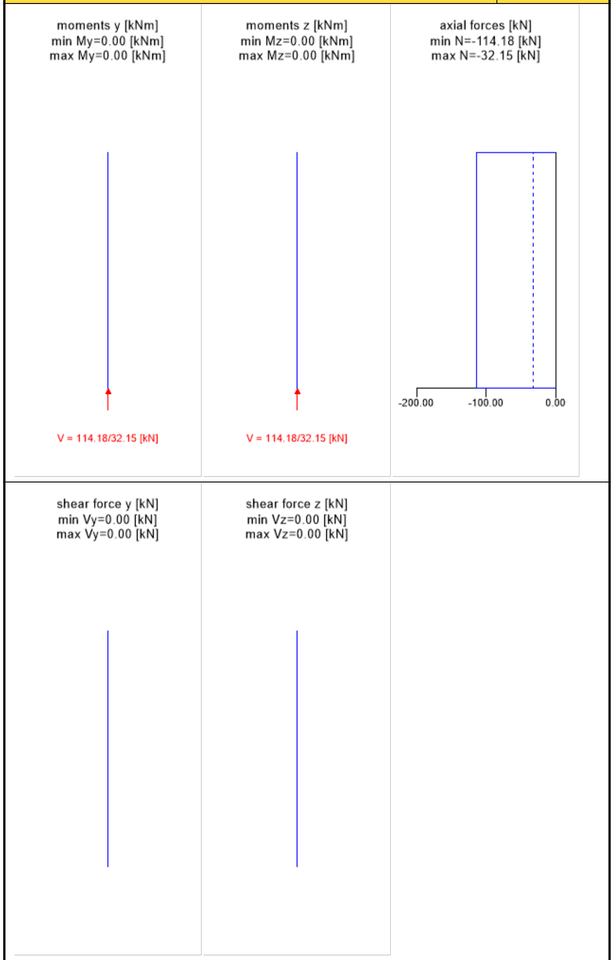
VEDLEGG G2.3

system



section: wooden beam 21.5/18; **material**: GL30c; **service class**: service class 1; **fire resistance class**: R 60

utilization 60 %



flexural stress analysis 2 %

$M_{y,d}$ =	0.00	kNm	$f_{m,k}$ =	30.00	N/mm²
$N_{c,d}$ =	-114.18	kN	$f_{c,k}$ =	24.50	N/mm²
$\sigma_{c,d}$ =	2.95	N/mm²	$f_{c,d}$ =	19.17	N/mm²
$\sigma_{m,y,d}$ =	0.00	N/mm² <	$f_{m,y,d}$ =	23.48	N/mm² ✓

shear stress analysis Y 0 %

V_d =	0.00	kN	$f_{v,k}$ =	3.50	N/mm²
$\tau_{v,d}$ =	0.00	N/mm² <	$f_{v,d}$ =	1.84	N/mm² ✓

shear stress analysis Z 0 %

V_d =	0.00	kN	$f_{v,k}$ =	3.50	N/mm²
$\tau_{v,d}$ =	0.00	N/mm² <	$f_{v,d}$ =	1.84	N/mm² ✓

shear stress analysis combined 0 %

$V_{y,d}$ =	0.00	kN	$V_{z,d}$ =	0.00	kN
$\tau_{v,y,d}$ =	0.00	N/mm²	$\tau_{v,z,d}$ =	0.00	N/mm²
			ratio =	0 %	✓

lateral torsional buckling analysis 17 %

$M_{y,d}$ =	0.00	kNm	$f_{m,k}$ =	30.00	N/mm²
$N_{c,d}$ =	-114.18	kN	$f_{c,k}$ =	24.50	N/mm²
$\sigma_{c,d}$ =	2.95	N/mm²	$f_{c,d}$ =	19.17	N/mm²
$\sigma_{m,y,d}$ =	0.00	N/mm² <	$f_{m,y,d}$ =	23.48	N/mm² ✓

buckling analysis 16 %

$M_{y,d}$ =	0.00	kNm	$f_{m,k}$ =	30.00	N/mm²
$N_{c,d}$ =	-114.18	kN	$f_{c,k}$ =	24.50	N/mm²
$\sigma_{c,d}$ =	2.95	N/mm²	$f_{c,d}$ =	19.17	N/mm²
$\sigma_{m,y,d}$ =	0.00	N/mm²	$f_{m,y,d}$ =	23.48	N/mm²
$\sigma_{m,z,d}$ =	0.00	N/mm² <	$f_{m,z,d}$ =	23.48	N/mm² ✓

flexural stress analysis fire 4 %

$M_{y,d}$ =	0.00	kNm	$f_{m,k}$ =	30.00	N/mm²
$N_{c,d}$ =	-54.02	kN	$f_{c,k}$ =	24.50	N/mm²
$\sigma_{c,d}$ =	5.63	N/mm²	$f_{c,d}$ =	28.18	N/mm²
$\sigma_{m,y,d}$ =	0.00	N/mm² <	$f_{m,y,d}$ =	34.50	N/mm² ✓

shear stress analysis Y fire 0 %

V_d =	0.00	kN	$f_{v,k}$ =	3.50	N/mm²
$\tau_{v,d}$ =	0.00	N/mm² <	$f_{v,d}$ =	2.70	N/mm² ✓

shear stress analysis Z fire 0 %

V_d =	0.00	kN	$f_{v,k}$ =	3.50	N/mm²
$\tau_{v,d}$ =	0.00	N/mm² <	$f_{v,d}$ =	2.70	N/mm² ✓

shear stress analysis combined fire 0 %

$V_{y,d}$ =	0.00	kN	$V_{z,d}$ =	0.00	kN
$\tau_{v,y,d}$ =	0.00	N/mm²	$\tau_{v,z,d}$ =	0.00	N/mm²
			ratio =	0 %	✓

lateral torsional buckling analysis fire 60 %

$M_{y,d}$ =	0.00	kNm	$f_{m,k}$ =	30.00	N/mm²
$N_{c,d}$ =	-54.02	kN	$f_{c,k}$ =	24.50	N/mm²
$\sigma_{c,d}$ =	5.63	N/mm²	$f_{c,d}$ =	28.18	N/mm²
$\sigma_{m,y,d}$ =	0.00	N/mm² <	$f_{m,y,d}$ =	34.50	N/mm² ✓

buckling analysis fire 42 %

$M_{y,d}$ =	0.00	kNm	$f_{m,k}$ =	30.00	N/mm²
$N_{c,d}$ =	-54.02	kN	$f_{c,k}$ =	24.50	N/mm²
$\sigma_{c,d}$ =	5.63	N/mm²	$f_{c,d}$ =	28.18	N/mm²
$\sigma_{m,y,d}$ =	0.00	N/mm²	$f_{m,y,d}$ =	34.50	N/mm²
$\sigma_{m,z,d}$ =	0.00	N/mm² <	$f_{m,z,d}$ =	34.50	N/mm² ✓

support reaction

load case category	k_{mod}	A_y	A_z	B_x	B_y	B_z
		[kN]	[kN]	[kN]	[kN]	[kN]
self weight CLT	0.6	0.00	0.00	0.39	0.00	0.00
		0.00	0.00	0.39	0.00	0.00
self weight	0.6	0.00	0.00	31.76	0.00	0.00
		0.00	0.00	31.76	0.00	0.00
wind load	0.9	0.00	0.00	0.00	0.00	0.00

support reaction						
load case category	k_{mod}	A_y	A_z	B_x	B_y	B_z
		[kN]	[kN]	[kN]	[kN]	[kN]
		0.00	0.00	18.30	0.00	0.00
snow load CEN > 1000m altitude	0.8	0.00	0.00	0.00	0.00	0.00
		0.00	0.00	43.73	0.00	0.00

Disclaimer

VEDLEGG G3.1

SØYLE 4-6

Referanse til standarder

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Lastvarighetsklasse for vindlasten: Øyeblikkslast

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Antar søyletverrrsnitt: $b := 215\ mm$ $h := 180\ mm$

Lengde: $L := 2150\ mm$

[5] Tab. 6 Limtre GL30c:

$f_{m.k} := 30\ \frac{N}{mm^2}$ $f_{c.90} := 2.5\ \frac{N}{mm^2}$ $f_{v.k} := 3.5\ \frac{N}{mm^2}$

$f_{c.0.k} := 24.5\ \frac{N}{mm^2}$ $E_{0.05} := 10800\ \frac{N}{mm^2}$ $\rho_m := 430\ \frac{kg}{m^3}$

$g := 9.81\ \frac{m}{s^2}$

LASTER

(Verdier hentet fra Calculatis)

Egenvekt tak, bjelke og fasade: $G_{tb} := 31.86\ kN$

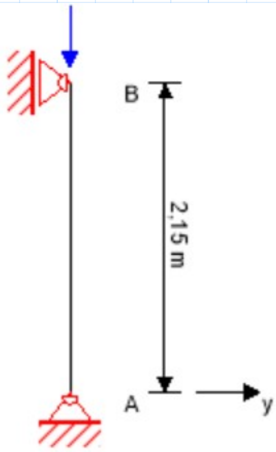
Egenvekt søyle: $G_s := 4.609\ \frac{kN}{m^3} \cdot b \cdot h \cdot L$ $G_s = 383.492\ N$

Permanent last: $G_k := G_{tb} + G_s$ $G_k = 32.243\ kN$

Snølast: $S_k := 43.96\ kN$

Vindlast (fra tak): $Q_k := 18.41\ kN$

LASTKOMBINASJONER



1. Kun egenlast $k_{mod} := 0.6$

$G_{f1} := 1.35 \cdot G_k$ $G_{f1} = 43.529\ kN$

2. Alle laster $k_{mod} := 1.1$

a) Egenlast dominerende:

$G_{f2} := 1.35 \cdot G_k$ $G_{f2} = 43.529\ kN$

$S_{f2} := 1.05 \cdot S_k$ $S_{f2} = 46.158\ kN$

$Q_{f2} := 1.05 \cdot Q_k$ $Q_{f2} = 19.331\ kN$

b) Snølast dominerende:

$G_{f3} := 1.2 \cdot G_k$ $G_{f3} = 38.692\ kN$

$S_{f3} := 1.5 \cdot S_k$ $S_{f3} = 65.94\ kN$

$Q_{f3} := 1.05 \cdot Q_k$ $Q_{f3} = 19.331\ kN$

c) Vindlast dominerende:

$G_{f4} := 1.2 \cdot G_k$ $G_{f4} = 38.692\ kN$

$S_{f4} := 1.05 \cdot S_k$ $S_{f4} = 46.158\ kN$

$Q_{f4} := 1.5 \cdot Q_k$ $Q_{f4} = 27.615\ kN$

3. Kun egenlast og snølast $k_{mod} := 0.9$

a) Egenlast dominerende:

$G_{f5} := 1.35 \cdot G_k$ $G_{f5} = 43.529\ kN$

$S_{f5} := 1.05 \cdot S_k$ $S_{f5} = 46.158\ kN$

b) Snølast dominerende:

$G_{f6} := 1.2 \cdot G_k$ $G_{f6} = 38.692\ kN$

$S_{f6} := 1.5 \cdot S_k$ $S_{f6} = 65.94\ kN$

KNEKKING

y-y: Knekklengde: $l_{ky} := 2150\ mm + 450\ mm$ (Søyle + bjelkehøyde)

Treghetsradius: $i_y := 0.29 \cdot h$ $i_y = 52.2\ mm$

Slankhet: $\lambda_y := \dfrac{l_{ky}}{i_y}$ $\lambda_y = 49.808$

[1] (6.21) $\lambda_{rel.y} := \dfrac{\lambda_y}{\pi} \cdot \sqrt{\dfrac{f_{c.0.k}}{E_{0.05}}}$ $\lambda_{rel.y} = 0.755$

[1] (6.29) $\beta_c := 0.1$ (limtre)

[1] (6.27) $k_y := 0.5 \cdot \left(1 + \beta_c \cdot \left(\lambda_{rel.y} - 0.3\right) + \lambda_{rel.y}^{\ 2}\right)$ $k_y = 0.808$

$k_{cy} := \dfrac{1}{k_y + \sqrt{k_y^{\ 2} - \lambda_{rel.y}^{\ 2}}}$ $k_{cy} = 0.913$

z-z: Knekklengde: $l_{kz} := 2150\ mm + 450\ mm$ (Søyle + bjelkehøyde)

Treghetsradius: $i_z := 0.29 \cdot b$ $i_z = 62.35\ mm$

Slankhet: $\lambda_z := \frac{l_{kz}}{i_z}$ $\lambda_z = 41.7$

[1] (6.21) $\lambda_{rel.z} := \frac{\lambda_z}{\pi} \cdot \sqrt{\frac{f_{c.0.k}}{E_{0.05}}}$ $\lambda_{rel.z} = 0.632$

[1] (6.29) $\beta_c := 0.1$ (limtre)

[1] (6.27) $k_z := 0.5 \cdot \left(1 + \beta_c \cdot (\lambda_{rel.z} - 0.3) + \lambda_{rel.z}^{\ 2}\right)$ $k_z = 0.716$

$k_{cz} := \frac{1}{k_z + \sqrt{k_z^{\ 2} - \lambda_{rel.z}^{\ 2}}}$ $k_{cz} = 0.949$

<u>BRANNDIMENSJONERING</u>

[3] Tab 3.1 Dimensjonerende forkullingshastighet for limtre: $\beta_n := 0.7\ \frac{mm}{min}$

[4] § 11-3 Brannklasse 2

[4] § 11-4 Brannmotstandstid R60: $t := 60\ min$

[2] Tab. NA.A1.1

$\psi_{0.s} := 0.7$ $\psi_{1.s} := 0.5$ $\psi_{2.s} := 0.2$

$\psi_{0.v} := 0.6$ $\psi_{1.v} := 0.2$ $\psi_{2.v} := 0$

$N_{fi} := 1.0 \cdot G_k + 1.0 \cdot \psi_{2.v} \cdot Q_k + 1.0 \cdot \psi_{1.s} \cdot S_k$ $N_{fi} = 54.223\ kN$

[3] 4.2.2 **Effektiv forkullingsdybde**

$t > 20\ min$ $\quad k_0 := 1.0$

$d_{char.n} := \beta_n \cdot t$

$d_0 := 7\ mm$

$d_{ef} := d_{char.n} + k_0 \cdot d_0$ $\quad d_{ef} = 49\ mm$

Dimensjon effektivt resttverrsnitt

$b_{ef.fi} := b - 2 \cdot d_{ef}$ $\quad b_{ef.fi} = 117\ mm$

$h_{ef.fi} := h - 2 \cdot d_{ef}$ $\quad h_{ef.fi} = 82\ mm$

Dimensjonerende fasthet

$f_{c.0.d.fi} := 1.15 \cdot f_{c.0.k}$ $\quad f_{c.0.d.fi} = 28.175\ \frac{N}{mm^2}$

$f_{m.d.fi} := 1.15 \cdot f_{m.k}$ $\quad f_{m.d.fi} = 34.5\ \frac{N}{mm^2}$

Knekking - brann

y-y: Treghetsradius: $i_{y.fi} := 0.29 \cdot h_{ef.fi}$ $\quad i_{y.fi} = 23.78\ mm$

Slankhet: $\lambda_{y.fi} := \frac{l_{ky}}{i_{y.fi}}$ $\quad \lambda_{y.fi} = 109.336$

[1] (6.21) $\lambda_{rel.y.fi} := \frac{\lambda_{y.fi}}{\pi} \cdot \sqrt{\frac{f_{c.0.k}}{E_{0.05}}}$ $\quad \lambda_{rel.y.fi} = 1.658$

[1] (6.29) $\beta_c := 0.1$ (limtre)

[1] (6.27) $k_{y.fi} := 0.5 \cdot \left(1 + \beta_c \cdot \left(\lambda_{rel.y.fi} - 0.3\right) + \lambda_{rel.y.fi}^{\ 2}\right)$ $\quad k_{y.fi} = 1.942$

$$k_{cy.fi} := \frac{1}{k_{y.fi} + \sqrt{k_{y.fi}^{2} - \lambda_{rel.y.fi}^{2}}}$$ $k_{cy.fi} = 0.339$

z-z: Treghetsradius: $i_{z.fi} := 0.29 \cdot b_{ef.fi}$ $i_{z.fi} = 33.93\ mm$

Slankhet: $\lambda_{z.fi} := \frac{l_{kz}}{i_{z.fi}}$ $\lambda_{z.fi} = 76.628$

[1] (6.21) $\lambda_{rel.z.fi} := \frac{\lambda_{z.fi}}{\pi} \cdot \sqrt{\frac{f_{c.0.k}}{E_{0.05}}}$ $\lambda_{rel.z.fi} = 1.162$

[1] (6.29) $\beta_c := 0.1$ (limtre)

[1] (6.27) $k_{z.fi} := 0.5 \cdot \left(1 + \beta_c \cdot \left(\lambda_{rel.z.fi} - 0.3\right) + \lambda_{rel.z.fi}^{2}\right)$ $k_{z.fi} = 1.218$

$$k_{cz.fi} := \frac{1}{k_{z.fi} + \sqrt{k_{z.fi}^{2} - \lambda_{rel.z.fi}^{2}}}$$ $k_{cz.fi} = 0.632$

[1] 6.1.4 **Aksialspenning**

$A_{fi} := b_{ef.fi} \cdot h_{ef.fi}$

$\sigma_{c.0.d.fi} := \frac{N_{fi}}{A_{fi}}$ $\sigma_{c.0.d.fi} = 5.652\ \frac{N}{mm^{2}}$

$\frac{\sigma_{c.0.d.fi}}{k_{cy.fi} \cdot f_{c.0.d.fi}} = 0.592$ < 1.0 OK

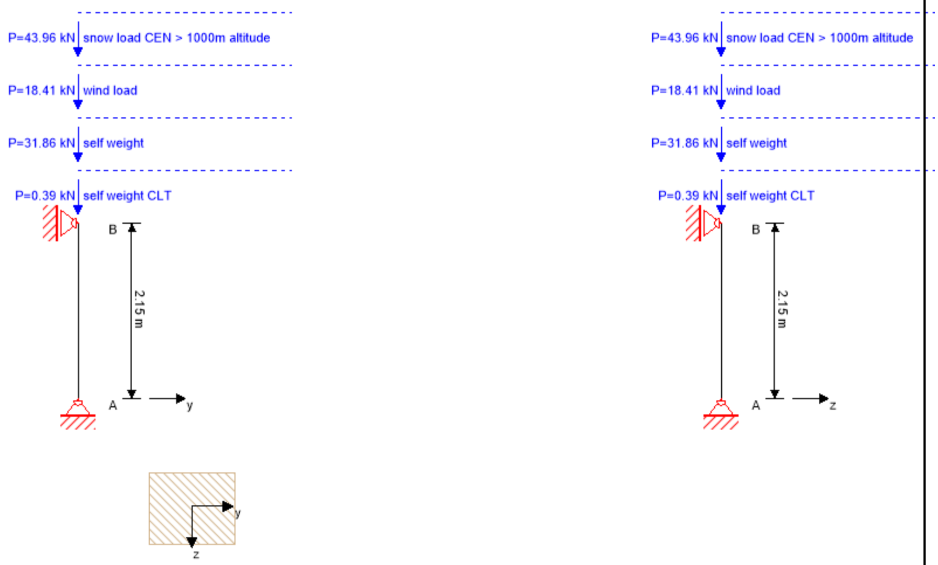
VEDLEGG G3.2

Søyle 4-6				
L	2,15		kcz	0,949
b	215		kcy	0,913
h	180		fcok	24,5
A	38700		Ym	1,15
W	1161000		fmk	30

Kombinasjon	1	2a	2b	2c	3a	3b
G	32,24	32,24	32,24	32,24	32,24	32,24
γG	1,35	1,35	1,20	1,20	1,35	1,20
S		43,96	43,96	43,96	43,96	43,96
γS		1,05	1,50	1,05	1,05	1,50
Q		18,41	18,41	18,41		
γQ		1,05	1,05	1,50		
Nf	43,52	109,01	123,96	112,46	89,68	104,63
σN	1,12	2,82	3,20	2,91	2,32	2,70
kmod	0,6	1,1	1,1	1,1	0,9	0,9
fcod	12,78	23,43	23,43	23,43	19,17	19,17
UF (6.23)	0,10	0,13	**0,15**	0,14	0,13	**0,15**
UF (6.24)	0,09	0,13	0,14	0,13	0,13	**0,15**

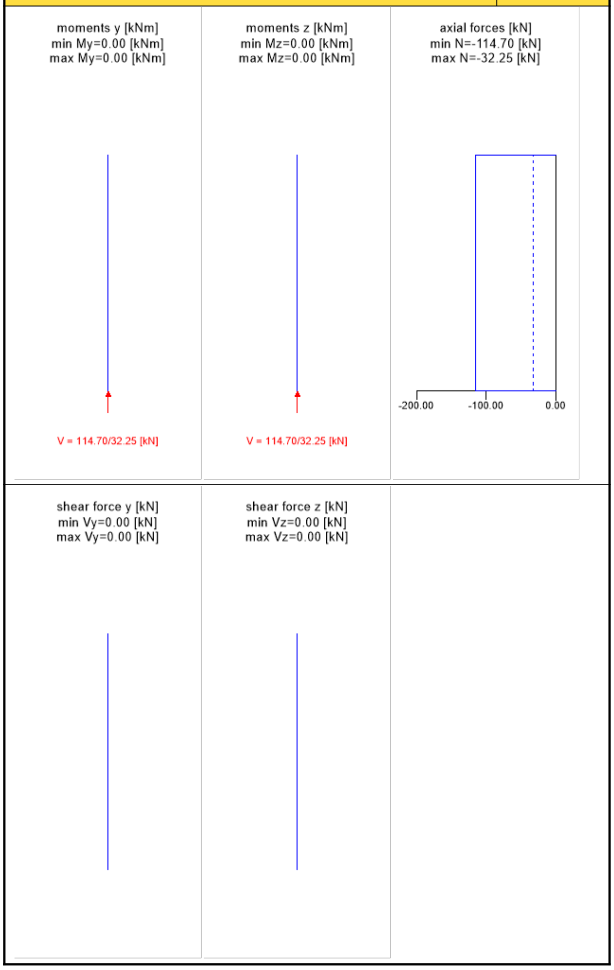
VEDLEGG G3.3

system



section: wooden beam 21.5/18; **material**: GL30c; **service class**: service class 1; **fire resistance class**: R 60

utilization — 60 %



flexural stress analysis — 2 %

$M_{y,d}$ =	0.00	kNm	$f_{m,k}$ =	30.00	N/mm²	
$N_{c,d}$ =	-	kN	$f_{c,k}$ =	24.50	N/mm²	
	114.70					
$\sigma_{c,d}$ =	2.96	N/mm²	$f_{c,d}$ =	19.17	N/mm²	
$\sigma_{m,y,d}$ =	0.00	N/mm²<	$f_{m,y,d}$ =	23.48	N/mm²	✓

shear stress analysis Y — 0 %

V_d =	0.00	kN	$f_{v,k}$ =	3.50	N/mm²	
$\tau_{v,d}$ =	0.00	N/mm²<	$f_{v,d}$ =	1.84	N/mm²	✓

shear stress analysis Z — 0 %

V_d =	0.00	kN	$f_{v,k}$ =	3.50	N/mm²	
$\tau_{v,d}$ =	0.00	N/mm²<	$f_{v,d}$ =	1.84	N/mm²	✓

shear stress analysis combined — 0 %

$V_{y,d}$ =	0.00	kN	$V_{z,d}$ =	0.00	kN	
$\tau_{v,y,d}$ =	0.00	N/mm²	$\tau_{v,z,d}$ =	0.00	N/mm²	
			ratio =	0 %		✓

lateral torsional buckling analysis — 17 %

$M_{y,d}$ =	0.00	kNm	$f_{m,k}$ =	30.00	N/mm²	
$N_{c,d}$ =	-	kN	$f_{c,k}$ =	24.50	N/mm²	
	114.70					
$\sigma_{c,d}$ =	2.96	N/mm²	$f_{c,d}$ =	19.17	N/mm²	
$\sigma_{m,y,d}$ =	0.00	N/mm²<	$f_{m,y,d}$ =	23.48	N/mm²	✓

buckling analysis — 16 %

$M_{y,d}$ =	0.00	kNm	$f_{m,k}$ =	30.00	N/mm²	
$N_{c,d}$ =	-	kN	$f_{c,k}$ =	24.50	N/mm²	
	114.70					
$\sigma_{c,d}$ =	2.96	N/mm²	$f_{c,d}$ =	19.17	N/mm²	
$\sigma_{m,y,d}$ =	0.00	N/mm²	$f_{m,y,d}$ =	23.48	N/mm²	
$\sigma_{m,z,d}$ =	0.00	N/mm²<	$f_{m,z,d}$ =	23.48	N/mm²	✓

flexural stress analysis fire — 4 %

$M_{y,d}$ =	0.00	kNm	$f_{m,k}$ =	30.00	N/mm²	
$N_{c,d}$ =	-54.23	kN	$f_{c,k}$ =	24.50	N/mm²	
$\sigma_{c,d}$ =	5.65	N/mm²	$f_{c,d}$ =	28.18	N/mm²	
$\sigma_{m,y,d}$ =	0.00	N/mm²<	$f_{m,y,d}$ =	34.50	N/mm²	✓

shear stress analysis Y fire — 0 %

V_d =	0.00	kN	$f_{v,k}$ =	3.50	N/mm²	
$\tau_{v,d}$ =	0.00	N/mm²<	$f_{v,d}$ =	2.70	N/mm²	✓

shear stress analysis Z fire — 0 %

V_d =	0.00	kN	$f_{v,k}$ =	3.50	N/mm²	
$\tau_{v,d}$ =	0.00	N/mm²<	$f_{v,d}$ =	2.70	N/mm²	✓

shear stress analysis combined fire — 0 %

$V_{y,d}$ =	0.00	kN	$V_{z,d}$ =	0.00	kN	
$\tau_{v,y,d}$ =	0.00	N/mm²	$\tau_{v,z,d}$ =	0.00	N/mm²	
			ratio =	0 %		✓

lateral torsional buckling analysis fire — 60 %

$M_{y,d}$ =	0.00	kNm	$f_{m,k}$ =	30.00	N/mm²	
$N_{c,d}$ =	-54.23	kN	$f_{c,k}$ =	24.50	N/mm²	
$\sigma_{c,d}$ =	5.65	N/mm²	$f_{c,d}$ =	28.18	N/mm²	
$\sigma_{m,y,d}$ =	0.00	N/mm²<	$f_{m,y,d}$ =	34.50	N/mm²	✓

buckling analysis fire — 42 %

$M_{y,d}$ =	0.00	kNm	$f_{m,k}$ =	30.00	N/mm²	
$N_{c,d}$ =	-54.23	kN	$f_{c,k}$ =	24.50	N/mm²	
$\sigma_{c,d}$ =	5.65	N/mm²	$f_{c,d}$ =	28.18	N/mm²	
$\sigma_{m,y,d}$ =	0.00	N/mm²	$f_{m,y,d}$ =	34.50	N/mm²	
$\sigma_{m,z,d}$ =	0.00	N/mm²<	$f_{m,z,d}$ =	34.50	N/mm²	✓

support reaction

load case category	k_{mod}	A_y	A_z	B_x	B_y	B_z
		[kN]	[kN]	[kN]	[kN]	[kN]
self weight CLT	0.6	0.00	0.00	0.39	0.00	0.00
		0.00	0.00	0.39	0.00	0.00
self weight	0.6	0.00	0.00	31.86	0.00	0.00
		0.00	0.00	31.86	0.00	0.00
wind load	0.9	0.00	0.00	0.00	0.00	0.00

support reaction						
load case category	k_{mod}	A_y	A_z	B_x	B_y	B_z
		[kN]	[kN]	[kN]	[kN]	[kN]
		0.00	0.00	18.41	0.00	0.00
snow load CEN > 1000m altitude	0.8	0.00	0.00	0.00	0.00	0.00
		0.00	0.00	43.96	0.00	0.00

Disclaimer

VEDLEGG G4.1

SØYLE 3-10

Referanse til standarder

[1] NS-EN 1995-1-1: Allmenne regler og regler for bygninger
[2] NS-EN 1990: Grunnlag for prosjektering av konstruksjoner
[3] NS-EN 1995-1-2: Brannteknisk dimensjonering
[4] Byggeteknisk forskrift (TEK17)
[5] Norsk Limtreprodusenters Forening, *Limtreboka - Beregningseksempler* (2018)

FORUTSETNINGER OG ANTAKELSER

[1] NA.901 Klimaklasse: 1

[1] Tab. 2.1 Lastvarighetsklasse for snølast: Korttidslast
Lastvarighetsklasse for vindlasten: Øyeblikkslast

[1] NA.2.3 Partialfaktor for limtre: $\gamma_M := 1.15$

Antar søyletverrrsnitt: $b := 215\ mm$ $h := 225\ mm$

Lastbredde: $Lb := 6640\ mm$

Lengde: $L := 2150\ mm$

[5] Tab. 6 Limtre GL30c:

$f_{m.k} := 30\ \frac{N}{mm^2}$ $f_{c.90.k} := 2.5\ \frac{N}{mm^2}$ $f_{v.k} := 3.5\ \frac{N}{mm^2}$

$f_{c.0.k} := 24.5\ \frac{N}{mm^2}$ $E_{0.05} := 10800\ \frac{N}{mm^2}$ $\rho_m := 430\ \frac{kg}{m^3}$

$g := 9.81\ \frac{m}{s^2}$

LASTER

(Verdier hente fra Calculatis)

Egenvekt: $G_b := 17.8\ kN + 47.36\ kN$ $G_b = 65.16\ kN$

Egenvekt søyle: $G_s := 4.609\ \frac{kN}{m^3} \cdot b \cdot h \cdot L$ $G_s = 479.365\ N$

Permanent last: $G_k := G_b + G_s$ $G_k = 65.639\ kN$

Snølast: $S_k := 21.3\ kN$

Nyttelast: $N_k := 26.6\ kN$

Vindlast (fra tak): $Q_k := 8.9\ kN$

Vindlast på vegg:

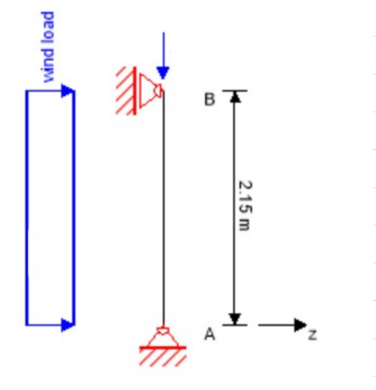
$q_{kast} := 1.34\ \frac{kN}{m^2}$ $C_A := 1.2$ $C_D := 0.724$

$q_{vind1} := q_{kast} \cdot (C_A + 0.2) \cdot Lb$ $q_{vind1} = 12.457\ \frac{kN}{m}$

$q_{vind2} := q_{kast} \cdot (C_D + 0.3) \cdot Lb$ $q_{vind2} = 9.111\ \frac{kN}{m}$

$q_k := \max(q_{vind1}, q_{vind2})$ $q_k = 12.457\ \frac{kN}{m}$

LASTKOMBINASJONER



1. Kun egenlast $k_{mod} := 0.6$

$G_{f1} := 1.35 \cdot G_k$ $G_{f1} = 88.613\ kN$

2. Alle laster $k_{mod} := 1.1$

a) Egenlast dominerende:

$G_{f2} := 1.35 \cdot G_k$ $G_{f2} = 88.613\ kN$

$S_{f2} := 1.05 \cdot S_k$ $S_{f2} = 22.365\ kN$

$N_{f2} := 1.05 \cdot N_k$ $N_{f2} = 27.93\ kN$

$Q_{f2} := 1.05 \cdot Q_k$ $Q_{f2} = 9.345\ kN$

$q_{f2} := 1.05 \cdot q_k$ $q_{f2} = 13.079\ \frac{kN}{m}$

b) Snølast dominerende:

$G_{f3} := 1.2 \cdot G_k$ $G_{f3} = 78.767\ kN$

$S_{f3} := 1.5 \cdot S_k$ $S_{f3} = 31.95\ kN$

$N_{f3} := 1.05 \cdot N_k$ $N_{f3} = 27.93\ kN$

$Q_{f3} := 1.05 \cdot Q_k$ $Q_{f3} = 9.345\ kN$

$q_{f3} := 1.05 \cdot q_k$ $q_{f3} = 13.079\ \frac{kN}{m}$

c) Nyttelast dominerende:

$G_{f4} := 1.2 \cdot G_k$ $G_{f4} = 78.767\ kN$

$S_{f4} := 1.05 \cdot S_k$ $S_{f4} = 22.365\ kN$

$N_{f4} := 1.5 \cdot N_k$ $N_{f4} = 39.9\ kN$

$Q_{f4} := 1.05 \cdot Q_k$ $Q_{f4} = 9.345\ kN$

$q_{f4} := 1.05 \cdot q_k$ $q_{f4} = 13.079\ \frac{kN}{m}$

d) Vindlast dominerende:

$G_{f5} := 1.2 \cdot G_k$ $G_{f5} = 78.767\ kN$

$S_{f5} := 1.05 \cdot S_k$ $S_{f5} = 22.365\ kN$

$N_{f5} := 1.05 \cdot N_k$ $N_{f5} = 27.93\ kN$

$Q_{f5} := 1.5 \cdot Q_k$ $Q_{f5} = 13.35\ kN$

$q_{f5} := 1.5 \cdot q_k$ $q_{f5} = 18.685\ \frac{kN}{m}$

3. Egenlast, nyttelast og snølast $k_{mod} := 0.9$

a) Egenlast dominerende:

$G_{f6} := 1.35 \cdot G_k$ $G_{f6} = 88.613\ kN$

$S_{f6} := 1.05 \cdot S_k$ $S_{f6} = 22.365\ kN$

$N_{f6} := 1.05 \cdot N_k$ $N_{f6} = 27.93\ kN$

b) Snølast dominerende:

$G_{f7} := 1.2 \cdot G_k$ $G_{f7} = 78.767\ kN$

$S_{f7} := 1.5 \cdot S_k$ $S_{f7} = 31.95\ kN$

$N_{f7} := 1.05 \cdot N_k$ $N_{f7} = 27.93\ kN$

b) Nyttelast dominerende:

$G_{f8} := 1.2 \cdot G_k$ $G_{f8} = 78.767\ kN$

$S_{f8} := 1.05 \cdot S_k$ $S_{f8} = 22.365\ kN$

$N_{f8} := 1.5 \cdot N_k$ $N_{f8} = 39.9\ kN$

3. Egenlast og nyttelast $k_{mod} := 0.8$

a) Egenlast dominerende:

$G_{f9} := 1.35 \cdot G_k$ $G_{f9} = 88.613\ kN$

$N_{f9} := 1.05 \cdot N_k$ $N_{f9} = 27.93\ kN$

a) Nyttelast dominerende:

$G_{f10} := 1.35 \cdot G_k$ $G_{f10} = 88.613\ kN$

$N_{f10} := 1.5 \cdot N_k$ $N_{f10} = 39.9\ kN$

[1] (3.2) Høydefaktor: $k_h := min\left(\left(\frac{600\ mm}{h}\right)^{0.1}, 1.1\right)$ $k_h = 1.1$

KNEKKING

y-y: Knekklengde: $l_{ky} := 2150\ mm + 550\ mm$ (søyle + bjelkehøyde)

Treghetsradius: $i_y := 0.29 \cdot h$ $i_y = 65.25\ mm$

Slankhet: $\lambda_y := \frac{l_{ky}}{i_y}$ $\lambda_y = 41.379$

[1] (6.21) $\lambda_{rel.y} := \frac{\lambda_y}{\pi} \cdot \sqrt{\frac{f_{c.0.k}}{E_{0.05}}}$ $\lambda_{rel.y} = 0.627$

[1] (6.29) $\beta_c := 0.1$ (limtre)

[1] (6.27) $k_y := 0.5 \cdot \left(1 + \beta_c \cdot (\lambda_{rel.y} - 0.3) + \lambda_{rel.y}^{2}\right)$ $k_y = 0.713$

$k_{cy} := \frac{1}{k_y + \sqrt{k_y^{2} - \lambda_{rel.y}^{2}}}$ $k_{cy} = 0.95$

z-z: Knekklengde: $l_{kz} := 2150\ mm + 550\ mm$

Treghetsradius: $i_z := 0.29 \cdot b$ $i_z = 62.35\ mm$

Slankhet: $\lambda_z := \frac{l_{kz}}{i_z}$ $\lambda_z = 43.304$

[1] (6.21) $\lambda_{rel.z} := \frac{\lambda_z}{\pi} \cdot \sqrt{\frac{f_{c.0.k}}{E_{0.05}}}$ $\lambda_{rel.z} = 0.657$

[1] (6.29) $\beta_c := 0.1$ (limtre)

[1] (6.27) $k_z := 0.5 \cdot \left(1 + \beta_c \cdot (\lambda_{rel.z} - 0.3) + \lambda_{rel.z}^{2}\right)$ $k_z = 0.733$

$$k_{cz} := \frac{1}{k_z + \sqrt{{k_z}^2 - {\lambda_{rel.z}}^2}} \qquad k_{cz} = 0.943$$

BRANNDIMENSJONERING

[3] Tab 3.1 Dimensjonerende forkullingshastighet for limtre: $\beta_n := 0.7 \ \frac{mm}{min}$

[4] § 11-3 Brannklasse 2

[4] § 11-4 Brannmotstandstid R60: $t := 60 \ min$

[2] Tab. NA.A1.1

$\psi_{0.s} := 0.7 \qquad \psi_{1.s} := 0.5 \qquad \psi_{2.s} := 0.2$

$\psi_{0.v} := 0.6 \qquad \psi_{1.v} := 0.2 \qquad \psi_{2.v} := 0$

$\psi_{0.n} := 0.7 \qquad \psi_{1.n} := 0.5 \qquad \psi_{2.n} := 0.3$

$N_{fi} := G_k + N_k \cdot \psi_{1.n} + \psi_{2.s} \cdot S_k \qquad N_{fi} = 83.199 \ kN$

[3] 4.2.2 **Effektiv forkullingsdybde**

$t > 20 \ min \qquad k_0 := 1.0$

$d_{char.n} := \beta_n \cdot t$

$d_0 := 7 \ mm$

$d_{ef} := d_{char.n} + k_0 \cdot d_0 \qquad d_{ef} = 49 \ mm$

Dimensjon effektivt resttverrsnitt

$b_{ef.fi} := b - 2 \cdot d_{ef} \qquad b_{ef.fi} = 117 \ mm$

$h_{ef.fi} := h - 2 \cdot d_{ef} \qquad h_{ef.fi} = 127 \ mm$

Dimensjonerende fasthet

$f_{c.0.d.fi} := 1.15 \cdot f_{c.0.k}$ $\quad f_{c.0.d.fi} = 28.175 \ \frac{N}{mm^2}$

$f_{m.d.fi} := 1.15 \cdot f_{m.k}$ $\quad f_{m.d.fi} = 34.5 \ \frac{N}{mm^2}$

Knekking - brann

y-y: Treghetsradius: $i_{y.fi} := 0.29 \cdot h_{ef.fi}$ $\quad i_{y.fi} = 36.83 \ mm$

Slankhet: $\lambda_{y.fi} := \frac{l_{ky}}{i_{y.fi}}$ $\quad \lambda_{y.fi} = 73.31$

[1] (6.21) $\lambda_{rel.y.fi} := \frac{\lambda_{y.fi}}{\pi} \cdot \sqrt{\frac{f_{c.0.k}}{E_{0.05}}}$ $\quad \lambda_{rel.y.fi} = 1.111$

[1] (6.29) $\beta_c := 0.1$ (limtre)

[1] (6.27) $k_{y.fi} := 0.5 \cdot \left(1 + \beta_c \cdot \left(\lambda_{rel.y.fi} - 0.3\right) + \lambda_{rel.y.fi}^{2}\right)$ $\quad k_{y.fi} = 1.158$

$k_{cy.fi} := \frac{1}{k_{y.fi} + \sqrt{k_{y.fi}^{2} - \lambda_{rel.y.fi}^{2}}}$ $\quad k_{cy.fi} = 0.674$

z-z: Treghetsradius: $i_{z.fi} := 0.29 \cdot b_{ef.fi}$ $\quad i_{z.fi} = 33.93 \ mm$

Slankhet: $\lambda_{z.fi} := \frac{l_{kz}}{i_{z.fi}}$ $\quad \lambda_{z.fi} = 79.576$

[1] (6.21) $\lambda_{rel.z.fi} := \frac{\lambda_{z.fi}}{\pi} \cdot \sqrt{\frac{f_{c.0.k}}{E_{0.05}}}$ $\quad \lambda_{rel.z.fi} = 1.206$

[1] (6.29) $\beta_c := 0.1$ (limtre)

[1] (6.27) $k_{z.fi} := 0.5 \cdot \left(1 + \beta_c \cdot \left(\lambda_{rel.z.fi} - 0.3\right) + \lambda_{rel.z.fi}^{2}\right)$ $\quad k_{z.fi} = 1.273$

$$k_{cz.fi} := \frac{1}{k_{z.fi} + \sqrt{k_{z.fi}^{2} - \lambda_{rel.z.fi}^{2}}} \qquad k_{cz.fi} = 0.595$$

[1] 6.1.6 **Bøyespenning**

$$W := \frac{1}{6} \cdot b_{ef.fi} \cdot h_{ef.fi}^{2}$$

$$M_{y.fi} := \frac{q_k \cdot \psi_{1.v} \cdot L^2}{8} \qquad M_{y.fi} = 1.44\ kN \cdot m$$

$$\sigma_{m.y.fi} := \frac{M_{y.fi}}{W} \qquad \sigma_{m.y.fi} = 4.577\ \frac{N}{mm^2}$$

$$\frac{\sigma_{m.y.fi}}{f_{m.d.fi}} = 0.133$$ < 1.0 OK

[1] 6.1.4 **Aksialspenning**

$$A_{fi} := b_{ef.fi} \cdot h_{ef.fi}$$

$$\sigma_{c.0.d.fi} := \frac{N_{fi}}{A_{fi}} \qquad \sigma_{c.0.d.fi} = 5.599\ \frac{N}{mm^2}$$

$$\frac{\sigma_{c.0.d.fi}}{k_{cz.fi} \cdot f_{c.0.d.fi}} = 0.334$$ < 1.0 OK

Kombinert virkning

[1] 6.1.6 (2) $k_m := 0.7$

[1] (6.23) $$\frac{\sigma_{c.0.d.fi}}{k_{cy.fi} \cdot f_{c.0.d.fi}} + \frac{\sigma_{m.y.fi}}{f_{m.d.fi}} = 0.428$$ < 1.0 OK

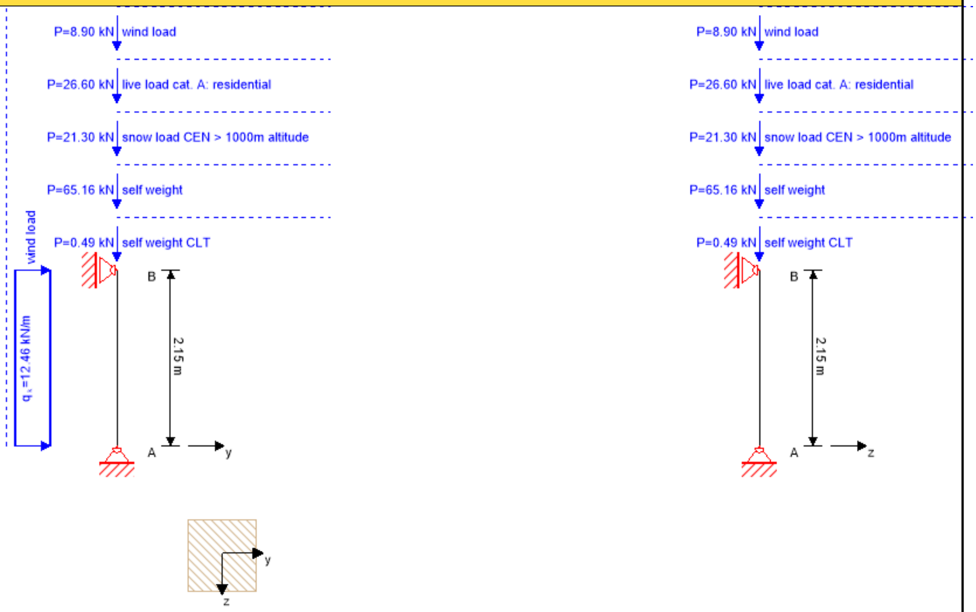
[1] (6.24) $$\frac{\sigma_{c.0.d.fi}}{k_{cz.fi} \cdot f_{c.0.d.fi}} + k_m \cdot \frac{\sigma_{m.y.fi}}{f_{m.d.fi}} = 0.427$$ < 1.0 OK

VEDLEGG G4.2

Søyle 3-10							
L	2,15		kcz	0,943		kh	1,1
b	215		kcy	0,95		km	0,7
h	225		fcok	24,5			
A	48375		Ym	1,15			
W	1814062,5		fmk	30			

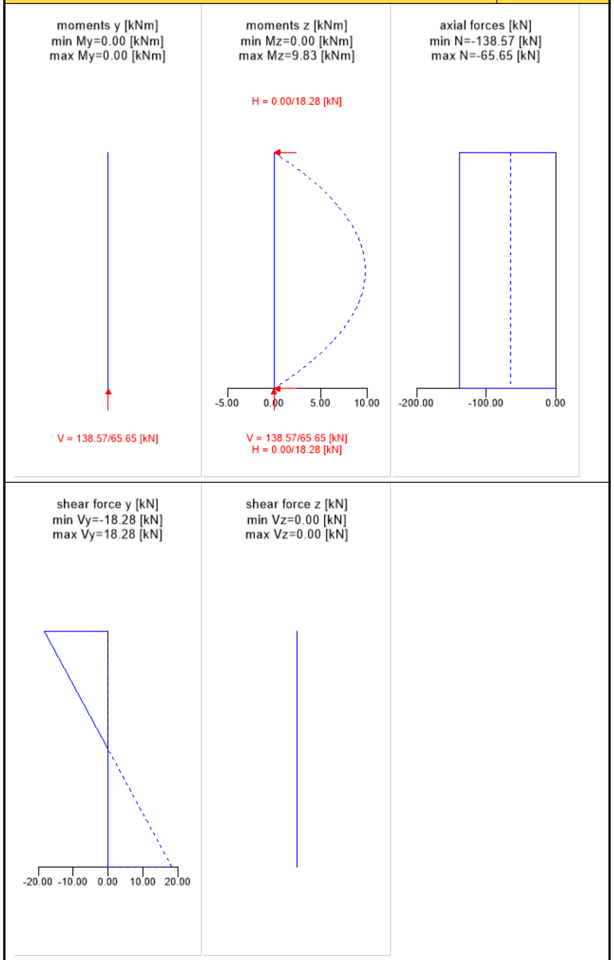
Kombinasjon	1	2a	2b	2c	2d	3a	3b	3c	4a	4b
G	65,64	65,64	65,64	65,64	65,64	65,64	65,64	65,64	65,64	65,64
γG	1,35	1,35	1,20	1,20	1,20	1,35	1,20	1,20	1,20	1,35
S		21,30	21,30	21,30	21,30	21,30	21,30	21,30		
γS		1,05	1,50	1,05	1,05	1,05	1,50	1,05		
N		26,60	26,60	26,60	26,60	26,60	26,60	26,60	26,60	26,60
γN		1,05	1,05	1,50	1,05	1,05	1,05	1,50	1,50	1,05
Q		8,90	8,90	8,90	8,90					
γQ		1,05	1,05	1,05	1,50					
q		12,46	12,46	12,46	12,46					
γq		1,05	1,05	1,05	1,50					
Nf	88,61	133,41	120,06	110,48	142,41	110,98	110,72	101,13	118,67	116,54
qf	0,00	13,08	13,08	13,08	18,69	0,00	0,00	0,00	0,00	0,00
Mf	0,00	7,56	7,56	7,56	10,80	0,00	0,00	0,00	0,00	0,00
σN	1,83	2,76	2,48	2,28	2,94	2,29	2,29	2,09	2,45	2,41
σM	0,00	4,17	4,17	4,17	5,95	0,00	0,00	0,00	0,00	0,00
kmod	0,6	1,1	1,1	1,1	1,1	0,9	0,9	0,9	0,8	0,8
fcod	12,78	23,43	23,43	23,43	23,43	19,17	19,17	19,17	17,04	17,04
fmyd	17,22	31,57	31,57	31,57	31,57	25,83	21,09	21,09	18,75	18,75
UF (6.23)	0,15	0,26	0,24	0,23	**0,32**	0,13	0,13	0,13	0,15	0,15
UF (6.24)	0,15	0,22	0,20	0,20	0,27	0,13	0,12	0,10	0,14	0,14

system



section: wooden beam 21.5/22.5; **material**: GL30c; **service class**: service class 1; **fire resistance class**: R 60

utilization 39 %



flexural stress analysis 26 %

$M_{y,d}$ =	0.00	kNm	$f_{m,k}$ =	30.00	N/mm²	
$N_{c,d}$ =	- 138.57	kN	$f_{c,k}$ =	24.50	N/mm²	
$\sigma_{c,d}$ =	2.86	N/mm²	$f_{c,d}$ =	19.17	N/mm²	
$\sigma_{m,y,d}$ =	0.00	N/mm²<	$f_{m,y,d}$ =	23.48	N/mm²	✓

shear stress analysis Y 31 %

V_d =	18.28	kN	$f_{v,k}$ =	3.50	N/mm²	
$\tau_{v,d}$ =	0.57	N/mm²<	$f_{v,d}$ =	1.84	N/mm²	✓

shear stress analysis Z 0 %

V_d =	0.00	kN	$f_{v,k}$ =	3.50	N/mm²	
$\tau_{v,d}$ =	0.00	N/mm²<	$f_{v,d}$ =	1.84	N/mm²	✓

shear stress analysis combined 10 %

$V_{y,d}$ =	18.28	kN	$V_{z,d}$ =	0.00	kN	
$\tau_{v,y,d}$ =	0.57	N/mm²	$\tau_{v,z,d}$ =	0.00	N/mm²	
			ratio =	10 %		✓

lateral torsional buckling analysis 18 %

$M_{y,d}$ =	0.00	kNm	$f_{m,k}$ =	30.00	N/mm²	
$N_{c,d}$ =	- 137.31	kN	$f_{c,k}$ =	24.50	N/mm²	
$\sigma_{c,d}$ =	2.84	N/mm²	$f_{c,d}$ =	17.04	N/mm²	
$\sigma_{m,y,d}$ =	0.00	N/mm²<	$f_{m,y,d}$ =	20.87	N/mm²	✓

buckling analysis 39 %

$M_{y,d}$ =	0.00	kNm	$f_{m,k}$ =	30.00	N/mm²	
$N_{c,d}$ =	- 138.57	kN	$f_{c,k}$ =	24.50	N/mm²	
$\sigma_{c,d}$ =	2.86	N/mm²	$f_{c,d}$ =	19.17	N/mm²	
$\sigma_{m,y,d}$ =	0.00	N/mm²	$f_{m,y,d}$ =	23.48	N/mm²	
$\sigma_{m,z,d}$ =	5.67	N/mm²<	$f_{m,z,d}$ =	23.48	N/mm²	✓

flexural stress analysis fire 18 %

$M_{y,d}$ =	0.00	kNm	$f_{m,k}$ =	30.00	N/mm²	
$N_{c,d}$ =	-79.67	kN	$f_{c,k}$ =	24.50	N/mm²	
$\sigma_{c,d}$ =	5.36	N/mm²	$f_{c,d}$ =	28.18	N/mm²	
$\sigma_{m,y,d}$ =	0.00	N/mm²<	$f_{m,y,d}$ =	34.50	N/mm²	✓

shear stress analysis Y fire 10 %

V_d =	2.68	kN	$f_{v,k}$ =	3.50	N/mm²	
$\tau_{v,d}$ =	0.27	N/mm²<	$f_{v,d}$ =	2.70	N/mm²	✓

shear stress analysis Z fire 0 %

V_d =	0.00	kN	$f_{v,k}$ =	3.50	N/mm²	
$\tau_{v,d}$ =	0.00	N/mm²<	$f_{v,d}$ =	2.70	N/mm²	✓

shear stress analysis combined fire 1 %

$V_{y,d}$ =	2.68	kN	$V_{z,d}$ =	0.00	kN	
$\tau_{v,y,d}$ =	0.27	N/mm²	$\tau_{v,z,d}$ =	0.00	N/mm²	
			ratio =	1 %		✓

lateral torsional buckling analysis fire 34 %

$M_{y,d}$ =	0.00	kNm	$f_{m,k}$ =	30.00	N/mm²	
$N_{c,d}$ =	-84.28	kN	$f_{c,k}$ =	24.50	N/mm²	
$\sigma_{c,d}$ =	5.67	N/mm²	$f_{c,d}$ =	28.18	N/mm²	
$\sigma_{m,y,d}$ =	0.00	N/mm²<	$f_{m,y,d}$ =	34.50	N/mm²	✓

buckling analysis fire 37 %

$M_{y,d}$ =	0.00	kNm	$f_{m,k}$ =	30.00	N/mm²	
$N_{c,d}$ =	-79.67	kN	$f_{c,k}$ =	24.50	N/mm²	
$\sigma_{c,d}$ =	5.36	N/mm²	$f_{c,d}$ =	28.18	N/mm²	
$\sigma_{m,y,d}$ =	0.00	N/mm²	$f_{m,y,d}$ =	34.50	N/mm²	
$\sigma_{m,z,d}$ =	4.97	N/mm²<	$f_{m,z,d}$ =	34.50	N/mm²	✓

support reaction

load case category	k_{mod}	A_y	A_z	B_x	B_y	B_z
		[kN]	[kN]	[kN]	[kN]	[kN]
self weight CLT	0.6	0.00	0.00	0.49	0.00	0.00
		0.00	0.00	0.49	0.00	0.00
self weight	0.6	0.00	0.00	65.16	0.00	0.00
		0.00	0.00	65.16	0.00	0.00
snow load CEN > 1000m altitude	0.8	0.00	0.00	0.00	0.00	0.00

support reaction						
load case category	k_{mod}	A_y	A_z	B_x	B_y	B_z
		[kN]	[kN]	[kN]	[kN]	[kN]
		0.00	0.00	21.30	0.00	0.00
live load cat. A: residential	0.8	0.00	0.00	0.00	0.00	0.00
		0.00	0.00	26.60	0.00	0.00
wind load	0.9	13.39	0.00	0.00	13.39	0.00
		0.00	0.00	8.90	0.00	0.00

Disclaimer

VEDLEGG G5.1

SØYLE 3-9

Referanse til standarder

[1] NS-EN 1995-1-1: Allmenne regler og regler for bygninger
[2] NS-EN 1990: Grunnlag for prosjektering av konstruksjoner
[3] NS-EN 1992-1-2: Brannteknisk dimensjonering
[4] Byggeteknisk forskrift (TEK17)
[5] Norsk Limtreprodusenters Forening, *Limtreboka - Beregningseksempler* (2018)

FORUTSETNINGER OG ANTAKELSER

[1] NA.901 Klimaklasse: 1

[1] Tab. 2.1 Lastvarighetsklasse for snølast: Korttidslast
Lastvarighetsklasse for vindlasten: Øyeblikkslast

[1] NA.2.3 Partialfaktor for limtre: $\gamma_M := 1.15$

Antar søyletverrrsnitt: $b := 215\ mm$ $h := 225\ mm$

Lengde: $L := 2150\ mm$

[5] Tab. 6 Limtre GL30c:

$f_{m.k} := 30\ \frac{N}{mm^2}$ $f_{c.90} := 2.5\ \frac{N}{mm^2}$ $f_{v.k} := 3.5\ \frac{N}{mm^2}$

$f_{c.0.k} := 24.5\ \frac{N}{mm^2}$ $E_{0.05} := 10800\ \frac{N}{mm^2}$ $\rho_m := 430\ \frac{kg}{m^3}$

$g := 9.81\ \frac{m}{s^2}$

LASTER

(Verdier hentet fra Calculatis)

Egenvekt: $G_b := 32.14\ kN + 75.41\ kN$ $G_b = 107.55\ kN$

Egenvekt søyle: $G_s := 4.609\ \frac{kN}{m^3} \cdot b \cdot h \cdot L$ $G_s = 479.365\ N$

Permanent last: $G_k := G_b + G_s$ $G_k = 108.029\ kN$

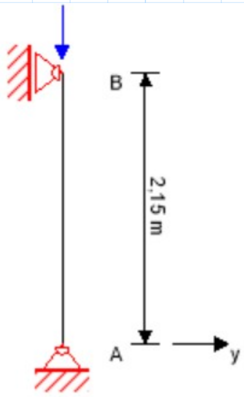
Snølast: $S_k := 43.73\ kN$

Vindlast: $Q_k := 18.3\ kN$

Nyttelast: $N_k := 54.66\ kN$

LASTKOMBINASJONER

1. Kun egenlast $k_{mod} := 0.6$



$G_{f1} := 1.35 \cdot G_k$ $G_{f1} = 145.84\ kN$

2. Alle laster $k_{mod} := 1.1$

a) Egenlast dominerende:

$G_{f2} := 1.35 \cdot G_k$ $G_{f2} = 145.84\ kN$

$S_{f2} := 1.05 \cdot S_k$ $S_{f2} = 45.917\ kN$

$Q_{f2} := 1.05 \cdot Q_k$ $Q_{f2} = 19.215\ kN$

$N_{f2} := 1.05 \cdot N_k$ $N_{f2} = 57.393\ kN$

b) Snølast dominerende:

$G_{f3} := 1.2 \cdot G_k$ $G_{f3} = 129.635\ kN$

$S_{f3} := 1.5 \cdot S_k$ $S_{f3} = 65.595\ kN$

$Q_{f3} := 1.05 \cdot Q_k$ $Q_{f3} = 19.215\ kN$

$N_{f3} := 1.05 \cdot N_k$ $N_{f3} = 57.393\ kN$

c) Vindlast dominerende:

$G_{f4} := 1.2 \cdot G_k$ $G_{f4} = 129.635\ kN$

$S_{f4} := 1.05 \cdot S_k$ $S_{f4} = 45.917\ kN$

$Q_{f4} := 1.5 \cdot Q_k$ $Q_{f4} = 27.45\ kN$

$N_{f4} := 1.05 \cdot N_k$ $N_{f4} = 57.393\ kN$

d) Nyttelast dominerende:

$G_{f5} := 1.2 \cdot G_k$ $G_{f5} = 129.635\ kN$

$S_{f5} := 1.05 \cdot S_k$ $S_{f5} = 45.917\ kN$

$Q_{f5} := 1.05 \cdot Q_k$ $Q_{f5} = 19.215\ kN$

$N_{f5} := 1.5 \cdot N_k$ $N_{f5} = 81.99\ kN$

3. Egenlast, nyttelast og snølast $k_{mod} := 0.9$

a) Egenlast dominerende:

$G_{f6} := 1.35 \cdot G_k$ $G_{f6} = 145.84\ kN$

$S_{f6} := 1.05 \cdot S_k$ $S_{f6} = 45.917\ kN$

$N_{f6} := 1.05 \cdot N_k$ $N_{f6} = 57.393\ kN$

b) Snølast dominerende:

$G_{f7} := 1.2 \cdot G_k$ $G_{f7} = 129.635\ kN$

$S_{f7} := 1.5 \cdot S_k$ $S_{f7} = 65.595\ kN$

$N_{f7} := 1.05 \cdot N_k$ $N_{f7} = 57.393\ kN$

c) Nyttelast dominerende:

$G_{f8} := 1.2 \cdot G_k$ $G_{f8} = 129.635\ kN$

$S_{f8} := 1.05 \cdot S_k$ $S_{f8} = 45.917\ kN$

$N_{f8} := 1.5 \cdot N_k$ $N_{f8} = 81.99\ kN$

4. Egenlast og nyttelast $k_{mod} := 0.8$

a) Egenlast dominerende:

$G_{f9} := 1.35 \cdot G_k$ $G_{f9} = 145.84\ kN$

$N_{f9} := 1.05 \cdot N_k$ $N_{f9} = 57.393\ kN$

b) Nyttelast dominerende:

$G_{f10} := 1.2 \cdot G_k$ $G_{f10} = 129.635\ kN$

$N_{f10} := 1.5 \cdot N_k$ $N_{f10} = 81.99\ kN$

KNEKKING

y-y: Knekklengde: $l_{ky} := 2150\ mm + 550\ mm$ (Søyle + bjelkehøyde)

Treghetsradius: $i_y := 0.29 \cdot h$ $i_y = 65.25\ mm$

Slankhet: $\lambda_y := \frac{l_{ky}}{i_y}$ $\lambda_y = 41.379$

[1] (6.21) $\lambda_{rel.y} := \frac{\lambda_y}{\pi} \cdot \sqrt{\frac{f_{c.0.k}}{E_{0.05}}}$ $\lambda_{rel.y} = 0.627$

[1] (6.29) $\beta_c := 0.1$ (limtre)

[1] (6.27) $k_y := 0.5 \cdot \left(1 + \beta_c \cdot (\lambda_{rel.y} - 0.3) + \lambda_{rel.y}^2\right)$ $k_y = 0.713$

$k_{cy} := \frac{1}{k_y + \sqrt{k_y^2 - \lambda_{rel.y}^2}}$ $k_{cy} = 0.95$

z-z: Knekklengde: $l_{kz} := 2150\ mm + 550\ mm$ (Søyle + bjelkehøyde)

Treghetsradius: $i_z := 0.29 \cdot b$ $i_z = 62.35\ mm$

Slankhet: $\lambda_z := \frac{l_{kz}}{i_z}$ $\lambda_z = 43.304$

[1] (6.21) $\lambda_{rel.z} := \frac{\lambda_z}{\pi} \cdot \sqrt{\frac{f_{c.0.k}}{E_{0.05}}}$ $\lambda_{rel.z} = 0.657$

[1] (6.29) $\beta_c := 0.1$ (limtre)

[1] (6.27) $k_z := 0.5 \cdot \left(1 + \beta_c \cdot (\lambda_{rel.z} - 0.3) + \lambda_{rel.z}^{\ 2}\right)$ $k_z = 0.733$

$k_{cz} := \frac{1}{k_z + \sqrt{k_z^{\ 2} - \lambda_{rel.z}^{\ 2}}}$ $k_{cz} = 0.943$

BRANNDIMENSJONERING

[3] Tab 3.1 Dimensjonerende forkullingshastighet for limtre: $\beta_n := 0.7\ \frac{mm}{min}$

[4] § 11-3 Brannklasse 2

[4] § 11-4 Brannmotstandstid R60: $t := 60\ min$

[2] Tab. NA.A1.1

$\psi_{0.s} := 0.7$ $\psi_{1.s} := 0.5$ $\psi_{2.s} := 0.2$

$\psi_{0.v} := 0.6$ $\psi_{1.v} := 0.2$ $\psi_{2.v} := 0$

$\psi_{0.n} := 0.7$ $\psi_{1.n} := 0.5$ $\psi_{2.n} := 0.3$

$N_{fi} := G_k + \psi_{1.n} \cdot N_k + \psi_{2.s} \cdot S_k + \psi_{2.v} \cdot Q_k$ $N_{fi} = 144.105\ kN$

[3] 4.2.2 **Effektiv forkullingsdybde**

$t > 20\ min$ $k_0 := 1.0$

$d_{char.n} := \beta_n \cdot t$

$d_0 := 7\ mm$

$d_{ef} := d_{char.n} + k_0 \cdot d_0$ $d_{ef} = 49\ mm$

Dimensjon effektivt resttverrsnitt

$b_{ef.fi} := b - 2 \cdot d_{ef}$ $b_{ef.fi} = 117\ mm$

$h_{ef.fi} := h - 2 \cdot d_{ef}$ $h_{ef.fi} = 127\ mm$

Dimensjonerende fasthet

$f_{c.0.d.fi} := 1.15 \cdot f_{c.0.k}$ $f_{c.0.d.fi} = 28.175\ \frac{N}{mm^2}$

$f_{m.d.fi} := 1.15 \cdot f_{m.k}$ $f_{m.d.fi} = 34.5\ \frac{N}{mm^2}$

Knekking - brann

y-y: Treghetsradius: $i_{y.fi} := 0.29 \cdot h_{ef.fi}$ $i_{y.fi} = 36.83\ mm$

Slankhet: $\lambda_{y.fi} := \frac{l_{ky}}{i_{y.fi}}$ $\lambda_{y.fi} = 73.31$

[1] (6.21) $\lambda_{rel.y.fi} := \frac{\lambda_{y.fi}}{\pi} \cdot \sqrt{\frac{f_{c.0.k}}{E_{0.05}}}$ $\lambda_{rel.y.fi} = 1.111$

[1] (6.29) $\beta_c := 0.1$ (limtre)

[1] (6.27) $k_{y.fi} := 0.5 \cdot \left(1 + \beta_c \cdot \left(\lambda_{rel.y.fi} - 0.3\right) + \lambda_{rel.y.fi}^{\ 2}\right)$ $k_{y.fi} = 1.158$

$$k_{cy.fi} := \frac{1}{k_{y.fi} + \sqrt{k_{y.fi}^{2} - \lambda_{rel.y.fi}^{2}}}$$ $k_{cy.fi} = 0.674$

z-z: Treghetsradius: $i_{z.fi} := 0.29 \cdot b_{ef.fi}$ $i_{z.fi} = 33.93\ mm$

Slankhet: $\lambda_{z.fi} := \frac{l_{kz}}{i_{z.fi}}$ $\lambda_{z.fi} = 79.576$

[1] (6.21) $\lambda_{rel.z.fi} := \frac{\lambda_{z.fi}}{\pi} \cdot \sqrt{\frac{f_{c.0.k}}{E_{0.05}}}$ $\lambda_{rel.z.fi} = 1.206$

[1] (6.29) $\beta_c := 0.1$ (limtre)

[1] (6.27) $k_{z.fi} := 0.5 \cdot \left(1 + \beta_c \cdot \left(\lambda_{rel.z.fi} - 0.3\right) + \lambda_{rel.z.fi}^{2}\right)$ $k_{z.fi} = 1.273$

$$k_{cz.fi} := \frac{1}{k_{z.fi} + \sqrt{k_{z.fi}^{2} - \lambda_{rel.z.fi}^{2}}}$$ $k_{cz.fi} = 0.595$

[1] 6.1.4 **Aksialspenning**

$A_{fi} := b_{ef.fi} \cdot h_{ef.fi}$

$\sigma_{c.0.d.fi} := \frac{N_{fi}}{A_{fi}}$ $\sigma_{c.0.d.fi} = 9.698\ \frac{N}{mm^2}$

$\frac{\sigma_{c.0.d.fi}}{k_{cz.fi} \cdot f_{c.0.d.fi}} = 0.578$ < 1.0 OK

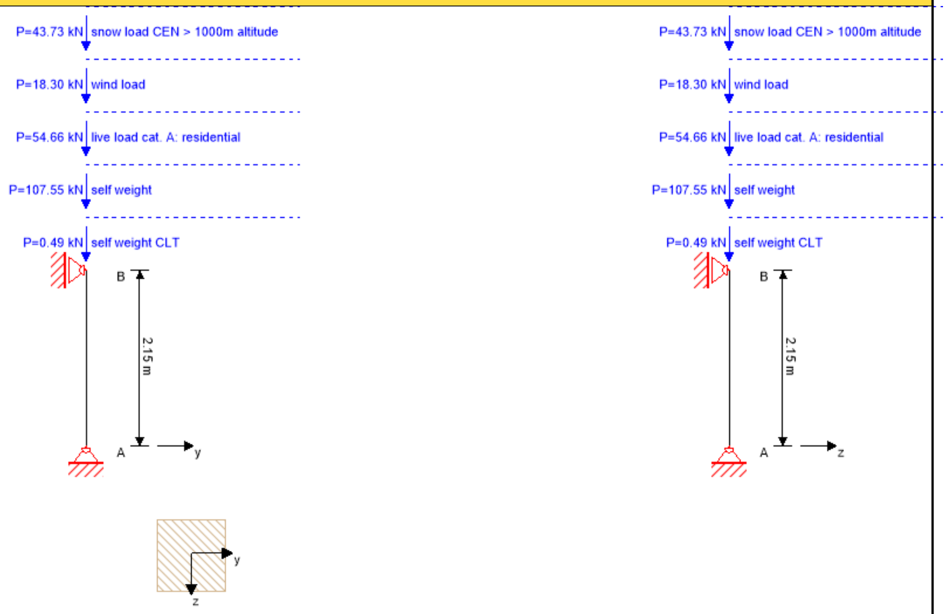
VEDLEGG G5.2

Søyle 3-9				
L	2,15		kcz	0,943
b	215		kcy	0,95
h	225		fcok	24,5
A	48375		Ym	1,15
W	1814062,5		fmk	30

Kombinasjon	1	2a	2b	2c	2d	3a	3b	3c	4a	4b
G	108,00	108,00	108,00	108,00	108,00	108,00	108,00	108,00	108,00	108,00
γG	1,35	1,35	1,20	1,20	1,20	1,35	1,20	1,20	1,35	1,20
S		43,73	43,73	43,73	43,73	43,73	43,73	43,73		
γS		1,05	1,50	1,05	1,05	1,05	1,50	1,05		
Q		18,30	18,30	18,30	18,30					
γQ		1,05	1,05	1,50	1,05					
N		54,66	54,66	54,66	54,66	54,66	54,66	54,66	54,66	54,66
γN		1,05	1,05	1,05	1,50	1,05	1,05	1,50	1,05	1,50
Nf	145,80	268,32	271,80	260,36	276,72	191,72	195,20	175,52	145,80	129,60
σN	3,01	5,55	5,62	5,38	5,72	3,96	4,04	3,63	3,01	2,68
kmod	0,6	1,1	1,1	1,1	1,1	0,9	0,9	0,9	0,8	0,8
fcod	12,78	23,43	23,43	23,43	23,43	19,17	19,17	19,17	17,04	17,04
UF (6.23)	0,25	0,25	0,25	0,24	**0,26**	0,22	0,22	0,20	0,19	0,19
UF (6.24)	0,25	0,25	**0,25**	0,24	**0,26**	0,22	0,22	0,20	0,19	0,19

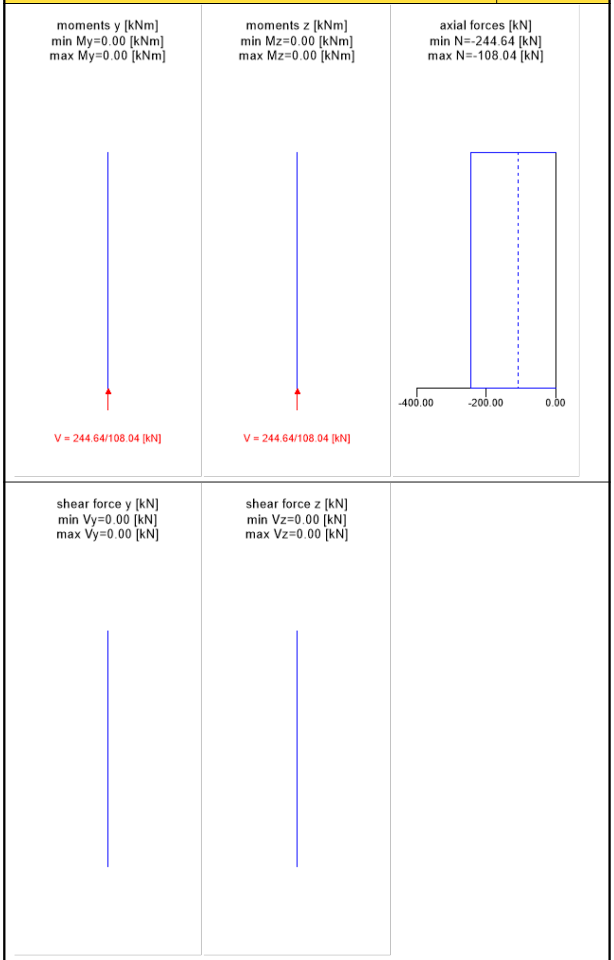
VEDLEGG G5.3

system



section: wooden beam 21.5/22.5; **material**: GL30c; **service class**: service class 1; **fire resistance class**: R 60

utilization — 59 %



flexural stress analysis — 9 %

$M_{y,d}$ =	0.00	kNm	$f_{m,k}$ =	30.00	N/mm²
$N_{c,d}$ =	-244.64	kN	$f_{c,k}$ =	24.50	N/mm²
$\sigma_{c,d}$ =	5.06	N/mm²	$f_{c,d}$ =	17.04	N/mm²
$\sigma_{m,y,d}$ =	0.00	N/mm² <	$f_{m,y,d}$ =	20.87	N/mm² ✓

shear stress analysis Y — 0 %

V_d =	0.00	kN	$f_{v,k}$ =	3.50	N/mm²
$\tau_{v,d}$ =	0.00	N/mm² <	$f_{v,d}$ =	1.84	N/mm² ✓

shear stress analysis Z — 0 %

V_d =	0.00	kN	$f_{v,k}$ =	3.50	N/mm²
$\tau_{v,d}$ =	0.00	N/mm² <	$f_{v,d}$ =	1.84	N/mm² ✓

shear stress analysis combined — 0 %

$V_{y,d}$ =	0.00	kN	$V_{z,d}$ =	0.00	kN
$\tau_{v,y,d}$ =	0.00	N/mm²	$\tau_{v,z,d}$ =	0.00	N/mm²
			ratio =	0 %	✓

lateral torsional buckling analysis — 31 %

$M_{y,d}$ =	0.00	kNm	$f_{m,k}$ =	30.00	N/mm²
$N_{c,d}$ =	-244.64	kN	$f_{c,k}$ =	24.50	N/mm²
$\sigma_{c,d}$ =	5.06	N/mm²	$f_{c,d}$ =	17.04	N/mm²
$\sigma_{m,y,d}$ =	0.00	N/mm² <	$f_{m,y,d}$ =	20.87	N/mm² ✓

buckling analysis — 30 %

$M_{y,d}$ =	0.00	kNm	$f_{m,k}$ =	30.00	N/mm²
$N_{c,d}$ =	-244.64	kN	$f_{c,k}$ =	24.50	N/mm²
$\sigma_{c,d}$ =	5.06	N/mm²	$f_{c,d}$ =	17.04	N/mm²
$\sigma_{m,y,d}$ =	0.00	N/mm²	$f_{m,y,d}$ =	20.87	N/mm²
$\sigma_{m,z,d}$ =	0.00	N/mm² <	$f_{m,z,d}$ =	20.87	N/mm² ✓

flexural stress analysis fire — 12 %

$M_{y,d}$ =	0.00	kNm	$f_{m,k}$ =	30.00	N/mm²
$N_{c,d}$ =	-146.30	kN	$f_{c,k}$ =	24.50	N/mm²
$\sigma_{c,d}$ =	9.85	N/mm²	$f_{c,d}$ =	28.18	N/mm²
$\sigma_{m,y,d}$ =	0.00	N/mm² <	$f_{m,y,d}$ =	34.50	N/mm² ✓

shear stress analysis Y fire — 0 %

V_d =	0.00	kN	$f_{v,k}$ =	3.50	N/mm²
$\tau_{v,d}$ =	0.00	N/mm² <	$f_{v,d}$ =	2.70	N/mm² ✓

shear stress analysis Z fire — 0 %

V_d =	0.00	kN	$f_{v,k}$ =	3.50	N/mm²
$\tau_{v,d}$ =	0.00	N/mm² <	$f_{v,d}$ =	2.70	N/mm² ✓

shear stress analysis combined fire — 0 %

$V_{y,d}$ =	0.00	kN	$V_{z,d}$ =	0.00	kN
$\tau_{v,y,d}$ =	0.00	N/mm²	$\tau_{v,z,d}$ =	0.00	N/mm²
			ratio =	0 %	✓

lateral torsional buckling analysis fire — 59 %

$M_{y,d}$ =	0.00	kNm	$f_{m,k}$ =	30.00	N/mm²
$N_{c,d}$ =	-146.30	kN	$f_{c,k}$ =	24.50	N/mm²
$\sigma_{c,d}$ =	9.85	N/mm²	$f_{c,d}$ =	28.18	N/mm²
$\sigma_{m,y,d}$ =	0.00	N/mm² <	$f_{m,y,d}$ =	34.50	N/mm² ✓

buckling analysis fire — 41 %

$M_{y,d}$ =	0.00	kNm	$f_{m,k}$ =	30.00	N/mm²
$N_{c,d}$ =	-146.30	kN	$f_{c,k}$ =	24.50	N/mm²
$\sigma_{c,d}$ =	9.85	N/mm²	$f_{c,d}$ =	28.18	N/mm²
$\sigma_{m,y,d}$ =	0.00	N/mm²	$f_{m,y,d}$ =	34.50	N/mm²
$\sigma_{m,z,d}$ =	0.00	N/mm² <	$f_{m,z,d}$ =	34.50	N/mm² ✓

support reaction

load case category	k_{mod}	A_y	A_z	B_x	B_y	B_z
		[kN]	[kN]	[kN]	[kN]	[kN]
self weight CLT	0.6	0.00	0.00	0.49	0.00	0.00
		0.00	0.00	0.49	0.00	0.00

support reaction						
load case category	k_{mod}	A_y	A_z	B_x	B_y	B_z
		[kN]	[kN]	[kN]	[kN]	[kN]
self weight	0.6	0.00	0.00	107.55	0.00	0.00
		0.00	0.00	107.55	0.00	0.00
live load cat. A: residential	0.8	0.00	0.00	0.00	0.00	0.00
		0.00	0.00	54.66	0.00	0.00
wind load	0.9	0.00	0.00	0.00	0.00	0.00
		0.00	0.00	18.30	0.00	0.00
snow load CEN > 1000m altitude	0.8	0.00	0.00	0.00	0.00	0.00
		0.00	0.00	43.73	0.00	0.00

VEDLEGG G6.1

SØYLE 2-10

Referanse til standarder

[1] NS-EN 1995-1-1: Allmenne regler og regler for bygninger
[2] NS-EN 1990: Grunnlag for prosjektering av konstruksjoner
[3] NS-EN 1995-1-2: Brannteknisk dimensjonering
[4] Byggeteknisk forskrift (TEK17)
[5] Norsk Limtreprodusenters Forening, *Limtreboka - Beregningseksempler* (2018)

FORUTSETNINGER OG ANTAKELSER

[1] NA.901 Klimaklasse: 1

[1] Tab. 2.1 Lastvarighetsklasse for snølast: Korttidslast
Lastvarighetsklasse for vindlasten: Øyeblikkslast

[1] NA.2.3 Partialfaktor for limtre: $\gamma_M := 1.15$

Antar søyletverrrsnitt: $b := 215\ mm$ $h := 225\ mm$

Lastbredde: $Lb := 6640\ mm$

Lengde: $L := 2150\ mm$

[5] Tab. 6 Limtre GL30c:

$f_{m.k} := 30\ \frac{N}{mm^2}$ $f_{c.90.k} := 2.5\ \frac{N}{mm^2}$ $f_{v.k} := 3.5\ \frac{N}{mm^2}$

$f_{c.0.k} := 24.5\ \frac{N}{mm^2}$ $E_{0.05} := 10800\ \frac{N}{mm^2}$ $\rho_m := 430\ \frac{kg}{m^3}$

$g := 9.81\ \frac{m}{s^2}$

LASTER

(Verdier hente fra Calculatis)

Egenvekt: $G_b := 17.8\ kN + 47.36\ kN \cdot 2$ $G_b = 112.52\ kN$

Egenvekt søyle: $G_s := 4.609\ \frac{kN}{m^3} \cdot b \cdot h \cdot L$ $G_s = 479.365\ N$

Permanent last: $G_k := G_b + G_s$ $G_k = 112.999\ kN$

Snølast: $S_k := 21.3\ kN$

Nyttelast: $N_k := 26.6\ kN \cdot 2$ $N_k = 53.2\ kN$

Vindlast (fra tak): $Q_k := 8.9\ kN$

Vindlast på vegg:

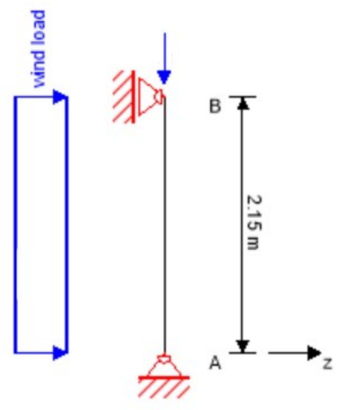
$q_{kast} := 1.34\ \frac{kN}{m^2}$ $C_A := 1.2$ $C_D := 0.724$

$q_{vind1} := q_{kast} \cdot (C_A + 0.2) \cdot Lb$ $q_{vind1} = 12.457\ \frac{kN}{m}$

$q_{vind2} := q_{kast} \cdot (C_D + 0.3) \cdot Lb$ $q_{vind2} = 9.111\ \frac{kN}{m}$

$q_k := \max(q_{vind1}, q_{vind2})$ $q_k = 12.457\ \frac{kN}{m}$

LASTKOMBINASJONER



1. Kun egenlast $k_{mod} := 0.6$

$G_{f1} := 1.35 \cdot G_k$ $G_{f1} = 152.549\ kN$

2. Alle laster $k_{mod} := 1.1$

a) Egenlast dominerende:

$G_{f2} := 1.35 \cdot G_k$ $G_{f2} = 152.549\ kN$

$S_{f2} := 1.05 \cdot S_k$ $S_{f2} = 22.365\ kN$

$N_{f2} := 1.05 \cdot N_k$ $N_{f2} = 55.86\ kN$

$Q_{f2} := 1.05 \cdot Q_k$ $\quad Q_{f2} = 9.345\ kN$

$q_{f2} := 1.05 \cdot q_k$ $\quad q_{f2} = 13.079\ \frac{kN}{m}$

b) Snølast dominerende:

$G_{f3} := 1.2 \cdot G_k$ $\quad G_{f3} = 135.599\ kN$

$S_{f3} := 1.5 \cdot S_k$ $\quad S_{f3} = 31.95\ kN$

$N_{f3} := 1.05 \cdot N_k$ $\quad N_{f3} = 55.86\ kN$

$Q_{f3} := 1.05 \cdot Q_k$ $\quad Q_{f3} = 9.345\ kN$

$q_{f3} := 1.05 \cdot q_k$ $\quad q_{f3} = 13.079\ \frac{kN}{m}$

c) Nyttelast dominerende:

$G_{f4} := 1.2 \cdot G_k$ $\quad G_{f4} = 135.599\ kN$

$S_{f4} := 1.05 \cdot S_k$ $\quad S_{f4} = 22.365\ kN$

$N_{f4} := 1.5 \cdot N_k$ $\quad N_{f4} = 79.8\ kN$

$Q_{f4} := 1.05 \cdot Q_k$ $\quad Q_{f4} = 9.345\ kN$

$q_{f4} := 1.05 \cdot q_k$ $\quad q_{f4} = 13.079\ \frac{kN}{m}$

d) Vindlast dominerende:

$G_{f5} := 1.2 \cdot G_k$ $\quad G_{f5} = 135.599\ kN$

$S_{f5} := 1.05 \cdot S_k$ $\quad S_{f5} = 22.365\ kN$

$N_{f5} := 1.05 \cdot N_k$ $\quad N_{f5} = 55.86\ kN$

$Q_{f5} := 1.5 \cdot Q_k$ $\quad Q_{f5} = 13.35\ kN$

$q_{f5} := 1.5 \cdot q_k$ $\quad q_{f5} = 18.685\ \frac{kN}{m}$

3. Egenlast, nyttelast og snølast $k_{mod} := 0.9$

a) Egenlast dominerende:

$G_{f6} := 1.35 \cdot G_k$ $G_{f6} = 152.549\ kN$

$S_{f6} := 1.05 \cdot S_k$ $S_{f6} = 22.365\ kN$

$N_{f6} := 1.05 \cdot N_k$ $N_{f6} = 55.86\ kN$

b) Snølast dominerende:

$G_{f7} := 1.2 \cdot G_k$ $G_{f7} = 135.599\ kN$

$S_{f7} := 1.5 \cdot S_k$ $S_{f7} = 31.95\ kN$

$N_{f7} := 1.05 \cdot N_k$ $N_{f7} = 55.86\ kN$

c) Nyttelast dominerende:

$G_{f8} := 1.2 \cdot G_k$ $G_{f8} = 135.599\ kN$

$S_{f8} := 1.05 \cdot S_k$ $S_{f8} = 22.365\ kN$

$N_{f8} := 1.5 \cdot N_k$ $N_{f8} = 79.8\ kN$

3. Egenlast og nyttelast $k_{mod} := 0.8$

a) Egenlast dominerende:

$G_{f9} := 1.35 \cdot G_k$ $G_{f9} = 152.549\ kN$

$N_{f9} := 1.05 \cdot N_k$ $N_{f9} = 55.86\ kN$

b) Nyttelast dominerende:

$G_{f10} := 1.35 \cdot G_k$ $G_{f10} = 152.549\ kN$

$N_{f10} := 1.5 \cdot N_k$ $N_{f10} = 79.8\ kN$

[1] (3.2) Høydefaktor: $k_h := min\left(\left(\frac{600\ mm}{h}\right)^{0.1}, 1.1\right)$ $k_h = 1.1$

KNEKKING

y-y: Knekklengde: $l_{ky} := 2150\ mm + 550\ mm$ (søyle + bjelkehøyde)

Treghetsradius: $i_y := 0.29 \cdot h$ $i_y = 65.25\ mm$

Slankhet: $\lambda_y := \frac{l_{ky}}{i_y}$ $\lambda_y = 41.379$

[1] (6.21) $\lambda_{rel.y} := \frac{\lambda_y}{\pi} \cdot \sqrt{\frac{f_{c.0.k}}{E_{0.05}}}$ $\lambda_{rel.y} = 0.627$

[1] (6.29) $\beta_c := 0.1$ (limtre)

[1] (6.27) $k_y := 0.5 \cdot \left(1 + \beta_c \cdot \left(\lambda_{rel.y} - 0.3\right) + {\lambda_{rel.y}}^2\right)$ $k_y = 0.713$

$k_{cy} := \frac{1}{k_y + \sqrt{{k_y}^2 - {\lambda_{rel.y}}^2}}$ $k_{cy} = 0.95$

z-z: Knekklengde: $l_{kz} := 2150\ mm + 550\ mm$

Treghetsradius: $i_z := 0.29 \cdot b$ $i_z = 62.35\ mm$

Slankhet: $\lambda_z := \frac{l_{kz}}{i_z}$ $\lambda_z = 43.304$

[1] (6.21) $\lambda_{rel.z} := \frac{\lambda_z}{\pi} \cdot \sqrt{\frac{f_{c.0.k}}{E_{0.05}}}$ $\lambda_{rel.z} = 0.657$

[1] (6.29) $\beta_c := 0.1$ (limtre)

[1] (6.27) $k_z := 0.5 \cdot \left(1 + \beta_c \cdot \left(\lambda_{rel.z} - 0.3\right) + {\lambda_{rel.z}}^2\right)$ $k_z = 0.733$

$$k_{cz} := \frac{1}{k_z + \sqrt{{k_z}^2 - {\lambda_{rel.z}}^2}} \qquad k_{cz} = 0.943$$

BRANNDIMENSJONERING

[3] Tab 3.1 Dimensjonerende forkullingshastighet for limtre: $\beta_n := 0.7 \frac{mm}{min}$

[4] § 11-3 Brannklasse 2

[4] § 11-4 Brannmotstandstid R60: $t := 60\ min$

[2] Tab. NA.A1.1

$\psi_{0.s} := 0.7 \qquad \psi_{1.s} := 0.5 \qquad \psi_{2.s} := 0.2$

$\psi_{0.v} := 0.6 \qquad \psi_{1.v} := 0.2 \qquad \psi_{2.v} := 0$

$\psi_{0.n} := 0.7 \qquad \psi_{1.n} := 0.5 \qquad \psi_{2.n} := 0.3$

$N_{fi} := G_k + N_k \cdot \psi_{1.n} + \psi_{2.s} \cdot S_k \qquad N_{fi} = 143.859\ kN$

[3] 4.2.2 **Effektiv forkullingsdybde**

$t > 20\ min \qquad k_0 := 1.0$

$d_{char.n} := \beta_n \cdot t$

$d_0 := 7\ mm$

$d_{ef} := d_{char.n} + k_0 \cdot d_0 \qquad d_{ef} = 49\ mm$

Dimensjon effektivt resttverrsnitt

$b_{ef.fi} := b - 2 \cdot d_{ef} \qquad b_{ef.fi} = 117\ mm$

$h_{ef.fi} := h - 2 \cdot d_{ef} \qquad h_{ef.fi} = 127\ mm$

Dimensjonerende fasthet

$f_{c.0.d.fi} := 1.15 \cdot f_{c.0.k}$ $\qquad f_{c.0.d.fi} = 28.175 \ \frac{N}{mm^2}$

$f_{m.d.fi} := 1.15 \cdot f_{m.k}$ $\qquad f_{m.d.fi} = 34.5 \ \frac{N}{mm^2}$

Knekking - brann

y-y: Treghetsradius: $i_{y.fi} := 0.29 \cdot h_{ef.fi}$ $\qquad i_{y.fi} = 36.83 \ mm$

Slankhet: $\lambda_{y.fi} := \frac{l_{ky}}{i_{y.fi}}$ $\qquad \lambda_{y.fi} = 73.31$

[1] (6.21) $\lambda_{rel.y.fi} := \frac{\lambda_{y.fi}}{\pi} \cdot \sqrt{\frac{f_{c.0.k}}{E_{0.05}}}$ $\qquad \lambda_{rel.y.fi} = 1.111$

[1] (6.29) $\beta_c := 0.1$ (limtre)

[1] (6.27) $k_{y.fi} := 0.5 \cdot \left(1 + \beta_c \cdot \left(\lambda_{rel.y.fi} - 0.3\right) + \lambda_{rel.y.fi}^{2}\right)$ $\qquad k_{y.fi} = 1.158$

$k_{cy.fi} := \frac{1}{k_{y.fi} + \sqrt{k_{y.fi}^{2} - \lambda_{rel.y.fi}^{2}}}$ $\qquad k_{cy.fi} = 0.674$

z-z: Treghetsradius: $i_{z.fi} := 0.29 \cdot b_{ef.fi}$ $\qquad i_{z.fi} = 33.93 \ mm$

Slankhet: $\lambda_{z.fi} := \frac{l_{kz}}{i_{z.fi}}$ $\qquad \lambda_{z.fi} = 79.576$

[1] (6.21) $\lambda_{rel.z.fi} := \frac{\lambda_{z.fi}}{\pi} \cdot \sqrt{\frac{f_{c.0.k}}{E_{0.05}}}$ $\qquad \lambda_{rel.z.fi} = 1.206$

[1] (6.29) $\beta_c := 0.1$ (limtre)

[1] (6.27) $k_{z.fi} := 0.5 \cdot \left(1 + \beta_c \cdot \left(\lambda_{rel.z.fi} - 0.3\right) + \lambda_{rel.z.fi}^{2}\right)$ $\qquad k_{z.fi} = 1.273$

$$k_{cz.fi} := \frac{1}{k_{z.fi} + \sqrt{k_{z.fi}^{2} - \lambda_{rel.z.fi}^{2}}} \qquad k_{cz.fi} = 0.595$$

[1] 6.1.6 **Bøyespenning**

$$W := \frac{1}{6} \cdot b_{ef.fi} \cdot h_{ef.fi}^{2}$$

$$M_{y.fi} := \frac{q_k \cdot \psi_{1.v} \cdot L^2}{8} \qquad M_{y.fi} = 1.44\ kN \cdot m$$

$$\sigma_{m.y.fi} := \frac{M_{y.fi}}{W} \qquad \sigma_{m.y.fi} = 4.577\ \frac{N}{mm^2}$$

$$\frac{\sigma_{m.y.fi}}{f_{m.d.fi}} = 0.133$$ < 1.0 OK

[1] 6.1.4 **Aksialspenning**

$$A_{fi} := b_{ef.fi} \cdot h_{ef.fi}$$

$$\sigma_{c.0.d.fi} := \frac{N_{fi}}{A_{fi}} \qquad \sigma_{c.0.d.fi} = 9.682\ \frac{N}{mm^2}$$

$$\frac{\sigma_{c.0.d.fi}}{k_{cz.fi} \cdot f_{c.0.d.fi}} = 0.577$$ < 1.0 OK

Kombinert virkning

[1] 6.1.6 (2) $k_m := 0.7$

[1] (6.23) $$\frac{\sigma_{c.0.d.fi}}{k_{cy.fi} \cdot f_{c.0.d.fi}} + \frac{\sigma_{m.y.fi}}{f_{m.d.fi}} = 0.643$$ < 1.0 OK

[1] (6.24) $$\frac{\sigma_{c.0.d.fi}}{k_{cz.fi} \cdot f_{c.0.d.fi}} + k_m \cdot \frac{\sigma_{m.y.fi}}{f_{m.d.fi}} = 0.67$$ < 1.0 OK

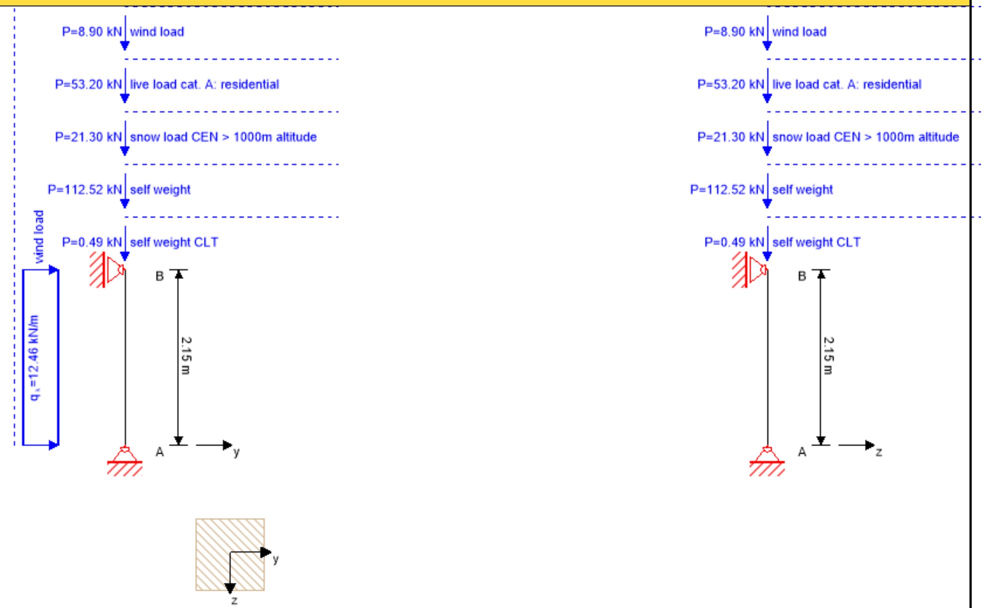
VEDLEGG G6.2

Søyle 2-10							
L	2,15		kcz	0,943		kh	1,1
b	215		kcy	0,95		km	0,7
h	225		fcok	24,5			
A	48375		Ym	1,15			
W	1814062,5		fmk	30			

Kombinasjon	1	2a	2b	2c	2d	3a	3b	3c	4a	4b
G	113,00	113,00	113,00	113,00	113,00	113,00	113,00	113,00	113,00	113,00
γG	1,35	1,35	1,20	1,20	1,20	1,35	1,20	1,20	1,20	1,35
S		21,30	21,30	21,30	21,30	21,30	21,30	21,30		
γS		1,05	1,50	1,05	1,05	1,05	1,50	1,05		
N		53,20	53,20	53,20	53,20	53,20	53,20	53,20	53,20	53,20
γN		1,05	1,05	1,50	1,05	1,05	1,05	1,50	1,50	1,05
Q		8,90	8,90	8,90	8,90					
γQ		1,05	1,05	1,05	1,50					
q		12,46	12,46	12,46	12,46					
γq		1,05	1,05	1,05	1,50					
Nf	152,55	197,34	176,90	167,31	227,18	174,92	167,55	157,97	215,40	208,41
qf	0,00	13,08	13,08	13,08	18,69	0,00	0,00	0,00	0,00	0,00
Mf	0,00	7,56	7,56	7,56	10,80	0,00	0,00	0,00	0,00	0,00
σN	3,15	4,08	3,66	3,46	4,70	3,62	3,46	3,27	4,45	4,31
σM	0,00	4,17	4,17	4,17	5,95	0,00	0,00	0,00	0,00	0,00
kmod	0,6	1,1	1,1	1,1	1,1	0,9	0,9	0,9	0,8	0,8
fcod	12,78	23,43	23,43	23,43	23,43	19,17	19,17	19,17	17,04	17,04
fmyd	17,22	31,57	31,57	31,57	31,57	25,83	21,09	21,09	18,75	18,75
UF (6.23)	0,26	0,32	0,30	0,29	**0,40**	0,20	0,19	0,21	0,28	0,27
UF (6.24)	0,26	0,28	0,26	0,25	0,34	0,20	0,17	0,16	0,25	0,24

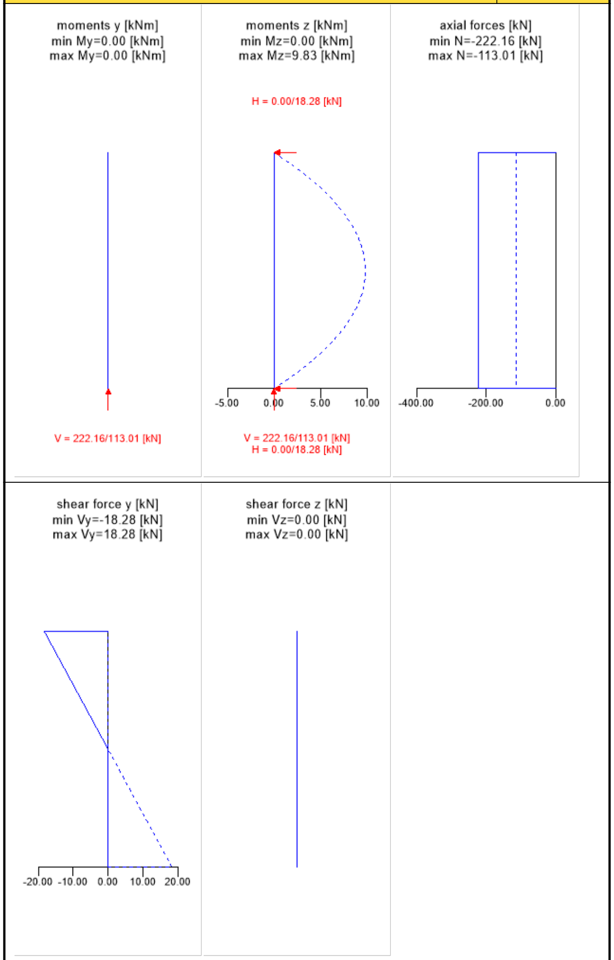
VEDLEGG G6.3

system



section: wooden beam 21.5/22.5; **material**: GL30c; **service class**: service class 1; **fire resistance class**: R 60

utilization — 58 %



flexural stress analysis — 30 %

$M_{y,d}$ =	0.00	kNm	$f_{m,k}$ =	30.00	N/mm²	
$N_{c,d}$ =	-	kN	$f_{c,k}$ =	24.50	N/mm²	
	222.16					
$\sigma_{c,d}$ =	4.59	N/mm²	$f_{c,d}$ =	19.17	N/mm²	
$\sigma_{m,y,d}$ =	0.00	N/mm²<	$f_{m,y,d}$ =	23.48	N/mm²	✓

shear stress analysis Y — 31 %

V_d =	18.28	kN	$f_{v,k}$ =	3.50	N/mm²	
$\tau_{v,d}$ =	0.57	N/mm²<	$f_{v,d}$ =	1.84	N/mm²	✓

shear stress analysis Z — 0 %

V_d =	0.00	kN	$f_{v,k}$ =	3.50	N/mm²	
$\tau_{v,d}$ =	0.00	N/mm²<	$f_{v,d}$ =	1.84	N/mm²	✓

shear stress analysis combined — 10 %

$V_{y,d}$ =	18.28	kN	$V_{z,d}$ =	0.00	kN	
$\tau_{v,y,d}$ =	0.57	N/mm²	$\tau_{v,z,d}$ =	0.00	N/mm²	
			ratio =	10 %		✓

lateral torsional buckling analysis — 30 %

$M_{y,d}$ =	0.00	kNm	$f_{m,k}$ =	30.00	N/mm²	
$N_{c,d}$ =	-	kN	$f_{c,k}$ =	24.50	N/mm²	
	231.80					
$\sigma_{c,d}$ =	4.79	N/mm²	$f_{c,d}$ =	17.04	N/mm²	
$\sigma_{m,y,d}$ =	0.00	N/mm²<	$f_{m,y,d}$ =	20.87	N/mm²	✓

buckling analysis — 49 %

$M_{y,d}$ =	0.00	kNm	$f_{m,k}$ =	30.00	N/mm²	
$N_{c,d}$ =	-	kN	$f_{c,k}$ =	24.50	N/mm²	
	222.16					
$\sigma_{c,d}$ =	4.59	N/mm²	$f_{c,d}$ =	19.17	N/mm²	
$\sigma_{m,y,d}$ =	0.00	N/mm²	$f_{m,y,d}$ =	23.48	N/mm²	
$\sigma_{m,z,d}$ =	5.67	N/mm²<	$f_{m,z,d}$ =	23.48	N/mm²	✓

flexural stress analysis fire — 25 %

$M_{y,d}$ =	0.00	kNm	$f_{m,k}$ =	30.00	N/mm²	
$N_{c,d}$ =	-	kN	$f_{c,k}$ =	24.50	N/mm²	
	135.01					
$\sigma_{c,d}$ =	9.09	N/mm²	$f_{c,d}$ =	28.18	N/mm²	
$\sigma_{m,y,d}$ =	0.00	N/mm²<	$f_{m,y,d}$ =	34.50	N/mm²	✓

shear stress analysis Y fire — 10 %

V_d =	2.68	kN	$f_{v,k}$ =	3.50	N/mm²	
$\tau_{v,d}$ =	0.27	N/mm²<	$f_{v,d}$ =	2.70	N/mm²	✓

shear stress analysis Z fire — 0 %

V_d =	0.00	kN	$f_{v,k}$ =	3.50	N/mm²	
$\tau_{v,d}$ =	0.00	N/mm²<	$f_{v,d}$ =	2.70	N/mm²	✓

shear stress analysis combined fire — 1 %

$V_{y,d}$ =	2.68	kN	$V_{z,d}$ =	0.00	kN	
$\tau_{v,y,d}$ =	0.27	N/mm²	$\tau_{v,z,d}$ =	0.00	N/mm²	
			ratio =	1 %		✓

lateral torsional buckling analysis fire — 58 %

$M_{y,d}$ =	0.00	kNm	$f_{m,k}$ =	30.00	N/mm²	
$N_{c,d}$ =	-	kN	$f_{c,k}$ =	24.50	N/mm²	
	143.87					
$\sigma_{c,d}$ =	9.68	N/mm²	$f_{c,d}$ =	28.18	N/mm²	
$\sigma_{m,y,d}$ =	0.00	N/mm²<	$f_{m,y,d}$ =	34.50	N/mm²	✓

buckling analysis fire — 52 %

$M_{y,d}$ =	0.00	kNm	$f_{m,k}$ =	30.00	N/mm²	
$N_{c,d}$ =	-	kN	$f_{c,k}$ =	24.50	N/mm²	
	135.01					
$\sigma_{c,d}$ =	9.09	N/mm²	$f_{c,d}$ =	28.18	N/mm²	
$\sigma_{m,y,d}$ =	0.00	N/mm²	$f_{m,y,d}$ =	34.50	N/mm²	
$\sigma_{m,z,d}$ =	4.97	N/mm²<	$f_{m,z,d}$ =	34.50	N/mm²	✓

support reaction

load case category	k_{mod}	A_y	A_z	B_x	B_y	B_z
		[kN]	[kN]	[kN]	[kN]	[kN]
self weight CLT	0.6	0.00	0.00	0.49	0.00	0.00
		0.00	0.00	0.49	0.00	0.00

support reaction						
load case category	k_{mod}	A_y	A_z	B_x	B_y	B_z
		[kN]	[kN]	[kN]	[kN]	[kN]
self weight	0.6	0.00	0.00	112.52	0.00	0.00
		0.00	0.00	112.52	0.00	0.00
snow load CEN > 1000m altitude	0.8	0.00	0.00	0.00	0.00	0.00
		0.00	0.00	21.30	0.00	0.00
live load cat. A: residential	0.8	0.00	0.00	0.00	0.00	0.00
		0.00	0.00	53.20	0.00	0.00
wind load	0.9	13.39	0.00	0.00	13.39	0.00
		0.00	0.00	8.90	0.00	0.00

Disclaimer

VEDLEGG G7.1

SØYLE 2-9

Referanse til standarder

[1] NS-EN 1995-1-1: Allmenne regler og regler for bygninger
[2] NS-EN 1990: Grunnlag for prosjektering av konstruksjoner
[3] NS-EN 1992-1-2: Brannteknisk dimensjonering
[4] Byggeteknisk forskrift (TEK17)
[5] Norsk Limtreprodusenters Forening, *Limtreboka - Beregningseksempler* (2018)

FORUTSETNINGER OG ANTAKELSER

[1] NA.901 Klimaklasse: 1

[1] Tab. 2.1 Lastvarighetsklasse for snølast: Korttidslast
Lastvarighetsklasse for vindlasten: Øyeblikkslast

[1] NA.2.3 Partialfaktor for limtre: $\gamma_M := 1.15$

Antar søyletverrrsnitt: $b := 215\ mm$ $h := 225\ mm$

Lengde: $L := 2150\ mm$

[5] Tab. 6 Limtre GL30c:

$f_{m.k} := 30\ \frac{N}{mm^2}$ $f_{c.90} := 2.5\ \frac{N}{mm^2}$ $f_{v.k} := 3.5\ \frac{N}{mm^2}$

$f_{c.0.k} := 24.5\ \frac{N}{mm^2}$ $E_{0.05} := 10800\ \frac{N}{mm^2}$ $\rho_m := 430\ \frac{kg}{m^3}$

$g := 9.81\ \frac{m}{s^2}$

LASTER

(Verdier hentet fra Calculatis)

Egenvekt: $G_b := 108.3\ kN + 75.41\ kN$ $G_b = 183.71\ kN$

Egenvekt søyle: $G_s := 4.609\ \frac{kN}{m^3} \cdot b \cdot h \cdot L$ $G_s = 479.365\ N$

Permanent last: $G_k := G_b + G_s$ $G_k = 184.189\ kN$

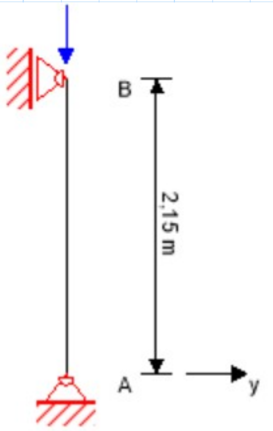
Snølast: $S_k := 43.73\ kN$

Vindlast : $Q_k := 18.3\ kN$

Nyttelast: $N_k := 54.66\ kN \cdot 2$ $N_k = 109.32\ kN$

LASTKOMBINASJONER

1. Kun egenlast $k_{mod} := 0.6$



$G_{f1} := 1.35 \cdot G_k$ $G_{f1} = 248.656\ kN$

2. Alle laster $k_{mod} := 1.1$

a) Egenlast dominerende:

$G_{f2} := 1.35 \cdot G_k$ $G_{f2} = 248.656\ kN$

$S_{f2} := 1.05 \cdot S_k$ $S_{f2} = 45.917\ kN$

$Q_{f2} := 1.05 \cdot Q_k$ $Q_{f2} = 19.215\ kN$

$N_{f2} := 1.05 \cdot N_k$ $N_{f2} = 114.786\ kN$

b) Snølast dominerende:

$G_{f3} := 1.2 \cdot G_k$ $G_{f3} = 221.027\ kN$

$S_{f3} := 1.5 \cdot S_k$ $S_{f3} = 65.595\ kN$

$Q_{f3} := 1.05 \cdot Q_k$ $Q_{f3} = 19.215\ kN$

$N_{f3} := 1.05 \cdot N_k$ $N_{f3} = 114.786\ kN$

c) Vindlast dominerende:

$G_{f4} := 1.2 \cdot G_k$ $G_{f4} = 221.027\ kN$

$S_{f4} := 1.05 \cdot S_k$ $S_{f4} = 45.917\ kN$

$Q_{f4} := 1.5 \cdot Q_k$ $Q_{f4} = 27.45\ kN$

$N_{f4} := 1.05 \cdot N_k$ $N_{f4} = 114.786\ kN$

d) Nyttelast dominerende:

$G_{f5} := 1.2 \cdot G_k$ $G_{f5} = 221.027\ kN$

$S_{f5} := 1.05 \cdot S_k$ $S_{f5} = 45.917\ kN$

$Q_{f5} := 1.05 \cdot Q_k$ $Q_{f5} = 19.215\ kN$

$N_{f5} := 1.5 \cdot N_k$ $N_{f5} = 163.98\ kN$

3. Egenlast, nyttelast og snølast $k_{mod} := 0.9$

a) Egenlast dominerende:

$G_{f6} := 1.35 \cdot G_k$ $G_{f6} = 248.656\ kN$

$S_{f6} := 1.05 \cdot S_k$ $S_{f6} = 45.917\ kN$

$N_{f6} := 1.05 \cdot N_k$ $N_{f6} = 114.786\ kN$

b) Snølast dominerende:

$G_{f7} := 1.2 \cdot G_k$ $G_{f7} = 221.027\ kN$

$S_{f7} := 1.5 \cdot S_k$ $S_{f7} = 65.595\ kN$

$N_{f7} := 1.05 \cdot N_k$ $N_{f7} = 114.786\ kN$

c) Nyttelast dominerende:

$G_{f8} := 1.2 \cdot G_k$ $G_{f8} = 221.027\ kN$

$S_{f8} := 1.05 \cdot S_k$ $S_{f8} = 45.917\ kN$

$N_{f8} := 1.5 \cdot N_k$ $N_{f8} = 163.98\ kN$

4. Egenlast og nyttelast $k_{mod} := 0.8$

a) Egenlast dominerende:

$G_{f9} := 1.35 \cdot G_k$ $G_{f9} = 248.656\ kN$

$N_{f9} := 1.05 \cdot N_k$ $N_{f9} = 114.786\ kN$

b) Nyttelast dominerende:

$G_{f10} := 1.2 \cdot G_k$ $G_{f10} = 221.027\ kN$

$N_{f10} := 1.5 \cdot N_k$ $N_{f10} = 163.98\ kN$

KNEKKING

y-y: Knekklengde: $l_{ky} := 2150\ mm + 550\ mm$ (Søyle + bjelkehøyde)

Treghetsradius: $i_y := 0.29 \cdot h$ $i_y = 65.25\ mm$

Slankhet: $\lambda_y := \dfrac{l_{ky}}{i_y}$ $\lambda_y = 41.379$

[1] (6.21) $\lambda_{rel.y} := \dfrac{\lambda_y}{\pi} \cdot \sqrt{\dfrac{f_{c.0.k}}{E_{0.05}}}$ $\lambda_{rel.y} = 0.627$

[1] (6.29) $\beta_c := 0.1$ (limtre)

[1] (6.27) $k_y := 0.5 \cdot \left(1 + \beta_c \cdot \left(\lambda_{rel.y} - 0.3\right) + \lambda_{rel.y}^{\ 2}\right)$ $k_y = 0.713$

$k_{cy} := \dfrac{1}{k_y + \sqrt{k_y^{\ 2} - \lambda_{rel.y}^{\ 2}}}$ $k_{cy} = 0.95$

z-z: Knekklengde: $l_{kz} := 2150\ mm + 550\ mm$ (Søyle + bjelkehøyde)

Treghetsradius: $i_z := 0.29 \cdot b$ $i_z = 62.35\ mm$

Slankhet: $\lambda_z := \frac{l_{kz}}{i_z}$ $\lambda_z = 43.304$

[1] (6.21) $\lambda_{rel.z} := \frac{\lambda_z}{\pi} \cdot \sqrt{\frac{f_{c.0.k}}{E_{0.05}}}$ $\lambda_{rel.z} = 0.657$

[1] (6.29) $\beta_c := 0.1$ (limtre)

[1] (6.27) $k_z := 0.5 \cdot \left(1 + \beta_c \cdot \left(\lambda_{rel.z} - 0.3\right) + \lambda_{rel.z}^{2}\right)$ $k_z = 0.733$

$k_{cz} := \frac{1}{k_z + \sqrt{k_z^{2} - \lambda_{rel.z}^{2}}}$ $k_{cz} = 0.943$

BRANNDIMENSJONERING

[3] Tab 3.1 Dimensjonerende forkullingshastighet for limtre: $\beta_n := 0.7\ \frac{mm}{min}$

[4] § 11-3 Brannklasse 2

[4] § 11-4 Brannmotstandstid R60: $t := 60\ min$

[2] Tab. NA.A1.1

$\psi_{0.s} := 0.7$ $\psi_{1.s} := 0.5$ $\psi_{2.s} := 0.2$

$\psi_{0.v} := 0.6$ $\psi_{1.v} := 0.2$ $\psi_{2.v} := 0$

$\psi_{0.n} := 0.7$ $\psi_{1.n} := 0.5$ $\psi_{2.n} := 0.3$

$N_{fi} := G_k + \psi_{1.n} \cdot N_k + \psi_{2.s} \cdot S_k + \psi_{2.v} \cdot Q_k$ $N_{fi} = 247.595\ kN$

[3] 4.2.2 **Effektiv forkullingsdybde**

$t > 20\ min$ $\quad k_0 := 1.0$

$d_{char.n} := \beta_n \cdot t$

$d_0 := 7\ mm$

$d_{ef} := d_{char.n} + k_0 \cdot d_0$ $\quad d_{ef} = 49\ mm$

Dimensjon effektivt resttverrsnitt

$b_{ef.fi} := b - 2 \cdot d_{ef}$ $\quad b_{ef.fi} = 117\ mm$

$h_{ef.fi} := h - 2 \cdot d_{ef}$ $\quad h_{ef.fi} = 127\ mm$

Dimensjonerende fasthet

$f_{c.0.d.fi} := 1.15 \cdot f_{c.0.k}$ $\quad f_{c.0.d.fi} = 28.175\ \frac{N}{mm^2}$

$f_{m.d.fi} := 1.15 \cdot f_{m.k}$ $\quad f_{m.d.fi} = 34.5\ \frac{N}{mm^2}$

Knekking - brann

y-y: Treghetsradius: $i_{y.fi} := 0.29 \cdot h_{ef.fi}$ $\quad i_{y.fi} = 36.83\ mm$

Slankhet: $\lambda_{y.fi} := \frac{l_{ky}}{i_{y.fi}}$ $\quad \lambda_{y.fi} = 73.31$

[1] (6.21) $\lambda_{rel.y.fi} := \frac{\lambda_{y.fi}}{\pi} \cdot \sqrt{\frac{f_{c.0.k}}{E_{0.05}}}$ $\quad \lambda_{rel.y.fi} = 1.111$

[1] (6.29) $\beta_c := 0.1$ (limtre)

[1] (6.27) $k_{y.fi} := 0.5 \cdot \left(1 + \beta_c \cdot \left(\lambda_{rel.y.fi} - 0.3\right) + \lambda_{rel.y.fi}^{2}\right)$ $\quad k_{y.fi} = 1.158$

$$k_{cy.fi} := \frac{1}{k_{y.fi} + \sqrt{{k_{y.fi}}^2 - {\lambda_{rel.y.fi}}^2}}$$ $k_{cy.fi} = 0.674$

z-z: Treghetsradius: $i_{z.fi} := 0.29 \cdot b_{ef.fi}$ $i_{z.fi} = 33.93\ mm$

Slankhet: $\lambda_{z.fi} := \frac{l_{kz}}{i_{z.fi}}$ $\lambda_{z.fi} = 79.576$

[1] (6.21) $\lambda_{rel.z.fi} := \frac{\lambda_{z.fi}}{\pi} \cdot \sqrt{\frac{f_{c.0.k}}{E_{0.05}}}$ $\lambda_{rel.z.fi} = 1.206$

[1] (6.29) $\beta_c := 0.1$ (limtre)

[1] (6.27) $k_{z.fi} := 0.5 \cdot \left(1 + \beta_c \cdot \left(\lambda_{rel.z.fi} - 0.3\right) + {\lambda_{rel.z.fi}}^2\right)$ $k_{z.fi} = 1.273$

$$k_{cz.fi} := \frac{1}{k_{z.fi} + \sqrt{{k_{z.fi}}^2 - {\lambda_{rel.z.fi}}^2}}$$ $k_{cz.fi} = 0.595$

[1] 6.1.4 **Aksialspenning**

$A_{fi} := b_{ef.fi} \cdot h_{ef.fi}$

$\sigma_{c.0.d.fi} := \frac{N_{fi}}{A_{fi}}$ $\sigma_{c.0.d.fi} = 16.663\ \frac{N}{mm^2}$

$\frac{\sigma_{c.0.d.fi}}{k_{cz.fi} \cdot f_{c.0.d.fi}} = 0.993$ < 1.0 OK

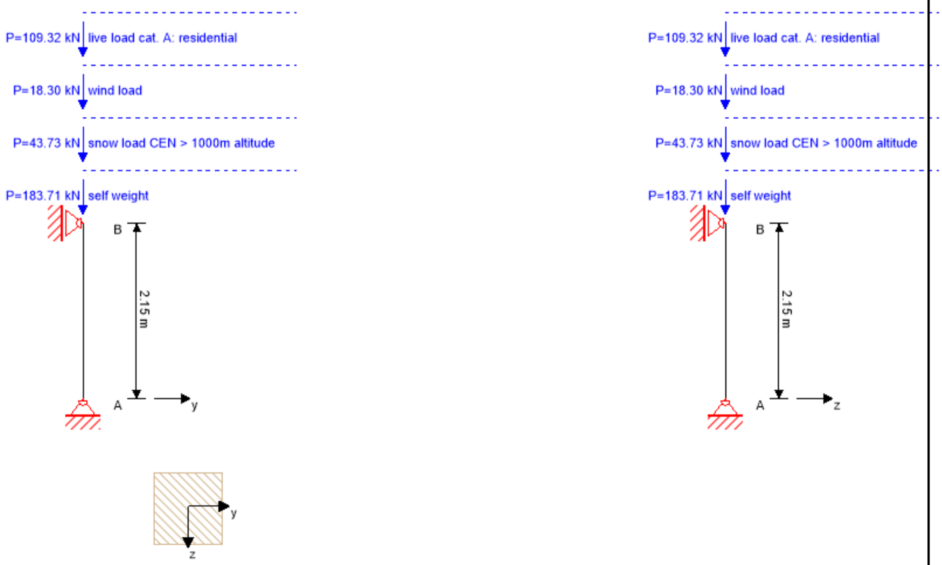
VEDLEGG G7.2

Søyle 2-9				
L	2,15		kcz	0,943
b	220		kcy	0,95
h	220		fcok	24,5
A	48400		Ym	1,15
W	1774666,67		fmk	30

Kombinasjon	1	2a	2b	2c	2d	3a	3b	3c	4a	4b
G	184,20	184,20	184,20	184,20	184,20	184,20	184,20	184,20	184,20	184,20
γG	1,35	1,35	1,20	1,20	1,20	1,35	1,20	1,20	1,35	1,20
S		43,73	43,73	43,73	43,73	43,73	43,73	43,73		
γS		1,05	1,50	1,05	1,05	1,05	1,50	1,05		
Q		18,30	18,30	18,30	18,30					
γQ		1,05	1,05	1,50	1,05					
N		109,32	109,32	109,32	109,32	109,32	109,32	109,32	109,32	109,32
γN		1,05	1,05	1,05	1,50	1,05	1,05	1,50	1,05	1,50
Nf	248,67	428,59	420,64	409,19	450,15	294,59	286,64	266,96	248,67	221,04
σN	5,14	8,86	8,69	8,45	9,30	6,09	5,92	5,52	5,14	4,57
kmod	0,6	1,1	1,1	1,1	1,1	0,9	0,9	0,9	0,8	0,8
fcod	12,78	23,43	23,43	23,43	23,43	19,17	19,17	19,17	17,04	17,04
UF (6.23)	0,42	0,40	0,39	0,38	0,42	0,33	0,33	0,30	0,32	0,32
UF (6.24)	**0,43**	0,40	0,39	0,38	0,42	0,34	0,33	0,31	0,32	0,32

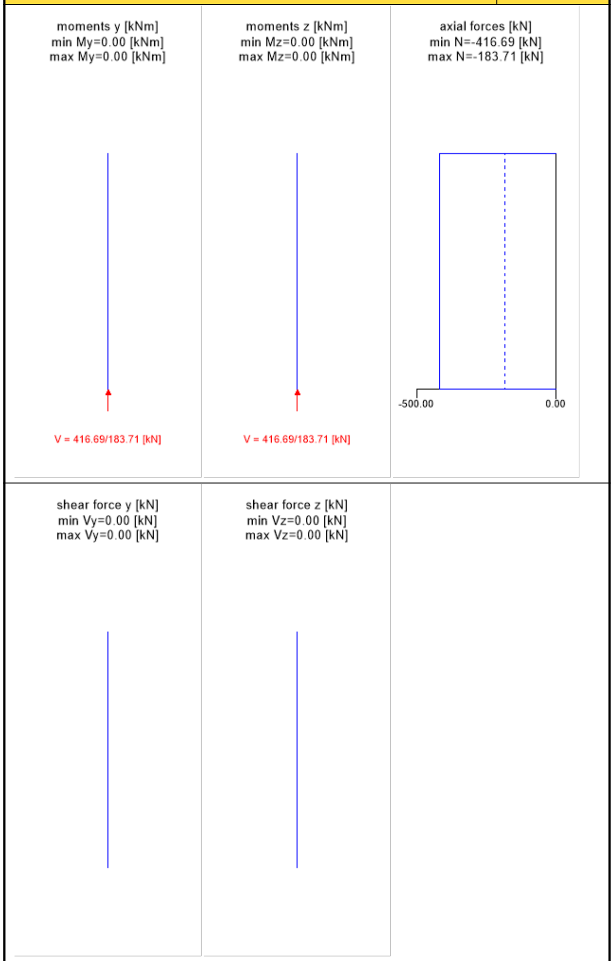
VEDLEGG G7.3

system



section: wooden beam 21.5/22.5; **material**: GL30c; **service class**: service class 1; **fire resistance class**: R 60

utilization 100 %



flexural stress analysis 26 %

$M_{y,d}$ =	0.00	kNm	$f_{m,k}$ =	30.00	N/mm²	
$N_{c,d}$ =	- 416.69	kN	$f_{c,k}$ =	24.50	N/mm²	
$\sigma_{c,d}$ =	8.61	N/mm²	$f_{c,d}$ =	17.04	N/mm²	
$\sigma_{m,y,d}$ =	0.00	N/mm² <	$f_{m,y,d}$ =	20.87	N/mm²	✓

shear stress analysis Y 0 %

V_d =	0.00	kN	$f_{v,k}$ =	3.50	N/mm²	
$\tau_{v,d}$ =	0.00	N/mm² <	$f_{v,d}$ =	1.84	N/mm²	✓

shear stress analysis Z 0 %

V_d =	0.00	kN	$f_{v,k}$ =	3.50	N/mm²	
$\tau_{v,d}$ =	0.00	N/mm² <	$f_{v,d}$ =	1.84	N/mm²	✓

shear stress analysis combined 0 %

$V_{y,d}$ =	0.00	kN	$V_{z,d}$ =	0.00	kN	
$\tau_{v,y,d}$ =	0.00	N/mm²	$\tau_{v,z,d}$ =	0.00	N/mm²	
			ratio =	0 %		✓

lateral torsional buckling analysis 54 %

$M_{y,d}$ =	0.00	kNm	$f_{m,k}$ =	30.00	N/mm²	
$N_{c,d}$ =	- 416.69	kN	$f_{c,k}$ =	24.50	N/mm²	
$\sigma_{c,d}$ =	8.61	N/mm²	$f_{c,d}$ =	17.04	N/mm²	
$\sigma_{m,y,d}$ =	0.00	N/mm² <	$f_{m,y,d}$ =	20.87	N/mm²	✓

buckling analysis 52 %

$M_{y,d}$ =	0.00	kNm	$f_{m,k}$ =	30.00	N/mm²	
$N_{c,d}$ =	- 416.69	kN	$f_{c,k}$ =	24.50	N/mm²	
$\sigma_{c,d}$ =	8.61	N/mm²	$f_{c,d}$ =	17.04	N/mm²	
$\sigma_{m,y,d}$ =	0.00	N/mm²	$f_{m,y,d}$ =	20.87	N/mm²	
$\sigma_{m,z,d}$ =	0.00	N/mm² <	$f_{m,z,d}$ =	20.87	N/mm²	✓

flexural stress analysis fire 35 %

$M_{y,d}$ =	0.00	kNm	$f_{m,k}$ =	30.00	N/mm²	
$N_{c,d}$ =	- 247.12	kN	$f_{c,k}$ =	24.50	N/mm²	
$\sigma_{c,d}$ =	16.63	N/mm²	$f_{c,d}$ =	28.18	N/mm²	
$\sigma_{m,y,d}$ =	0.00	N/mm² <	$f_{m,y,d}$ =	34.50	N/mm²	✓

shear stress analysis Y fire 0 %

V_d =	0.00	kN	$f_{v,k}$ =	3.50	N/mm²	
$\tau_{v,d}$ =	0.00	N/mm² <	$f_{v,d}$ =	2.70	N/mm²	✓

shear stress analysis Z fire 0 %

V_d =	0.00	kN	$f_{v,k}$ =	3.50	N/mm²	
$\tau_{v,d}$ =	0.00	N/mm² <	$f_{v,d}$ =	2.70	N/mm²	✓

shear stress analysis combined fire 0 %

$V_{y,d}$ =	0.00	kN	$V_{z,d}$ =	0.00	kN	
$\tau_{v,y,d}$ =	0.00	N/mm²	$\tau_{v,z,d}$ =	0.00	N/mm²	
			ratio =	0 %		✓

lateral torsional buckling analysis fire 100 %

$M_{y,d}$ =	0.00	kNm	$f_{m,k}$ =	30.00	N/mm²	
$N_{c,d}$ =	- 247.12	kN	$f_{c,k}$ =	24.50	N/mm²	
$\sigma_{c,d}$ =	16.63	N/mm²	$f_{c,d}$ =	28.18	N/mm²	
$\sigma_{m,y,d}$ =	0.00	N/mm² <	$f_{m,y,d}$ =	34.50	N/mm²	✓

buckling analysis fire 70 %

$M_{y,d}$ =	0.00	kNm	$f_{m,k}$ =	30.00	N/mm²	
$N_{c,d}$ =	- 247.12	kN	$f_{c,k}$ =	24.50	N/mm²	
$\sigma_{c,d}$ =	16.63	N/mm²	$f_{c,d}$ =	28.18	N/mm²	
$\sigma_{m,y,d}$ =	0.00	N/mm²	$f_{m,y,d}$ =	34.50	N/mm²	
$\sigma_{m,z,d}$ =	0.00	N/mm² <	$f_{m,z,d}$ =	34.50	N/mm²	✓

support reaction

load case category	k_{mod}	A_y	A_z	B_x	B_y	B_z
		[kN]	[kN]	[kN]	[kN]	[kN]
self weight	0.6	0.00	0.00	183.71	0.00	0.00
		0.00	0.00	183.71	0.00	0.00

storaenso

support reaction

load case category	k_{mod}	A_y	A_z	B_x	B_y	B_z
		[kN]	[kN]	[kN]	[kN]	[kN]
snow load CEN > 1000m altitude	0.8	0.00	0.00	0.00	0.00	0.00
		0.00	0.00	43.73	0.00	0.00
wind load	0.9	0.00	0.00	0.00	0.00	0.00
		0.00	0.00	18.30	0.00	0.00
live load cat. A: residential	0.8	0.00	0.00	0.00	0.00	0.00
		0.00	0.00	109.32	0.00	0.00

Disclaimer

VEDLEGG G8.1

SØYLE 2-6

Referanse til standarder

[1] NS-EN 1995-1-1: Allmenne regler og regler for bygninger
[2] NS-EN 1990: Grunnlag for prosjektering av konstruksjoner
[3] NS-EN 1992-1-2: Brannteknisk dimensjonering
[4] Byggeteknisk forskrift (TEK17)
[5] Norsk Limtreprodusenters Forening, *Limtreboka - Beregningseksempler* (2018)

FORUTSETNINGER OG ANTAKELSER

[1] NA.901 Klimaklasse: 1

[1] Tab. 2.1 Lastvarighetsklasse for snølast: Korttidslast
Lastvarighetsklasse for vindlasten: Øyeblikkslast

[1] NA.2.3 Partialfaktor for limtre: $\gamma_M := 1.15$

Antar søyletverrrsnitt: $b := 215\ mm$ $h := 225\ mm$

Lengde: $L := 2150\ mm$

[5] Tab. 6 Limtre GL30c:

$f_{m.k} := 30\ \frac{N}{mm^2}$ $f_{c.90} := 2.5\ \frac{N}{mm^2}$ $f_{v.k} := 3.5\ \frac{N}{mm^2}$

$f_{c.0.k} := 24.5\ \frac{N}{mm^2}$ $E_{0.05} := 10800\ \frac{N}{mm^2}$ $\rho_m := 430\ \frac{kg}{m^3}$

$g := 9.81\ \frac{m}{s^2}$

LASTER (Verdier hentet fra Calculatis)

Egenvekt: $G_{tb} := 27.51\ kN + 62.61\ kN$ $G_{tb} = 90.12\ kN$

Egenvekt søyle: $G_s := 4.609\ \frac{kN}{m^3} \cdot b \cdot h \cdot L$ $G_s = 479.365\ N$

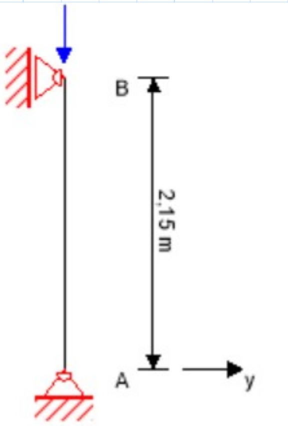
Permanent last: $G_k := G_{tb} + G_s$ $G_k = 90.599\ kN$

Snølast: $S_k := 81.15\ kN$

Vindlast (fra tak): $Q_k := 15.24\ kN$

Nyttelast: $N_k := 45.45\ kN$

LASTKOMBINASJONER



1. Kun egenlast $k_{mod} := 0.6$

$G_{f1} := 1.35 \cdot G_k$ $G_{f1} = 122.309\ kN$

2. Alle laster $k_{mod} := 1.1$

a) Egenlast dominerende:

$G_{f2} := 1.35 \cdot G_k$ $G_{f2} = 122.309\ kN$

$N_{f2} := 1.05 \cdot N_k$ $N_{f2} = 47.723\ kN$

$S_{f2} := 1.05 \cdot S_k$ $S_{f2} = 85.208\ kN$

$Q_{f2} := 1.05 \cdot Q_k$ $Q_{f2} = 16.002\ kN$

b) Snølast dominerende:

$G_{f3} := 1.2 \cdot G_k$ $G_{f3} = 108.719\ kN$

$N_{f3} := 1.05 \cdot N_k$ $N_{f3} = 47.723\ kN$

$S_{f3} := 1.5 \cdot S_k$ $S_{f3} = 121.725\ kN$

$Q_{f3} := 1.05 \cdot Q_k$ $Q_{f3} = 16.002\ kN$

c) Vindlast dominerende:

$G_{f4} := 1.2 \cdot G_k$ $G_{f4} = 108.719\ kN$

$N_{f4} := 1.05 \cdot N_k$ $N_{f4} = 47.723\ kN$

$S_{f4} := 1.05 \cdot S_k$ $S_{f4} = 85.208\ kN$

$Q_{f4} := 1.5 \cdot Q_k$ $Q_{f4} = 22.86\ kN$

d) Nyttelast dominerende:

$G_{f5} := 1.2 \cdot G_k$ $G_{f5} = 108.719\ kN$

$N_{f5} := 1.5 \cdot N_k$ $N_{f5} = 68.175\ kN$

$S_{f5} := 1.05 \cdot S_k$ $S_{f5} = 85.208\ kN$

$Q_{f5} := 1.05 \cdot Q_k$ $Q_{f5} = 16.002\ kN$

3. Egenlast, nyttelast og snølast $k_{mod} := 0.9$

a) Egenlast dominerende:

$G_{f6} := 1.35 \cdot G_k$ $G_{f6} = 122.309\ kN$

$N_{f6} := 1.05 \cdot N_k$ $N_{f6} = 47.723\ kN$

$S_{f6} := 1.05 \cdot S_k$ $S_{f6} = 85.208\ kN$

b) Snølast dominerende:

$G_{f7} := 1.2 \cdot G_k$ $G_{f7} = 108.719\ kN$

$N_{f7} := 1.05 \cdot N_k$ $N_{f7} = 47.723\ kN$

$S_{f7} := 1.5 \cdot S_k$ $S_{f7} = 121.725\ kN$

c) Nyttelast dominerende:

$G_{f8} := 1.2 \cdot G_k$ $G_{f8} = 108.719\ kN$

$N_{f8} := 1.5 \cdot N_k$ $N_{f8} = 68.175\ kN$

$S_{f8} := 1.05 \cdot S_k$ $S_{f8} = 85.208\ kN$

4. Egenlast og nyttelast $k_{mod} := 0.8$

a) Egenlast dominerende:

$G_{f9} := 1.35 \cdot G_k$ $G_{f9} = 122.309\ kN$

$N_{f9} := 1.05 \cdot N_k$ $N_{f9} = 47.723\ kN$

b) Snølast dominerende:

$G_{f10} := 1.2 \cdot G_k$ $G_{f10} = 108.719\ kN$

$N_{f10} := 1.5 \cdot N_k$ $N_{f10} = 68.175\ kN$

KNEKKING

y-y: Knekklengde: $l_{ky} := 2150\ mm + 550\ mm$ (søyle + bjelkehøyde)

Treghetsradius: $i_y := 0.29 \cdot h$ $i_y = 65.25\ mm$

Slankhet: $\lambda_y := \frac{l_{ky}}{i_y}$ $\lambda_y = 41.379$

[1] (6.21) $\lambda_{rel.y} := \frac{\lambda_y}{\pi} \cdot \sqrt{\frac{f_{c.0.k}}{E_{0.05}}}$ $\lambda_{rel.y} = 0.627$

[1] (6.29) $\beta_c := 0.1$ (limtre)

[1] (6.27) $k_y := 0.5 \cdot \left(1 + \beta_c \cdot \left(\lambda_{rel.y} - 0.3\right) + \lambda_{rel.y}^{\ 2}\right)$ $k_y = 0.713$

$k_{cy} := \frac{1}{k_y + \sqrt{k_y^{\ 2} - \lambda_{rel.y}^{\ 2}}}$ $k_{cy} = 0.95$

z-z: Knekklengde: $l_{kz} := 2150\ mm + 550\ mm$ (søyle + bjelkehøyde)

Treghetsradius: $i_z := 0.29 \cdot b$ $i_z = 62.35\ mm$

Slankhet: $\lambda_z := \frac{l_{kz}}{i_z}$ $\lambda_z = 43.304$

[1] (6.21) $\lambda_{rel.z} := \frac{\lambda_z}{\pi} \cdot \sqrt{\frac{f_{c.0.k}}{E_{0.05}}}$ $\lambda_{rel.z} = 0.657$

[1] (6.29) $\beta_c := 0.1$ (limtre)

[1] (6.27) $k_z := 0.5 \cdot \left(1 + \beta_c \cdot \left(\lambda_{rel.z} - 0.3\right) + \lambda_{rel.z}^{\ 2}\right)$ $k_z = 0.733$

$k_{cz} := \frac{1}{k_z + \sqrt{k_z^{\ 2} - \lambda_{rel.z}^{\ 2}}}$ $k_{cz} = 0.943$

<u>BRANNDIMENSJONERING</u>

[3] Tab 3.1 Dimensjonerende forkullingshastighet for limtre: $\beta_n := 0.7\ \frac{mm}{min}$

[4] § 11-3 Brannklasse 2

[4] § 11-4 Brannmotstandstid R60: $t := 60\ min$

[2] Tab. NA.A1.1

$\psi_{0.s} := 0.7$ $\psi_{1.s} := 0.5$ $\psi_{2.s} := 0.2$

$\psi_{0.v} := 0.6$ $\psi_{1.v} := 0.2$ $\psi_{2.v} := 0$

$\psi_{0.n} := 0.7$ $\psi_{1.n} := 0.5$ $\psi_{2.n} := 0.3$

$N_{fi} := G_k + N_k \cdot \psi_{2.n} + \psi_{1.s} \cdot S_k + \psi_{2.v} \cdot Q_k$ $N_{fi} = 144.809\ kN$

[3] 4.2.2 **Effektiv forkullingsdybde**

$t > 20\ min$ $\quad k_0 := 1.0$

$d_{char.n} := \beta_n \cdot t$

$d_0 := 7\ mm$

$d_{ef} := d_{char.n} + k_0 \cdot d_0$ $\quad d_{ef} = 49\ mm$

Dimensjon effektivt resttverrsnitt

$b_{ef.fi} := b - 2 \cdot d_{ef}$ $\quad b_{ef.fi} = 117\ mm$

$h_{ef.fi} := h - 2 \cdot d_{ef}$ $\quad h_{ef.fi} = 127\ mm$

Dimensjonerende fasthet

$f_{c.0.d.fi} := 1.15 \cdot f_{c.0.k}$ $\quad f_{c.0.d.fi} = 28.175\ \frac{N}{mm^2}$

$f_{m.d.fi} := 1.15 \cdot f_{m.k}$ $\quad f_{m.d.fi} = 34.5\ \frac{N}{mm^2}$

Knekking - brann

y-y: Treghetsradius: $i_{y.fi} := 0.29 \cdot h_{ef.fi}$ $\quad i_{y.fi} = 36.83\ mm$

Slankhet: $\lambda_{y.fi} := \frac{l_{ky}}{i_{y.fi}}$ $\quad \lambda_{y.fi} = 73.31$

[1] (6.21) $\lambda_{rel.y.fi} := \frac{\lambda_{y.fi}}{\pi} \cdot \sqrt{\frac{f_{c.0.k}}{E_{0.05}}}$ $\quad \lambda_{rel.y.fi} = 1.111$

[1] (6.29) $\beta_c := 0.1$ (limtre)

[1] (6.27) $k_{y.fi} := 0.5 \cdot \left(1 + \beta_c \cdot \left(\lambda_{rel.y.fi} - 0.3\right) + \lambda_{rel.y.fi}^{2}\right)$ $\quad k_{y.fi} = 1.158$

$$k_{cy.fi} := \frac{1}{k_{y.fi} + \sqrt{{k_{y.fi}}^2 - {\lambda_{rel.y.fi}}^2}} \qquad k_{cy.fi} = 0.674$$

z-z: Treghetsradius: $i_{z.fi} := 0.29 \cdot b_{ef.fi}$ $\qquad i_{z.fi} = 33.93\ mm$

Slankhet: $\lambda_{z.fi} := \frac{l_{kz}}{i_{z.fi}}$ $\qquad \lambda_{z.fi} = 79.576$

[1] (6.21) $\lambda_{rel.z.fi} := \frac{\lambda_{z.fi}}{\pi} \cdot \sqrt{\frac{f_{c.0.k}}{E_{0.05}}}$ $\qquad \lambda_{rel.z.fi} = 1.206$

[1] (6.29) $\beta_c := 0.1$ (limtre)

[1] (6.27) $k_{z.fi} := 0.5 \cdot \left(1 + \beta_c \cdot \left(\lambda_{rel.z.fi} - 0.3\right) + {\lambda_{rel.z.fi}}^2\right)$ $\qquad k_{z.fi} = 1.273$

$$k_{cz.fi} := \frac{1}{k_{z.fi} + \sqrt{{k_{z.fi}}^2 - {\lambda_{rel.z.fi}}^2}} \qquad k_{cz.fi} = 0.595$$

[1] 6.1.4 **Aksialspenning**

$A_{fi} := b_{ef.fi} \cdot h_{ef.fi}$

$\sigma_{c.0.d.fi} := \frac{N_{fi}}{A_{fi}}$ $\qquad \sigma_{c.0.d.fi} = 9.746\ \frac{N}{mm^2}$

$\frac{\sigma_{c.0.d.fi}}{k_{cz.fi} \cdot f_{c.0.d.fi}} = 0.581$ < 1.0 OK

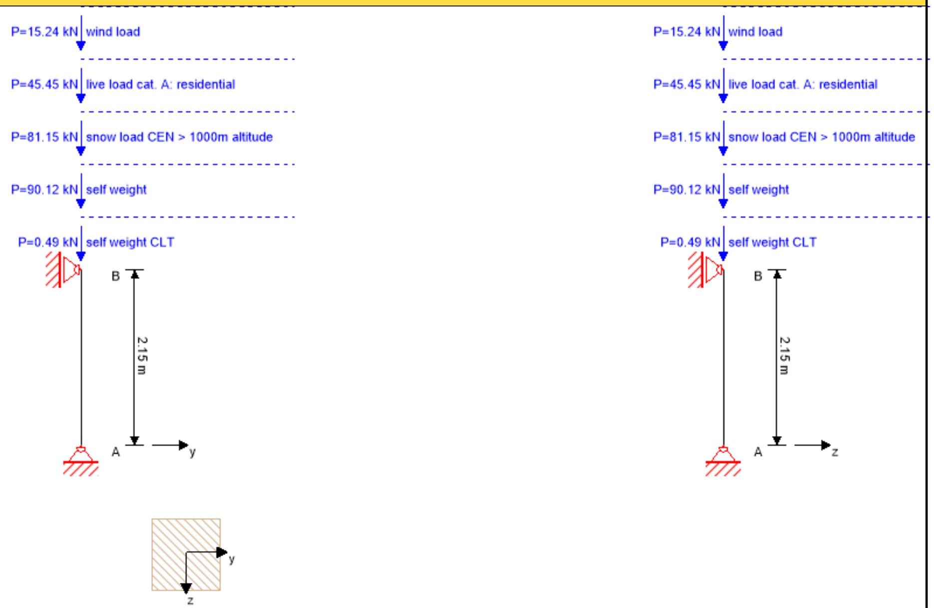
VEDLEGG G8.2

Søyle 2-6				
L	2,15		kcz	0,943
b	215		kcy	0,95
h	225		fcok	24,5
A	48375		Ym	1,15
W	1814062,5		fmk	30

Kombinasjon	1	2a	2b	2c	2d	3a	3b	3c	4a	4b
G	90,60	90,60	90,60	90,60	90,60	90,60	90,60	90,60	90,60	90,60
γG	1,35	1,35	1,20	1,20	1,20	1,35	1,20	1,20	1,35	1,20
S		81,15	81,15	81,15	81,15	81,15	81,15	81,15		
γS		1,05	1,50	1,05	1,05	1,05	1,50	1,05		
Q		15,24	15,24	15,24	15,24					
γQ		1,05	1,05	1,50	1,05					
N		45,45	45,45	45,45	45,45	45,45	45,45	45,45	45,45	45,45
γN		1,05	1,05	1,05	1,50	1,05	1,05	1,50	1,05	1,50
Nf	122,31	271,24	294,17	264,51	278,10	207,52	230,45	193,93	122,31	108,72
σN	2,53	5,61	6,08	5,47	5,75	4,29	4,76	4,01	2,53	2,25
kmod	0,6	1,1	1,1	1,1	1,1	0,9	0,9	0,9	0,8	0,8
fcod	12,78	23,43	23,43	23,43	23,43	19,17	19,17	19,17	17,04	17,04
UF (6.23)	0,21	0,25	0,27	0,25	0,26	0,24	0,26	0,22	0,16	0,16
UF (6.24)	0,21	0,25	**0,28**	0,25	0,26	0,24	0,26	0,22	0,16	0,16

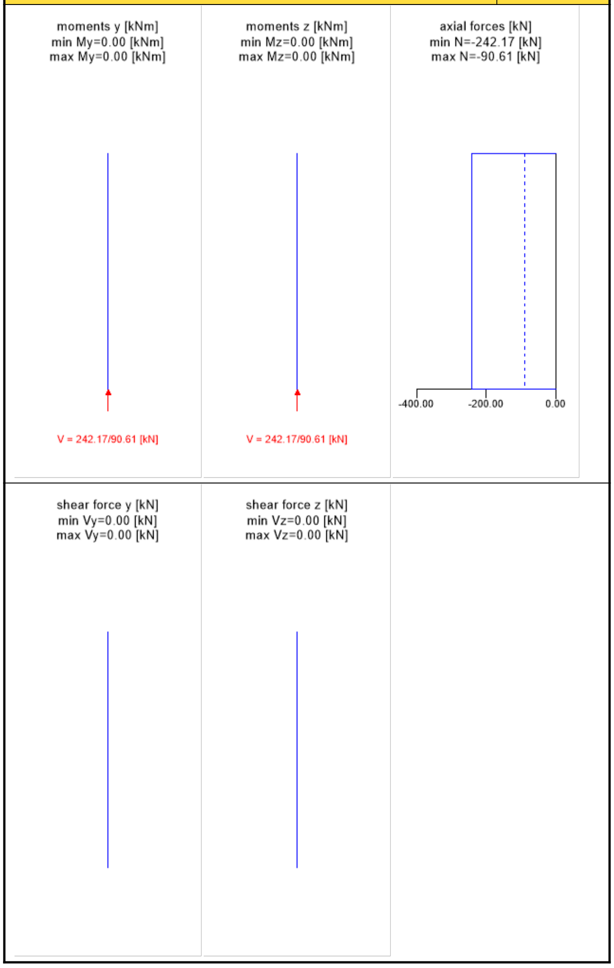
VEDLEGG G8.3

system



section: wooden beam 21.5/22.5; **material**: GL30c; **service class**: service class 1; **fire resistance class**: R 60

utilization 59 %



flexural stress analysis 9 %

$M_{y,d}$ =	0.00	kNm	$f_{m,k}$ =	30.00	N/mm²
$N_{c,d}$ =	-	kN	$f_{c,k}$ =	24.50	N/mm²
	242.17				
$\sigma_{c,d}$ =	5.01	N/mm²	$f_{c,d}$ =	17.04	N/mm²
$\sigma_{m,y,d}$ =	0.00	N/mm² <	$f_{m,y,d}$ =	20.87	N/mm² ✓

shear stress analysis Y 0 %

V_d =	0.00	kN	$f_{v,k}$ =	3.50	N/mm²
$\tau_{v,d}$ =	0.00	N/mm² <	$f_{v,d}$ =	1.84	N/mm² ✓

shear stress analysis Z 0 %

V_d =	0.00	kN	$f_{v,k}$ =	3.50	N/mm²
$\tau_{v,d}$ =	0.00	N/mm² <	$f_{v,d}$ =	1.84	N/mm² ✓

shear stress analysis combined 0 %

$V_{y,d}$ =	0.00	kN	$V_{z,d}$ =	0.00	kN
$\tau_{v,y,d}$ =	0.00	N/mm²	$\tau_{v,z,d}$ =	0.00	N/mm²
			ratio =	0 %	✓

lateral torsional buckling analysis 31 %

$M_{y,d}$ =	0.00	kNm	$f_{m,k}$ =	30.00	N/mm²
$N_{c,d}$ =	-	kN	$f_{c,k}$ =	24.50	N/mm²
	242.17				
$\sigma_{c,d}$ =	5.01	N/mm²	$f_{c,d}$ =	17.04	N/mm²
$\sigma_{m,y,d}$ =	0.00	N/mm² <	$f_{m,y,d}$ =	20.87	N/mm² ✓

buckling analysis 30 %

$M_{y,d}$ =	0.00	kNm	$f_{m,k}$ =	30.00	N/mm²
$N_{c,d}$ =	-	kN	$f_{c,k}$ =	24.50	N/mm²
	242.17				
$\sigma_{c,d}$ =	5.01	N/mm²	$f_{c,d}$ =	17.04	N/mm²
$\sigma_{m,y,d}$ =	0.00	N/mm²	$f_{m,y,d}$ =	20.87	N/mm²
$\sigma_{m,z,d}$ =	0.00	N/mm² <	$f_{m,z,d}$ =	20.87	N/mm² ✓

flexural stress analysis fire 12 %

$M_{y,d}$ =	0.00	kNm	$f_{m,k}$ =	30.00	N/mm²
$N_{c,d}$ =	-	kN	$f_{c,k}$ =	24.50	N/mm²
	144.82				
$\sigma_{c,d}$ =	9.75	N/mm²	$f_{c,d}$ =	28.18	N/mm²
$\sigma_{m,y,d}$ =	0.00	N/mm² <	$f_{m,y,d}$ =	34.50	N/mm² ✓

shear stress analysis Y fire 0 %

V_d =	0.00	kN	$f_{v,k}$ =	3.50	N/mm²
$\tau_{v,d}$ =	0.00	N/mm² <	$f_{v,d}$ =	2.70	N/mm² ✓

shear stress analysis Z fire 0 %

V_d =	0.00	kN	$f_{v,k}$ =	3.50	N/mm²
$\tau_{v,d}$ =	0.00	N/mm² <	$f_{v,d}$ =	2.70	N/mm² ✓

shear stress analysis combined fire 0 %

$V_{y,d}$ =	0.00	kN	$V_{z,d}$ =	0.00	kN
$\tau_{v,y,d}$ =	0.00	N/mm²	$\tau_{v,z,d}$ =	0.00	N/mm²
			ratio =	0 %	✓

lateral torsional buckling analysis fire 59 %

$M_{y,d}$ =	0.00	kNm	$f_{m,k}$ =	30.00	N/mm²
$N_{c,d}$ =	-	kN	$f_{c,k}$ =	24.50	N/mm²
	144.82				
$\sigma_{c,d}$ =	9.75	N/mm²	$f_{c,d}$ =	28.18	N/mm²
$\sigma_{m,y,d}$ =	0.00	N/mm² <	$f_{m,y,d}$ =	34.50	N/mm² ✓

buckling analysis fire 41 %

$M_{y,d}$ =	0.00	kNm	$f_{m,k}$ =	30.00	N/mm²
$N_{c,d}$ =	-	kN	$f_{c,k}$ =	24.50	N/mm²
	144.82				
$\sigma_{c,d}$ =	9.75	N/mm²	$f_{c,d}$ =	28.18	N/mm²
$\sigma_{m,y,d}$ =	0.00	N/mm²	$f_{m,y,d}$ =	34.50	N/mm²
$\sigma_{m,z,d}$ =	0.00	N/mm² <	$f_{m,z,d}$ =	34.50	N/mm² ✓

support reaction

load case category	k_{mod}	A_y	A_z	B_x	B_y	B_z
		[kN]	[kN]	[kN]	[kN]	[kN]
self weight CLT	0.6	0.00	0.00	0.49	0.00	0.00
		0.00	0.00	0.49	0.00	0.00

support reaction						
load case category	k_{mod}	A_y	A_z	B_x	B_y	B_z
		[kN]	[kN]	[kN]	[kN]	[kN]
self weight	0.6	0.00	0.00	90.12	0.00	0.00
		0.00	0.00	90.12	0.00	0.00
snow load CEN > 1000m altitude	0.8	0.00	0.00	0.00	0.00	0.00
		0.00	0.00	81.15	0.00	0.00
live load cat. A: residential	0.8	0.00	0.00	0.00	0.00	0.00
		0.00	0.00	45.45	0.00	0.00
wind load	0.9	0.00	0.00	0.00	0.00	0.00
		0.00	0.00	15.24	0.00	0.00

VEDLEGG G9.1

SØYLE 1-10

Referanse til standarder

[1] NS-EN 1995-1-1: Allmenne regler og regler for bygninger
[2] NS-EN 1990: Grunnlag for prosjektering av konstruksjoner
[3] NS-EN 1995-1-2: Brannteknisk dimensjonering
[4] Byggeteknisk forskrift (TEK17)
[5] Norsk Limtreprodusenters Forening, *Limtreboka - Beregningseksempler* (2018)

FORUTSETNINGER OG ANTAKELSER

[1] NA.901 Klimaklasse: 1

[1] Tab. 2.1 Lastvarighetsklasse for snølast: Korttidslast
Lastvarighetsklasse for vindlasten: Øyeblikkslast

[1] NA.2.3 Partialfaktor for limtre: $\gamma_M := 1.15$

Antar søyletverrrsnitt: $b := 215\ mm$ $h := 315\ mm$

Lastbredde: $Lb := 6640\ mm$

Lengde: $L := 2150\ mm$

[5] Tab. 6 Limtre GL30c:

$f_{m.k} := 30\ \frac{N}{mm^2}$ $f_{c.90.k} := 2.5\ \frac{N}{mm^2}$ $f_{v.k} := 3.5\ \frac{N}{mm^2}$

$f_{c.0.k} := 24.5\ \frac{N}{mm^2}$ $E_{0.05} := 10800\ \frac{N}{mm^2}$ $\rho_m := 430\ \frac{kg}{m^3}$

$g := 9.81\ \frac{m}{s^2}$

LASTER

(Verdier hentet fra Calculatis)

Egenvekt: $G_b := 17.8\ kN + 47.36\ kN \cdot 3$ $G_b = 159.88\ kN$

Egenvekt søyle: $G_s := 4.609\ \frac{kN}{m^3} \cdot b \cdot h \cdot L$ $G_s = 671.111\ N$

Permanent last: $G_k := G_b + G_s$ $G_k = 160.551\ kN$

Snølast: $S_k := 21.3\ kN$

Nyttelast: $N_k := 26.6\ kN \cdot 3$ $N_k = 79.8\ kN$

Vindlast (fra tak): $Q_k := 8.9\ kN$

Vindlast på vegg: $q_{kast} := 1.34\ \frac{kN}{m^2}$ $C_A := 1.2$ $C_D := 0.724$

$q_{vind1} := q_{kast} \cdot (C_A + 0.2) \cdot Lb$ $q_{vind1} = 12.457\ \frac{kN}{m}$

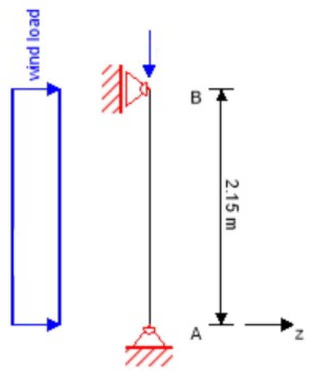
$q_{vind2} := q_{kast} \cdot (C_D + 0.3) \cdot Lb$ $q_{vind2} = 9.111\ \frac{kN}{m}$

$q_k := \max(q_{vind1}, q_{vind2})$ $q_k = 12.457\ \frac{kN}{m}$

LASTKOMBINASJONER

1. Kun egenlast $k_{mod} := 0.6$

$G_{f1} := 1.35 \cdot G_k$ $G_{f1} = 216.744\ kN$



2. Alle laster $k_{mod} := 1.1$

a) Egenlast dominerende:

$G_{f2} := 1.35 \cdot G_k$ $G_{f2} = 216.744\ kN$

$S_{f2} := 1.05 \cdot S_k$ $S_{f2} = 22.365\ kN$

$N_{f2} := 1.05 \cdot N_k$ $N_{f2} = 83.79\ kN$

$Q_{f2} := 1.05 \cdot Q_k$ $Q_{f2} = 9.345\ kN$

$q_{f2} := 1.05 \cdot q_k$ $q_{f2} = 13.079\ \frac{kN}{m}$

b) Snølast dominerende:

$G_{f3} := 1.2 \cdot G_k$ $\quad G_{f3} = 192.661\ kN$

$S_{f3} := 1.5 \cdot S_k$ $\quad S_{f3} = 31.95\ kN$

$N_{f3} := 1.05 \cdot N_k$ $\quad N_{f3} = 83.79\ kN$

$Q_{f3} := 1.05 \cdot Q_k$ $\quad Q_{f3} = 9.345\ kN$

$q_{f3} := 1.05 \cdot q_k$ $\quad q_{f3} = 13.079\ \frac{kN}{m}$

c) Nyttelast dominerende:

$G_{f4} := 1.2 \cdot G_k$ $\quad G_{f4} = 192.661\ kN$

$S_{f4} := 1.05 \cdot S_k$ $\quad S_{f4} = 22.365\ kN$

$N_{f4} := 1.5 \cdot N_k$ $\quad N_{f4} = 119.7\ kN$

$Q_{f4} := 1.05 \cdot Q_k$ $\quad Q_{f4} = 9.345\ kN$

$q_{f4} := 1.05 \cdot q_k$ $\quad q_{f4} = 13.079\ \frac{kN}{m}$

d) Vindlast dominerende:

$G_{f5} := 1.2 \cdot G_k$ $\quad G_{f5} = 192.661\ kN$

$S_{f5} := 1.05 \cdot S_k$ $\quad S_{f5} = 22.365\ kN$

$N_{f5} := 1.05 \cdot N_k$ $\quad N_{f5} = 83.79\ kN$

$Q_{f5} := 1.5 \cdot Q_k$ $\quad Q_{f5} = 13.35\ kN$

$q_{f5} := 1.5 \cdot q_k$ $\quad q_{f5} = 18.685\ \frac{kN}{m}$

3. Egenlast, nyttelast og snølast $k_{mod} := 0.9$

a) Egenlast dominerende:

$G_{f6} := 1.35 \cdot G_k$ $G_{f6} = 216.744\ kN$

$S_{f6} := 1.05 \cdot S_k$ $S_{f6} = 22.365\ kN$

$N_{f6} := 1.05 \cdot N_k$ $N_{f6} = 83.79\ kN$

b) Snølast dominerende:

$G_{f7} := 1.2 \cdot G_k$ $G_{f7} = 192.661\ kN$

$S_{f7} := 1.5 \cdot S_k$ $S_{f7} = 31.95\ kN$

$N_{f7} := 1.05 \cdot N_k$ $N_{f7} = 83.79\ kN$

c) Nyttelast dominerende:

$G_{f8} := 1.2 \cdot G_k$ $G_{f8} = 192.661\ kN$

$S_{f8} := 1.05 \cdot S_k$ $S_{f8} = 22.365\ kN$

$N_{f8} := 1.5 \cdot N_k$ $N_{f8} = 119.7\ kN$

3. Egenlast og nyttelast $k_{mod} := 0.8$

a) Egenlast dominerende:

$G_{f9} := 1.35 \cdot G_k$ $G_{f9} = 216.744\ kN$

$N_{f9} := 1.05 \cdot N_k$ $N_{f9} = 83.79\ kN$

b) Nyttelast dominerende:

$G_{f10} := 1.35 \cdot G_k$ $G_{f10} = 216.744\ kN$

$N_{f10} := 1.5 \cdot N_k$ $N_{f10} = 119.7\ kN$

[1] (3.2) Høydefaktor: $k_h := min\left(\left(\frac{600\ mm}{h}\right)^{0.1}, 1.1\right)$ $k_h = 1.067$

KNEKKING

y-y: Knekklengde: $l_{ky} := 2150\ mm + 550\ mm$ (søyle + bjelkehøyde)

Treghetsradius: $i_y := 0.29 \cdot h$ $i_y = 91.35\ mm$

Slankhet: $\lambda_y := \frac{l_{ky}}{i_y}$ $\lambda_y = 29.557$

[1] (6.21) $\lambda_{rel.y} := \frac{\lambda_y}{\pi} \cdot \sqrt{\frac{f_{c.0.k}}{E_{0.05}}}$ $\lambda_{rel.y} = 0.448$

[1] (6.29) $\beta_c := 0.1$ (limtre)

[1] (6.27) $k_y := 0.5 \cdot \left(1 + \beta_c \cdot \left(\lambda_{rel.y} - 0.3\right) + \lambda_{rel.y}^{2}\right)$ $k_y = 0.608$

$k_{cy} := \frac{1}{k_y + \sqrt{k_y^{2} - \lambda_{rel.y}^{2}}}$ $k_{cy} = 0.982$

z-z: Knekklengde: $l_{kz} := 2150\ mm + 550\ mm$ (søyle + bjelkehøyde)

Treghetsradius: $i_z := 0.29 \cdot b$ $i_z = 62.35\ mm$

Slankhet: $\lambda_z := \frac{l_{kz}}{i_z}$ $\lambda_z = 43.304$

[1] (6.21) $\lambda_{rel.z} := \frac{\lambda_z}{\pi} \cdot \sqrt{\frac{f_{c.0.k}}{E_{0.05}}}$ $\lambda_{rel.z} = 0.657$

[1] (6.29) $\beta_c := 0.1$ (limtre)

[1] (6.27) $k_z := 0.5 \cdot \left(1 + \beta_c \cdot \left(\lambda_{rel.z} - 0.3\right) + \lambda_{rel.z}^{2}\right)$ $k_z = 0.733$

$$k_{cz} := \frac{1}{k_z + \sqrt{{k_z}^2 - {\lambda_{rel.z}}^2}} \qquad k_{cz} = 0.943$$

BRANNDIMENSJONERING

[3] Tab 3.1 Dimensjonerende forkullingshastighet for limtre: $\beta_n := 0.7 \frac{mm}{min}$

[4] § 11-3 Brannklasse 2

[4] § 11-4 Brannmotstandstid R60: $t := 60\ min$

[2] Tab. NA.A1.1

$\psi_{0.s} := 0.7 \qquad \psi_{1.s} := 0.5 \qquad \psi_{2.s} := 0.2$

$\psi_{0.v} := 0.6 \qquad \psi_{1.v} := 0.2 \qquad \psi_{2.v} := 0$

$\psi_{0.n} := 0.7 \qquad \psi_{1.n} := 0.5 \qquad \psi_{2.n} := 0.3$

$N_{fi} := G_k + N_k \cdot \psi_{1.n} + \psi_{2.s} \cdot S_k \qquad N_{fi} = 204.711\ kN$

[3] 4.2.2 **Effektiv forkullingsdybde**

$t > 20\ min \qquad k_0 := 1.0$

$d_{char.n} := \beta_n \cdot t$

$d_0 := 7\ mm$

$d_{ef} := d_{char.n} + k_0 \cdot d_0 \qquad d_{ef} = 49\ mm$

Dimensjon effektivt resttverrsnitt

$b_{ef.fi} := b - 2 \cdot d_{ef} \qquad b_{ef.fi} = 117\ mm$

$h_{ef.fi} := h - 2 \cdot d_{ef} \qquad h_{ef.fi} = 217\ mm$

Dimensjonerende fasthet

$f_{c.0.d.fi} := 1.15 \cdot f_{c.0.k}$ $\qquad f_{c.0.d.fi} = 28.175 \ \frac{N}{mm^2}$

$f_{m.d.fi} := 1.15 \cdot f_{m.k}$ $\qquad f_{m.d.fi} = 34.5 \ \frac{N}{mm^2}$

Knekking - brann

y-y: Treghetsradius: $i_{y.fi} := 0.29 \cdot h_{ef.fi}$ $\qquad i_{y.fi} = 62.93 \ mm$

Slankhet: $\lambda_{y.fi} := \frac{l_{ky}}{i_{y.fi}}$ $\qquad \lambda_{y.fi} = 42.905$

[1] (6.21) $\lambda_{rel.y.fi} := \frac{\lambda_{y.fi}}{\pi} \cdot \sqrt{\frac{f_{c.0.k}}{E_{0.05}}}$ $\qquad \lambda_{rel.y.fi} = 0.65$

[1] (6.29) $\beta_c := 0.1$ (limtre)

[1] (6.27) $k_{y.fi} := 0.5 \cdot \left(1 + \beta_c \cdot \left(\lambda_{rel.y.fi} - 0.3\right) + \lambda_{rel.y.fi}^{\ 2}\right)$ $\qquad k_{y.fi} = 0.729$

$k_{cy.fi} := \frac{1}{k_{y.fi} + \sqrt{k_{y.fi}^{\ 2} - \lambda_{rel.y.fi}^{\ 2}}}$ $\qquad k_{cy.fi} = 0.945$

z-z: Treghetsradius: $i_{z.fi} := 0.29 \cdot b_{ef.fi}$ $\qquad i_{z.fi} = 33.93 \ mm$

Slankhet: $\lambda_{z.fi} := \frac{l_{kz}}{i_{z.fi}}$ $\qquad \lambda_{z.fi} = 79.576$

[1] (6.21) $\lambda_{rel.z.fi} := \frac{\lambda_{z.fi}}{\pi} \cdot \sqrt{\frac{f_{c.0.k}}{E_{0.05}}}$ $\qquad \lambda_{rel.z.fi} = 1.206$

[1] (6.29) $\beta_c := 0.1$ (limtre)

[1] (6.27) $k_{z.fi} := 0.5 \cdot \left(1 + \beta_c \cdot \left(\lambda_{rel.z.fi} - 0.3\right) + \lambda_{rel.z.fi}^{\ 2}\right)$ $\qquad k_{z.fi} = 1.273$

$$k_{cz.fi} := \frac{1}{k_{z.fi} + \sqrt{k_{z.fi}^{2} - \lambda_{rel.z.fi}^{2}}} \qquad k_{cz.fi} = 0.595$$

[1] 6.1.6 **Bøyespenning**

$$W := \frac{1}{6} \cdot b_{ef.fi} \cdot h_{ef.fi}^{2}$$

$$M_{y.fi} := \frac{q_k \cdot \psi_{1.v} \cdot L^{2}}{8} \qquad M_{y.fi} = 1.44 \; kN \cdot m$$

$$\sigma_{m.y.fi} := \frac{M_{y.fi}}{W} \qquad \sigma_{m.y.fi} = 1.568 \; \frac{N}{mm^{2}}$$

$$\frac{\sigma_{m.y.fi}}{f_{m.d.fi}} = 0.045$$ < 1.0 OK

[1] 6.1.4 **Aksialspenning**

$$A_{fi} := b_{ef.fi} \cdot h_{ef.fi}$$

$$\sigma_{c.0.d.fi} := \frac{N_{fi}}{A_{fi}} \qquad \sigma_{c.0.d.fi} = 8.063 \; \frac{N}{mm^{2}}$$

$$\frac{\sigma_{c.0.d.fi}}{k_{cz.fi} \cdot f_{c.0.d.fi}} = 0.481$$ < 1.0 OK

Kombinert virkning

[1] 6.1.6 (2) $k_m := 0.7$

[2] (6.23) $\frac{\sigma_{c.0.d.fi}}{k_{cy.fi} \cdot f_{c.0.d.fi}} + \frac{\sigma_{m.y.fi}}{f_{m.d.fi}} = 0.348$ < 1.0 OK

[2] (6.24) $\frac{\sigma_{c.0.d.fi}}{k_{cz.fi} \cdot f_{c.0.d.fi}} + k_m \cdot \frac{\sigma_{m.y.fi}}{f_{m.d.fi}} = 0.512$ < 1.0 OK

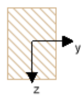
VEDLEGG G9.2

Søyle 1-10							
L	2,15		kcz	0,943		kh	1,1
b	215		kcy	0,982		km	0,7
h	315		fcok	24,5			
A	67725		Ym	1,15			
W	3555562,5		fmk	30			

Kombinasjon	1	2a	2b	2c	2d	3a	3b	3c	4a	4b
G	160,46	160,46	160,46	160,46	160,46	160,46	160,46	160,46	160,46	160,46
γG	1,35	1,35	1,20	1,20	1,20	1,35	1,20	1,20	1,20	1,35
S		21,30	21,30	21,30	21,30	21,30	21,30	21,30		
γS		1,05	1,50	1,05	1,05	1,05	1,50	1,05		
N		79,80	79,80	79,80	79,80	79,80	79,80	79,80	79,80	79,80
γN		1,05	1,05	1,50	1,05	1,05	1,05	1,50	1,50	1,05
Q		8,90	8,90	8,90	8,90					
γQ		1,05	1,05	1,05	1,50					
q		12,46	12,46	12,46	12,46					
γq		1,05	1,05	1,05	1,50					
Nf	216,62	261,41	233,85	224,26	312,06	238,99	224,50	214,92	312,25	300,41
qf	0,00	13,08	13,08	13,08	18,69	0,00	0,00	0,00	0,00	0,00
Mf	0,00	7,56	7,56	7,56	10,80	0,00	0,00	0,00	0,00	0,00
σN	3,20	3,86	3,45	3,31	4,61	3,53	3,31	3,17	4,61	4,44
σM	0,00	2,13	2,13	2,13	3,04	0,00	0,00	0,00	0,00	0,00
kmod	0,6	1,1	1,1	1,1	1,1	0,9	0,9	0,9	0,8	0,8
fcod	12,78	23,43	23,43	23,43	23,43	19,17	19,17	19,17	17,04	17,04
fmyd	17,22	31,57	31,57	31,57	31,57	25,83	21,09	21,09	18,75	18,75
UF (6.23)	0,25	0,24	0,22	0,21	**0,30**	0,19	0,18	0,20	0,28	0,27
UF (6.24)	0,27	0,22	0,20	0,20	0,28	0,20	0,17	0,15	0,26	0,25

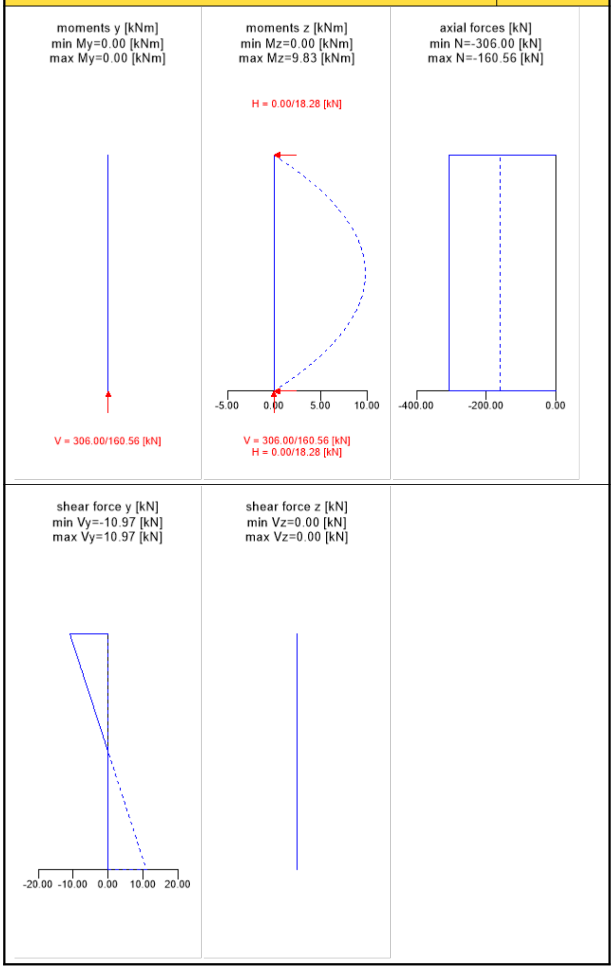
VEDLEGG G9.3

system



section: wooden beam 21.5/31.5; **material**: GL30c; **service class**: service class 1; **fire resistance class**: R 60

utilization 48 %



flexural stress analysis 23 %

$M_{y,d}$ =	0.00	kNm	$f_{m,k}$ =	30.00	N/mm²
$N_{c,d}$ =	- 306.00	kN	$f_{c,k}$ =	24.50	N/mm²
$\sigma_{c,d}$ =	4.52	N/mm²	$f_{c,d}$ =	19.17	N/mm²
$\sigma_{m,y,d}$ =	0.00	N/mm²<	$f_{m,y,d}$ =	23.48	N/mm² ✓

shear stress analysis Y 22 %

V_d =	18.28	kN	$f_{v,k}$ =	3.50	N/mm²
$\tau_{v,d}$ =	0.40	N/mm²<	$f_{v,d}$ =	1.84	N/mm² ✓

shear stress analysis Z 0 %

V_d =	0.00	kN	$f_{v,k}$ =	3.50	N/mm²
$\tau_{v,d}$ =	0.00	N/mm²<	$f_{v,d}$ =	1.84	N/mm² ✓

shear stress analysis combined 5 %

$V_{y,d}$ =	18.28	kN	$V_{z,d}$ =	0.00	kN
$\tau_{v,y,d}$ =	0.40	N/mm²	$\tau_{v,z,d}$ =	0.00	N/mm²
			ratio =	5 %	✓

lateral torsional buckling analysis 28 %

$M_{y,d}$ =	0.00	kNm	$f_{m,k}$ =	30.00	N/mm²
$N_{c,d}$ =	- 306.18	kN	$f_{c,k}$ =	24.50	N/mm²
$\sigma_{c,d}$ =	4.52	N/mm²	$f_{c,d}$ =	17.04	N/mm²
$\sigma_{m,y,d}$ =	0.00	N/mm²<	$f_{m,y,d}$ =	20.87	N/mm² ✓

buckling analysis 41 %

$M_{y,d}$ =	0.00	kNm	$f_{m,k}$ =	30.00	N/mm²
$N_{c,d}$ =	- 306.00	kN	$f_{c,k}$ =	24.50	N/mm²
$\sigma_{c,d}$ =	4.52	N/mm²	$f_{c,d}$ =	19.17	N/mm²
$\sigma_{m,y,d}$ =	0.00	N/mm²	$f_{m,y,d}$ =	23.48	N/mm²
$\sigma_{m,z,d}$ =	4.05	N/mm²<	$f_{m,z,d}$ =	23.48	N/mm² ✓

flexural stress analysis fire 16 %

$M_{y,d}$ =	0.00	kNm	$f_{m,k}$ =	30.00	N/mm²
$N_{c,d}$ =	- 190.54	kN	$f_{c,k}$ =	24.50	N/mm²
$\sigma_{c,d}$ =	7.50	N/mm²	$f_{c,d}$ =	28.18	N/mm²
$\sigma_{m,y,d}$ =	0.00	N/mm²<	$f_{m,y,d}$ =	34.50	N/mm² ✓

shear stress analysis Y fire 6 %

V_d =	2.68	kN	$f_{v,k}$ =	3.50	N/mm²
$\tau_{v,d}$ =	0.16	N/mm²<	$f_{v,d}$ =	2.70	N/mm² ✓

shear stress analysis Z fire 0 %

V_d =	0.00	kN	$f_{v,k}$ =	3.50	N/mm²
$\tau_{v,d}$ =	0.00	N/mm²<	$f_{v,d}$ =	2.70	N/mm² ✓

shear stress analysis combined fire 0 %

$V_{y,d}$ =	2.68	kN	$V_{z,d}$ =	0.00	kN
$\tau_{v,y,d}$ =	0.16	N/mm²	$\tau_{v,z,d}$ =	0.00	N/mm²
			ratio =	0 %	✓

lateral torsional buckling analysis fire 48 %

$M_{y,d}$ =	0.00	kNm	$f_{m,k}$ =	30.00	N/mm²
$N_{c,d}$ =	- 204.72	kN	$f_{c,k}$ =	24.50	N/mm²
$\sigma_{c,d}$ =	8.06	N/mm²	$f_{c,d}$ =	28.18	N/mm²
$\sigma_{m,y,d}$ =	0.00	N/mm²<	$f_{m,y,d}$ =	34.50	N/mm² ✓

buckling analysis fire 36 %

$M_{y,d}$ =	0.00	kNm	$f_{m,k}$ =	30.00	N/mm²
$N_{c,d}$ =	- 190.54	kN	$f_{c,k}$ =	24.50	N/mm²
$\sigma_{c,d}$ =	7.50	N/mm²	$f_{c,d}$ =	28.18	N/mm²
$\sigma_{m,y,d}$ =	0.00	N/mm²	$f_{m,y,d}$ =	34.50	N/mm²
$\sigma_{m,z,d}$ =	2.91	N/mm²<	$f_{m,z,d}$ =	34.50	N/mm² ✓

support reaction

load case category	k_{mod}	A_y	A_z	B_x	B_y	B_z
		[kN]	[kN]	[kN]	[kN]	[kN]
self weight CLT	0.6	0.00	0.00	0.68	0.00	0.00
		0.00	0.00	0.68	0.00	0.00

support reaction						
load case category	k_{mod}	A_y	A_z	B_x	B_y	B_z
		[kN]	[kN]	[kN]	[kN]	[kN]
self weight	0.6	0.00	0.00	159.88	0.00	0.00
		0.00	0.00	159.88	0.00	0.00
live load cat. A: residential	0.8	0.00	0.00	0.00	0.00	0.00
		0.00	0.00	79.80	0.00	0.00
wind load	0.9	13.39	0.00	0.00	13.39	0.00
		0.00	0.00	8.90	0.00	0.00
snow load CEN > 1000m altitude	0.8	0.00	0.00	0.00	0.00	0.00
		0.00	0.00	21.30	0.00	0.00

Disclaimer

VEDLEGG G10.1

SØYLE 1-9

Referanser

[1] NS-EN 1995-1-1: Allmenne regler og regler for bygninger
[2] NS-EN 1990: Grunnlag for prosjektering av konstruksjoner
[3] NS-EN 1992-1-2: Brannteknisk dimensjonering
[4] Byggeteknisk forskrift (TEK17)
[5] Norsk Limtreprodusenters Forening, *Limtreboka - Beregningseksempler* (2018)

FORUTSETNINGER OG ANTAKELSER

[1] NA.901 Klimaklasse: 1

[1] Tab. 2.1 Lastvarighetsklasse for snølast: Korttidslast
Lastvarighetsklasse for vindlasten: Øyeblikkslast

[1] NA.2.3 Partialfaktor for limtre: $\gamma_M := 1.15$

Antar søyletverrrsnitt: $b := 215\ mm$ $h := 315\ mm$

Lengde: $L := 2150\ mm$

[5] Tab. 6 Limtre GL30c:

$f_{m.k} := 30\ \frac{N}{mm^2}$ $f_{c.90} := 2.5\ \frac{N}{mm^2}$ $f_{v.k} := 3.5\ \frac{N}{mm^2}$

$f_{c.0.k} := 24.5\ \frac{N}{mm^2}$ $E_{0.05} := 10800\ \frac{N}{mm^2}$ $\rho_m := 430\ \frac{kg}{m^3}$

$g := 9.81\ \frac{m}{s^2}$

LASTER

(Verdier hentet fra Calculatis)

Egenvekt: $G_b := 184.2\ kN + 75.41\ kN$ $G_b = 259.61\ kN$

Egenvekt søyle: $G_s := 4.609\ \frac{kN}{m^3} \cdot b \cdot h \cdot L$ $G_s = 671.111\ N$

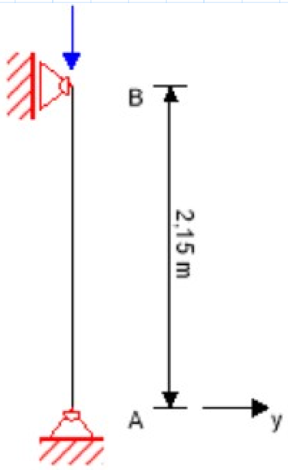
Permanent last: $G_k := G_b + G_s$ $G_k = 260.281\ kN$

Snølast: $S_k := 44.25\ kN$

Vindlast (fra tak): $Q_k := 18.3\ kN$

Nyttelast: $N_k := 54.66\ kN \cdot 3$ $N_k = 163.98\ kN$

LASTKOMBINASJONER



1. Kun egenlast $k_{mod} := 0.6$

$G_{f1} := 1.35 \cdot G_k$ $G_{f1} = 351.379\ kN$

2. Alle laster $k_{mod} := 1.1$

a) Egenlast dominerende:

$G_{f2} := 1.35 \cdot G_k$ $G_{f2} = 351.379\ kN$

$S_{f2} := 1.05 \cdot S_k$ $S_{f2} = 46.463\ kN$

$Q_{f2} := 1.05 \cdot Q_k$ $Q_{f2} = 19.215\ kN$

$N_{f2} := 1.05 \cdot N_k$ $N_{f2} = 172.179\ kN$

b) Snølast dominerende:

$G_{f3} := 1.2 \cdot G_k$ $G_{f3} = 312.337\ kN$

$S_{f3} := 1.5 \cdot S_k$ $S_{f3} = 66.375\ kN$

$Q_{f3} := 1.05 \cdot Q_k$ $Q_{f3} = 19.215\ kN$

$N_{f3} := 1.05 \cdot N_k$ $N_{f3} = 172.179\ kN$

c) Vindlast dominerende:

$G_{f4} := 1.2 \cdot G_k$ $G_{f4} = 312.337\ kN$

$S_{f4} := 1.05 \cdot S_k$ $S_{f4} = 46.463\ kN$

$Q_{f4} := 1.5 \cdot Q_k$ $Q_{f4} = 27.45\ kN$

$N_{f4} := 1.05 \cdot N_k$ $N_{f4} = 172.179\ kN$

d) Nyttelast dominerende:

$G_{f5} := 1.2 \cdot G_k$ $G_{f5} = 312.337\ kN$

$S_{f5} := 1.05 \cdot S_k$ $S_{f5} = 46.463\ kN$

$Q_{f5} := 1.05 \cdot Q_k$ $Q_{f5} = 19.215\ kN$

$N_{f5} := 1.5 \cdot N_k$ $N_{f5} = 245.97\ kN$

3. Egenlast, nyttelast og snølast $k_{mod} := 0.9$

a) Egenlast dominerende:

$G_{f6} := 1.35 \cdot G_k$ $G_{f6} = 351.379\ kN$

$S_{f6} := 1.05 \cdot S_k$ $S_{f6} = 46.463\ kN$

$N_{f6} := 1.05 \cdot N_k$ $N_{f6} = 172.179\ kN$

b) Snølast dominerende:

$G_{f7} := 1.2 \cdot G_k$ $G_{f7} = 312.337\ kN$

$S_{f7} := 1.5 \cdot S_k$ $S_{f7} = 66.375\ kN$

$N_{f7} := 1.05 \cdot N_k$ $N_{f7} = 172.179\ kN$

c) Nyttelast dominerende:

$G_{f8} := 1.2 \cdot G_k$ $G_{f8} = 312.337\ kN$

$S_{f8} := 1.05 \cdot S_k$ $S_{f8} = 46.463\ kN$

$N_{f8} := 1.5 \cdot N_k$ $N_{f8} = 245.97\ kN$

4. Egenlast og nyttelast $k_{mod} := 0.8$

a) Egenlast dominerende:

$G_{f9} := 1.35 \cdot G_k$ $G_{f9} = 351.379\ kN$

$N_{f9} := 1.05 \cdot N_k$ $N_{f9} = 172.179\ kN$

b) Nyttelast dominerende:

$G_{f10} := 1.2 \cdot G_k$ $G_{f10} = 312.337\ kN$

$N_{f10} := 1.5 \cdot N_k$ $N_{f10} = 245.97\ kN$

<u>KNEKKING</u>

y-y: Knekklengde: $l_{ky} := 2150\ mm + 550\ mm$ (Søyle + bjelkehøyde)

Treghetsradius: $i_y := 0.29 \cdot h$ $i_y = 91.35\ mm$

Slankhet: $\lambda_y := \dfrac{l_{ky}}{i_y}$ $\lambda_y = 29.557$

[1] (6.21) $\lambda_{rel.y} := \dfrac{\lambda_y}{\pi} \cdot \sqrt{\dfrac{f_{c.0.k}}{E_{0.05}}}$ $\lambda_{rel.y} = 0.448$

[1] (6.29) $\beta_c := 0.1$ (limtre)

[1] (6.27) $k_y := 0.5 \cdot \left(1 + \beta_c \cdot \left(\lambda_{rel.y} - 0.3\right) + \lambda_{rel.y}^{\ 2}\right)$ $k_y = 0.608$

$k_{cy} := \dfrac{1}{k_y + \sqrt{k_y^{\ 2} - \lambda_{rel.y}^{\ 2}}}$ $k_{cy} = 0.982$

z-z: Knekklengde: $l_{kz} := 2150\ mm + 550\ mm$ (Søyle + bjelkehøyde)

Treghetsradius: $i_z := 0.29 \cdot b$ $i_z = 62.35\ mm$

Slankhet: $\lambda_z := \dfrac{l_{kz}}{i_z}$ $\lambda_z = 43.304$

[1] (6.21) $\lambda_{rel.z} := \dfrac{\lambda_z}{\pi} \cdot \sqrt{\dfrac{f_{c.0.k}}{E_{0.05}}}$ $\lambda_{rel.z} = 0.657$

[1] (6.29) $\beta_c := 0.1$ (limtre)

[1] (6.27) $k_z := 0.5 \cdot \left(1 + \beta_c \cdot \left(\lambda_{rel.z} - 0.3\right) + \lambda_{rel.z}^{\ 2}\right)$ $k_z = 0.733$

$k_{cz} := \dfrac{1}{k_z + \sqrt{k_z^{\ 2} - \lambda_{rel.z}^{\ 2}}}$ $k_{cz} = 0.943$

<u>BRANNDIMENSJONERING</u>

[3] Tab 3.1 Dimensjonerende forkullingshastighet for limtre: $\beta_n := 0.7\ \dfrac{mm}{min}$

[4] § 11-3 Brannklasse 2

[4] § 11-4 Brannmotstandstid R60: $t := 60\ min$

[2] Tab. NA.A1.1

$\psi_{0.s} := 0.7$ $\psi_{1.s} := 0.5$ $\psi_{2.s} := 0.2$

$\psi_{0.v} := 0.6$ $\psi_{1.v} := 0.2$ $\psi_{2.v} := 0$

$\psi_{0.n} := 0.7$ $\psi_{1.n} := 0.5$ $\psi_{2.n} := 0.3$

$N_{fi} := G_k + \psi_{1.n} \cdot N_k + \psi_{2.s} \cdot S_k + \psi_{2.v} \cdot Q_k$ $N_{fi} = 351.121\ kN$

[3] 4.2.2 **Effektiv forkullingsdybde**

$t > 20\ min$ $k_0 := 1.0$

$d_{char.n} := \beta_n \cdot t$

$d_0 := 7\ mm$

$d_{ef} := d_{char.n} + k_0 \cdot d_0$ $d_{ef} = 49\ mm$

Dimensjon effektivt resttverrsnitt

$b_{ef.fi} := b - 2 \cdot d_{ef}$ $b_{ef.fi} = 117\ mm$

$h_{ef.fi} := h - 2 \cdot d_{ef}$ $h_{ef.fi} = 217\ mm$

Dimensjonerende fasthet

$f_{c.0.d.fi} := 1.15 \cdot f_{c.0.k}$ $f_{c.0.d.fi} = 28.175\ \frac{N}{mm^2}$

$f_{m.d.fi} := 1.15 \cdot f_{m.k}$ $f_{m.d.fi} = 34.5\ \frac{N}{mm^2}$

Knekking - brann

y-y: Treghetsradius: $i_{y.fi} := 0.29 \cdot h_{ef.fi}$ $i_{y.fi} = 62.93\ mm$

Slankhet: $\lambda_{y.fi} := \frac{l_{ky}}{i_{y.fi}}$ $\lambda_{y.fi} = 42.905$

[1] (6.21) $\lambda_{rel.y.fi} := \frac{\lambda_{y.fi}}{\pi} \cdot \sqrt{\frac{f_{c.0.k}}{E_{0.05}}}$ $\lambda_{rel.y.fi} = 0.65$

[1] (6.29) $\beta_c := 0.1$ (limtre)

[1] (6.27) $k_{y.fi} := 0.5 \cdot \left(1 + \beta_c \cdot (\lambda_{rel.y.fi} - 0.3) + \lambda_{rel.y.fi}^{\ 2}\right)$ $k_{y.fi} = 0.729$

$$k_{cy.fi} := \frac{1}{k_{y.fi} + \sqrt{{k_{y.fi}}^2 - {\lambda_{rel.y.fi}}^2}}$$ $k_{cy.fi} = 0.945$

z-z: Treghetsradius: $i_{z.fi} := 0.29 \cdot b_{ef.fi}$ $i_{z.fi} = 33.93\ mm$

Slankhet: $\lambda_{z.fi} := \frac{l_{kz}}{i_{z.fi}}$ $\lambda_{z.fi} = 79.576$

[1] (6.21) $\lambda_{rel.z.fi} := \frac{\lambda_{z.fi}}{\pi} \cdot \sqrt{\frac{f_{c.0.k}}{E_{0.05}}}$ $\lambda_{rel.z.fi} = 1.206$

[1] (6.29) $\beta_c := 0.1$ (limtre)

[1] (6.27) $k_{z.fi} := 0.5 \cdot \left(1 + \beta_c \cdot \left(\lambda_{rel.z.fi} - 0.3\right) + {\lambda_{rel.z.fi}}^2\right)$ $k_{z.fi} = 1.273$

$$k_{cz.fi} := \frac{1}{k_{z.fi} + \sqrt{{k_{z.fi}}^2 - {\lambda_{rel.z.fi}}^2}}$$ $k_{cz.fi} = 0.595$

[1] 6.1.4 **Aksialspenning**

$A_{fi} := b_{ef.fi} \cdot h_{ef.fi}$

$\sigma_{c.0.d.fi} := \frac{N_{fi}}{A_{fi}}$ $\sigma_{c.0.d.fi} = 13.83\ \frac{N}{mm^2}$

$\frac{\sigma_{c.0.d.fi}}{k_{cz.fi} \cdot f_{c.0.d.fi}} = 0.824$ < 1.0 OK

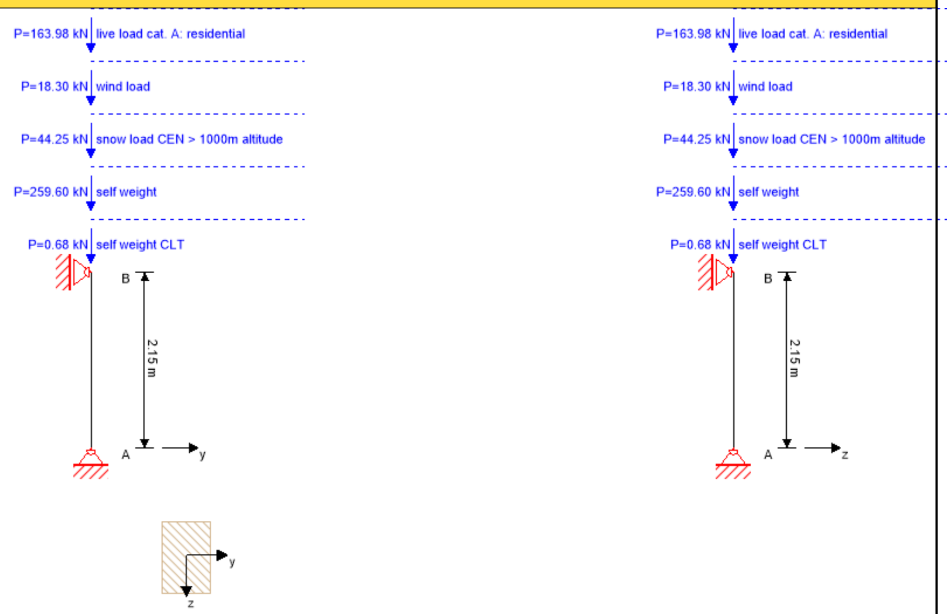
VEDLEGG G10.2

Søyle 1-9				
L	2,15		kcz	0,971
b	215		kcy	0,943
h	315		fcok	24,5
A	67725		Ym	1,15
W	3555562,5		fmk	30

Kombinasjon	1	2a	2b	2c	2d	3a	3b	3c	4a	4b
G	260,19	260,19	260,19	260,19	260,19	260,19	260,19	260,19	260,19	260,19
γG	1,35	1,35	1,20	1,20	1,20	1,35	1,20	1,20	1,35	1,20
S		44,25	44,25	44,25	44,25	44,25	44,25	44,25		
γS		1,05	1,50	1,05	1,05	1,05	1,50	1,05		
Q		18,30	18,30	18,30	18,30					
γQ		1,05	1,05	1,50	1,05					
N		163,98	163,98	163,98	163,98	163,98	163,98	163,98	163,98	163,98
γN		1,05	1,05	1,05	1,50	1,05	1,05	1,50	1,05	1,50
Nf	351,26	589,11	570,00	558,32	623,88	397,72	378,60	358,69	351,26	312,23
σN	5,19	8,70	8,42	8,24	9,21	5,87	5,59	5,30	5,19	4,61
kmod	0,6	1,1	1,1	1,1	1,1	0,9	0,9	0,9	0,8	0,8
fcod	12,78	23,43	23,43	23,43	23,43	19,17	19,17	19,17	17,04	17,04
UF (6.23)	**0,43**	0,39	0,38	0,37	0,42	0,32	0,31	0,29	0,32	0,32
UF (6.24)	0,42	0,38	0,37	0,36	0,40	0,32	0,30	0,28	0,31	0,31

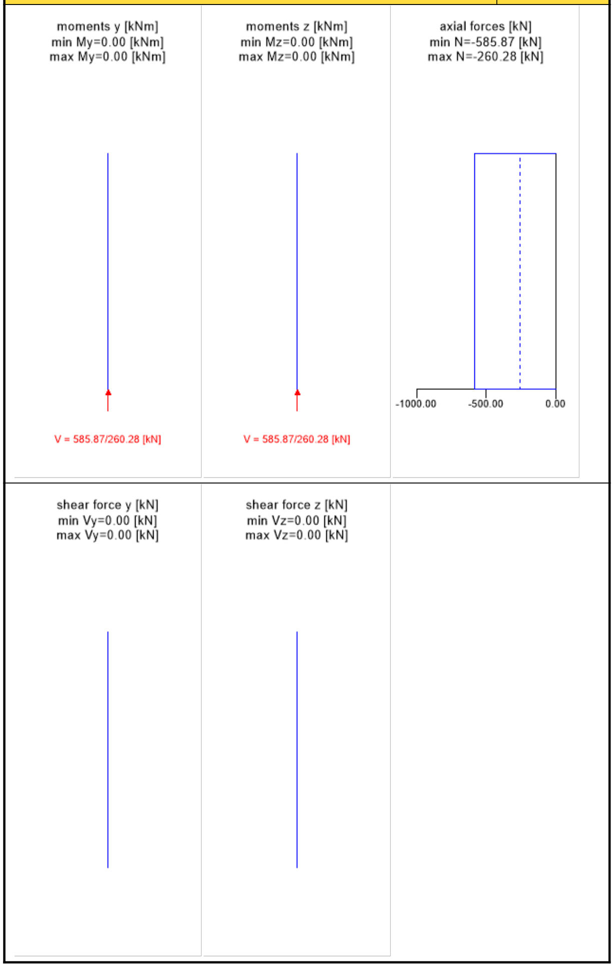
VEDLEGG G10.3

system



section: wooden beam 21.5/31.5; **material**: GL30c; **service class**: service class 1; **fire resistance class**: R 60

utilization — 83 %



flexural stress analysis — 26 %

$M_{y,d}$ =	0.00	kNm	$f_{m,k}$ =	30.00	N/mm²	
$N_{c,d}$ =	-	kN	$f_{c,k}$ =	24.50	N/mm²	
	585.87					
$\sigma_{c,d}$ =	8.65	N/mm²	$f_{c,d}$ =	17.04	N/mm²	
$\sigma_{m,y,d}$ =	0.00	N/mm² <	$f_{m,y,d}$ =	20.87	N/mm²	✓

shear stress analysis Y — 0 %

V_d =	0.00	kN	$f_{v,k}$ =	3.50	N/mm²	
$\tau_{v,d}$ =	0.00	N/mm² <	$f_{v,d}$ =	1.84	N/mm²	✓

shear stress analysis Z — 0 %

V_d =	0.00	kN	$f_{v,k}$ =	3.50	N/mm²	
$\tau_{v,d}$ =	0.00	N/mm² <	$f_{v,d}$ =	1.84	N/mm²	✓

shear stress analysis combined — 0 %

$V_{y,d}$ =	0.00	kN	$V_{z,d}$ =	0.00	kN	
$\tau_{v,y,d}$ =	0.00	N/mm²	$\tau_{v,z,d}$ =	0.00	N/mm²	
			ratio =	0 %		✓

lateral torsional buckling analysis — 54 %

$M_{y,d}$ =	0.00	kNm	$f_{m,k}$ =	30.00	N/mm²	
$N_{c,d}$ =	-	kN	$f_{c,k}$ =	24.50	N/mm²	
	585.87					
$\sigma_{c,d}$ =	8.65	N/mm²	$f_{c,d}$ =	17.04	N/mm²	
$\sigma_{m,y,d}$ =	0.00	N/mm² <	$f_{m,y,d}$ =	20.87	N/mm²	✓

buckling analysis — 51 %

$M_{y,d}$ =	0.00	kNm	$f_{m,k}$ =	30.00	N/mm²	
$N_{c,d}$ =	-	kN	$f_{c,k}$ =	24.50	N/mm²	
	585.87					
$\sigma_{c,d}$ =	8.65	N/mm²	$f_{c,d}$ =	17.04	N/mm²	
$\sigma_{m,y,d}$ =	0.00	N/mm²	$f_{m,y,d}$ =	20.87	N/mm²	
$\sigma_{m,z,d}$ =	0.00	N/mm² <	$f_{m,z,d}$ =	20.87	N/mm²	✓

flexural stress analysis fire — 24 %

$M_{y,d}$ =	0.00	kNm	$f_{m,k}$ =	30.00	N/mm²	
$N_{c,d}$ =	-	kN	$f_{c,k}$ =	24.50	N/mm²	
	351.12					
$\sigma_{c,d}$ =	13.83	N/mm²	$f_{c,d}$ =	28.18	N/mm²	
$\sigma_{m,y,d}$ =	0.00	N/mm² <	$f_{m,y,d}$ =	34.50	N/mm²	✓

shear stress analysis Y fire — 0 %

V_d =	0.00	kN	$f_{v,k}$ =	3.50	N/mm²	
$\tau_{v,d}$ =	0.00	N/mm² <	$f_{v,d}$ =	2.70	N/mm²	✓

shear stress analysis Z fire — 0 %

V_d =	0.00	kN	$f_{v,k}$ =	3.50	N/mm²	
$\tau_{v,d}$ =	0.00	N/mm² <	$f_{v,d}$ =	2.70	N/mm²	✓

shear stress analysis combined fire — 0 %

$V_{y,d}$ =	0.00	kN	$V_{z,d}$ =	0.00	kN	
$\tau_{v,y,d}$ =	0.00	N/mm²	$\tau_{v,z,d}$ =	0.00	N/mm²	
			ratio =	0 %		✓

lateral torsional buckling analysis fire — 83 %

$M_{y,d}$ =	0.00	kNm	$f_{m,k}$ =	30.00	N/mm²	
$N_{c,d}$ =	-	kN	$f_{c,k}$ =	24.50	N/mm²	
	351.12					
$\sigma_{c,d}$ =	13.83	N/mm²	$f_{c,d}$ =	28.18	N/mm²	
$\sigma_{m,y,d}$ =	0.00	N/mm² <	$f_{m,y,d}$ =	34.50	N/mm²	✓

buckling analysis fire — 51 %

$M_{y,d}$ =	0.00	kNm	$f_{m,k}$ =	30.00	N/mm²	
$N_{c,d}$ =	-	kN	$f_{c,k}$ =	24.50	N/mm²	
	351.12					
$\sigma_{c,d}$ =	13.83	N/mm²	$f_{c,d}$ =	28.18	N/mm²	
$\sigma_{m,y,d}$ =	0.00	N/mm²	$f_{m,y,d}$ =	34.50	N/mm²	
$\sigma_{m,z,d}$ =	0.00	N/mm² <	$f_{m,z,d}$ =	34.50	N/mm²	✓

support reaction

load case category	k_{mod}	A_y	A_z	B_x	B_y	B_z
		[kN]	[kN]	[kN]	[kN]	[kN]
self weight CLT	0.6	0.00	0.00	0.68	0.00	0.00
		0.00	0.00	0.68	0.00	0.00

support reaction						
load case category	k_{mod}	A_y	A_z	B_x	B_y	B_z
		[kN]	[kN]	[kN]	[kN]	[kN]
self weight	0.6	0.00	0.00	259.60	0.00	0.00
		0.00	0.00	259.60	0.00	0.00
snow load CEN > 1000m altitude	0.8	0.00	0.00	0.00	0.00	0.00
		0.00	0.00	44.25	0.00	0.00
wind load	0.9	0.00	0.00	0.00	0.00	0.00
		0.00	0.00	18.30	0.00	0.00
live load cat. A: residential	0.8	0.00	0.00	0.00	0.00	0.00
		0.00	0.00	163.98	0.00	0.00

Disclaimer

VEDLEGG H

Dekker

H. Dekker

- H1. Tak
  - *H1.1 Mathcad*
  - *H1.2 Calculatis*
- H2. Etasjeskiller
  - *H2.1 Mathcad*
  - *H2.2 Calculatis*
- H3. Takterrasse
  - *H3.1 Mathcad*
  - *H3.2 Calculatis*

VEDLEGG H1.1

TAK I MASSIVTRE

Referanser til standarer

[1] NS-EN 1995-1-1: Allmenne regler og regler for bygninger
[2] NS-EN 1991-1-4: Laster på konstruksjoner
[3] NS-EN 338: Konstruksjonstrevirke - Fasthetsklasser

Prosjekteringsgrunnlag:

- 5-sjiktsmassivtre element
- 3 langsgående sjikt og 2 tverrgående sjikt
- Total tykkelse på elementet er 180mm

Bruker Schubanalogieverfahren metoden for å regne på massivtre. Dette er en mer nøyaktig modell, da den tar hensyn til sjiktenes ulike elastisitesmoduler.

Tak med alle sjikt av kvalitet C24

FORUTSETNINGER OG ANTAKELSER

$d_1 := 40\ mm$ $d_3 := d_1 = 40\ mm$ $d_5 := d_1 = 40\ mm$

$d_2 := 30\ mm$ $d_4 := d_2 = 30\ mm$

$h := d_1 + d_2 + d_3 + d_4 + d_5 = 180\ mm$

$b := 1000\ mm$ (Ser på 1 m bredde av elemetet)

$L := 6\ m$

Avstand til elementet sitt tyngdepunkt fra hvert enkelt massivtreelement:

$z_1 := \frac{h}{2} - \frac{d_1}{2}$ $z_1 = 70\ mm$ $z_5 := z_1 = 70\ mm$

$z_2 := \frac{h}{2} - d_1 - \frac{d_2}{2}$ $z_2 = 35\ mm$ $z_4 := z_2 = 35\ mm$

$z_3 := 0\ mm$

[1] NA.2.3 Partialfaktor: $\gamma_M := 1.25$

[1] Tab. 3.1 $k_{mod} := 0.8$ (Forenkler, dette er tss)

[3] $E_0 := 11000 \cdot \frac{N}{mm^2}$ $E_{90} := \frac{E_0}{30}$

Bøyefasthet: $f_{mk} := 24 \frac{N}{mm^2}$

LASTER

Egenvekt: $q_g := 1.025 \frac{kN}{m^2} \cdot b$

Snølast: $s_k := 1.6 \frac{kN}{m^2} \cdot b$

Vindlast: $q_{kast} := 1.34 \frac{kN}{m^2} \cdot b$

Sug på tak: $C_H := -0.7$

$q_H := q_{kast} \cdot (C_H - 0.2)$ $q_H = -1.206 \frac{kN}{m}$

Trykk på tak: $C_I := 0.2$

$q_I := q_{kast} \cdot (C_I + 0.3)$ $q_I = 0.67 \frac{kN}{m}$

Dimensjonerende verdier med kombinasjonsfaktor:

$q_1 := 1.0 \cdot q_g + 1.5 \cdot q_H$ $q_1 = -0.784 \frac{kN}{m}$ (sug)

$q_{2.ef} := 1.2 \cdot q_g + 1.5 \cdot s_k + 1.05 \cdot q_I$ $q_{2.ef} = 4.334 \frac{kN}{m}$ (trykk)

Opptredende moment: $M_{Ed} := \frac{q_{2.ef} \cdot L^2}{8}$ $M_{Ed} = 19.501 \; kN \cdot m$

BØYESTIVHET

Bøyestivhet til bjelke A

$EI_A := \frac{b}{12} \cdot \left(3 \cdot \left(E_0 \cdot {d_1}^3\right) + 2 \left(E_{90} \cdot {d_2}^3\right)\right)$

$EI_A = \left(1.777 \cdot 10^{11}\right) N \cdot mm^2$

Bøyestivhet til bjelke B

$$EI_B := b \cdot \left(2 \cdot \left(E_0 \cdot d_1 \cdot {z_1}^2\right) + 2 \cdot \left(E_{90} \cdot d_2 \cdot {z_2}^2\right)\right)$$

$$EI_B = \left(4.339 \cdot 10^{12}\right)\ N \cdot mm^2$$

Total bøyestivhet

$$EI_{ef} := EI_A + EI_B \qquad EI_{ef} = \left(4.517 \cdot 10^{12}\right)\ N \cdot mm^2$$

Effektiv E-modul

$$E_{element} := \frac{EI_{ef}}{\frac{1}{12} \cdot b \cdot h^3} \qquad E_{element} = \left(9.293 \cdot 10^3\right)\ \frac{N}{mm^2}$$

MOMENTFORDELING

Momentfordeling mellom bjelke A og B

$$M_A := \frac{M_{Ed}}{EI_{ef}} \cdot EI_A \qquad M_A = 0.767\ kN \cdot m$$

$$M_B := \frac{M_{Ed}}{EI_{ef}} \cdot EI_B \qquad M_B = 18.734\ kN \cdot m$$

Momentfordeling til hvert sjikt

$$I_1 := \frac{1}{12} \cdot b \cdot {d_1}^3 \qquad I_2 := \frac{1}{12} \cdot b \cdot {d_2}^3$$

$$M_{A1} := \frac{E_0 \cdot I_1}{EI_A} \cdot M_A \qquad M_{A1} = 0.253\ kN \cdot m$$

$$M_{A2} := \frac{E_{90} \cdot I_2}{EI_A} \cdot M_A \qquad M_{A2} = 0.004\ kN \cdot m$$

$$M_{A3} := M_{A1} = 0.253\ kN \cdot m$$

$$M_{A4} := M_{A2} = 0.004\ kN \cdot m$$

$M_{A5} := M_{A1} = 0.253\ kN \cdot m$

NORMALKRAFTFORDELING TIL HVERT SJIKT

$N_{B1} := \frac{E_0 \cdot (b \cdot d_1) \cdot z_1}{EI_B} \cdot M_B$ $\quad N_{B1} = 132.981\ kN$

$N_{B2} := \frac{E_{90} \cdot (b \cdot d_2) \cdot z_2}{EI_B} \cdot M_B$ $\quad N_{B2} = 1.662\ kN$

$N_{B3} := \frac{E_0 \cdot (b \cdot d_3) \cdot z_3}{EI_B} \cdot M_B$ $\quad N_{B3} = 0\ kN$

$N_{B4} := N_{B2} = 1.662\ kN$

$N_{B5} := N_{B1} = 132.981\ kN$

BØYESPENNING TIL HVERT SJIKT UT FRA MOMENTFORDELING FRA BJELKE A

$\sigma_{M1} := \frac{M_{A1}}{I_1} \cdot \left(\frac{d_1}{2}\right)$ $\quad \sigma_{M1} = 0.95\ \frac{N}{mm^2}$

$\sigma_{M2} := \frac{M_{A2}}{I_2} \cdot \left(\frac{d_2}{2}\right)$ $\quad \sigma_{M2} = 0.024\ \frac{N}{mm^2}$

$\sigma_{M3} := \frac{M_{A3}}{I_1} \cdot \left(\frac{d_3}{2}\right)$ $\quad \sigma_{M3} = 0.95\ \frac{N}{mm^2}$

$\sigma_{M4} := \frac{M_{A4}}{I_2} \cdot \left(\frac{d_4}{2}\right)$ $\quad \sigma_{M4} = 0.024\ \frac{N}{mm^2}$

$\sigma_{M5} := \frac{M_{A5}}{I_1} \cdot \left(\frac{d_5}{2}\right)$ $\quad \sigma_{M5} = 0.95\ \frac{N}{mm^2}$

NORMALSPENNING TIL HVERT SJIKT UT FRA NORMALFORDELING FRA BJELKE B

$\sigma_{N1} := \frac{N_{B1}}{b \cdot d_1}$ $\sigma_{N1} = 3.325 \frac{N}{mm^2}$

$\sigma_{N2} := \frac{N_{B2}}{b \cdot d_2}$ $\sigma_{N2} = 0.055 \frac{N}{mm^2}$

$\sigma_{N3} := \frac{N_{B3}}{b \cdot d_3}$ $\sigma_{N3} = 0 \ Pa$

$\sigma_{N4} := \frac{N_{B4}}{b \cdot d_4}$ $\sigma_{N4} = 0.055 \frac{N}{mm^2}$

$\sigma_{N5} := \frac{N_{B5}}{b \cdot d_5}$ $\sigma_{N5} = 3.325 \frac{N}{mm^2}$

SUM SPENNINGER I OVERGANG MELLOM HVERT SJIKT

Sjikt 1: $\sigma_{N1} = 3.325 \frac{N}{mm^2}$

$\sigma_{N1} + \sigma_{M1} = 4.274 \frac{N}{mm^2}$

$\sigma_{N1} - \sigma_{M1} = 2.375 \frac{N}{mm^2}$

Sjikt 2: $\sigma_{N2} = 0.055 \frac{N}{mm^2}$

$\sigma_{N2} + \sigma_{M2} = 0.079 \frac{N}{mm^2}$

$\sigma_{N2} - \sigma_{M2} = 0.032 \frac{N}{mm^2}$

Sjikt 3: $\sigma_{N3} = 0 \frac{N}{mm^2}$

$\sigma_{N3} + \sigma_{M3} = 0.95 \frac{N}{mm^2}$

$\sigma_{N3} - \sigma_{M3} = -0.95 \frac{N}{mm^2}$

Sjikt 4: $\sigma_{N4} = 0.055\ \frac{N}{mm^2}$

$\sigma_{N4} + \sigma_{M4} = 0.079\ \frac{N}{mm^2}$

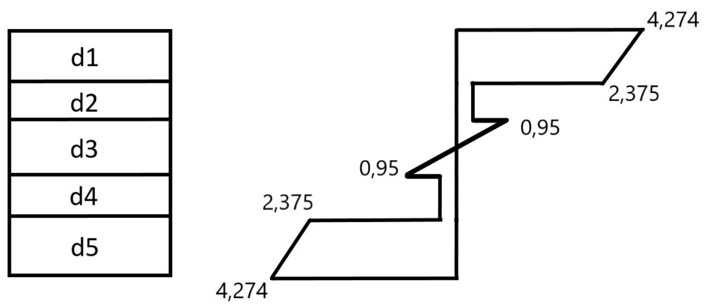
$\sigma_{N4} - \sigma_{M4} = 0.032\ \frac{N}{mm^2}$

Sjikt 5: $\sigma_{N5} = 3.325\ \frac{N}{mm^2}$

$\sigma_{N5} + \sigma_{M5} = 4.274\ \frac{N}{mm^2}$

$\sigma_{N5} - \sigma_{M5} = 2.375\ \frac{N}{mm^2}$

Spenningsfordeling i kNm over tverrsnittet til et 5-sjikts element med alle sjikt av kvalitet C24 pga. et ytre bøyemoment på 19.501 kNm



KONTROLL

$f_{md} := \frac{f_{mk} \cdot k_{mod}}{\gamma_M} = 15.36\ \frac{N}{mm^2}$ > $\sigma_{N5} + \sigma_{M5} = 4.274\ \frac{N}{mm^2}$ OK

Utnyttelse

$\frac{\sigma_{N5} + \sigma_{M5}}{f_{md}} = 0.278$ > 1 OK

BRUKSGRENSEKONTROLL

Nedbøyning

$k_{def} := 0.6$ (Klimaklasse 1)

[1] Tab. NA.A1.1

$\psi_{0.s} := 0.7$ $\psi_{1.s} := 0.5$ $\psi_{2.s} := 0.2$

$\psi_{0.v} := 0.6$ $\psi_{1.v} := 0.2$ $\psi_{2.v} := 0$

Øyeblikkelig nedbøyning

$\delta_g := \frac{5}{384} \cdot \frac{q_g \cdot L^4}{EI_{ef}}$ $\delta_g = 3.83\ mm$

$\delta_s := \frac{5}{384} \cdot \frac{s_k \cdot L^4}{EI_{ef}}$ $\delta_s = 5.978\ mm$

$\delta_v := \frac{5}{384} \cdot \frac{q_I \cdot L^4}{EI_{ef}}$ $\delta_v = 2.503\ mm$

$w_{inst} := \delta_g + \delta_s + \delta_v$ $w_{inst} = 12.311\ mm$ < $\frac{L}{400} = 15\ mm$

Langtidsnedbøyning

$\delta_{gfin} := \delta_g \cdot (1 + k_{def})$ $\delta_{gfin} = 6.127\ mm$

$\delta_{qfin} := \delta_s \cdot (1 + \psi_{2.s} \cdot k_{def})$ $\delta_{qfin} = 6.695\ mm$

$\delta_{qfin} := \delta_v \cdot (1 + \psi_{2.v} \cdot k_{def})$ $\delta_{qfin} = 2.503\ mm$

$w_{fin} := \delta_{gfin} + \delta_{qfin}$ $w_{fin} = 8.631\ mm$ < $\frac{L}{250} = 24\ mm$

BRANN DIMENSJONERING TAK
Sjiktdata, avstand fra tyngdepunkt og total bøyestivhet er lik fra bruddgrense

Referanser til standarder
[1] NS-EN 1995-1-2: Brannteknisk dimensjonering
[2] NS-EN 1990: Grunnlag for prosjektering av konstruksjoner
[3] Byggeteknisk forskrift (TEK17)
[4] NS-EN 1995-1-1: Allmenne regler og regler for bygninger

Tak med alle sjikt av kvalitet C24 FORUTSETNINGER OG ANTAKELSER

[1] Tab 3.1 Dimensjonerende forkullingshastighet for gran: $\beta_n := 0.7 \frac{mm}{min}$

[3] § 11-3 Brannklasse 2 (Brann en side)

[3] § 11-4 Brannmotstandstid R60: $t := 60\ min$

[2] NA.A1.1 Nyttelast, kategori A: $\psi_1 := 0.5$ $\psi_2 := 0.2$

Dim. last: $q_{brann} := q_g + \psi_1 \cdot s_k + \psi_2 \cdot q_I$

$q_{brann} = 1.959 \frac{kN}{m}$

[1] 4.2.2 EFFETIV FORKULLINGSDYBDE

$k_0 := 1$ for t > 20 min

$d_{char.n} := \beta_n \cdot t$ $d_0 := 7\ mm$

$d_{ef} := d_{char.n} + k_0 \cdot d_0$ $d_{ef} = 49\ mm$

DIMENSJONERENDE MOMENT- OG SKJÆRBELASTING PÅ TAK

$M_{Ed.fi} := \frac{q_{brann} \cdot L^2}{8}$ $M_{Ed.fi} = 8.816\ kN \cdot m$

$V_{Ed.fi} := \frac{q_{brann} \cdot L}{2}$ $V_{Ed.fi} = 5.877\ kN$

DIMENSJONERENDE FASTHETER

[4] Tab. 3.1 $k_{mod} := 0.8$

NS-EN 338 $f_m := 24 \frac{N}{mm^2}$ $f_v := 4 \frac{N}{mm^2}$ $f_{c0k} := 21 \frac{N}{mm^2}$ $f_{t0k} := 14 \frac{N}{mm^2}$

[4] Tab NA.2.3 Partialfaktor for bestandighet: $\gamma_{fi} := 1$

$f_{m.d} := f_m \cdot \frac{k_{mod}}{\gamma_{fi}}$ $f_{m.d} = 19.2 \frac{N}{mm^2}$

$f_{v.Rd} := f_v \frac{k_{mod}}{\gamma_{fi}}$ $f_{v.Rd} = 3.2 \frac{N}{mm^2}$

RESTTVERSNITT ETTER BRANN

$h_{ef.fi} := h - d_{ef}$ $h_{ef.fi} = 131\ mm$

Sjikt d5

$d_{5.fi} := d_5 - d_{ef}$ $d_{5.fi} = -9\ mm$

Hele sjikt 5 forsvinner i brannen, i tillegg til 9 mm videre inn i sjikt 4.

Sjikt d4

$d_{4.fi} := d_4 + d_{5.fi}$ $d_{4.fi} = 21\ mm$

NØYTRALAKSE ETTER BRANN

Finner den nye nøytralaksen etter brann ved å vekte breddene med E-modulen.

Topplag d1

$b_{d1} := b = \left(1 \cdot 10^3\right)\ mm$

Tverrlag d2

$b_{d2} := \frac{b \cdot E_{90}}{E_0}$ $b_{d2} = 33.333\ mm$

Midtlag d3

$b_{d3} := \frac{b \cdot E_0}{E_0}$ $b_{d3} = \left(1 \cdot 10^3\right)\ mm$

Tverrlag d4

$b_{d4} := \frac{b \cdot E_{90}}{E_0}$ $b_{d4} = 33.333\ mm$

Nøytralakse

$$y0 := \sum_{i=1}^{n} \frac{(A_i \cdot y_i)}{A_i}$$

$$y_0 := \frac{d_{4.fi} \cdot b_{d4} \cdot \left(\frac{d_{4.fi}}{2}\right) + d_3 \cdot b_{d3} \cdot \left(d_{4.fi} + \frac{d_3}{2}\right) + d_2 \cdot b_{d2} \cdot \left(d_{4.fi} + d_3 + \frac{d_2}{2}\right) + d_1 \cdot b_{d1} \cdot \left(d_{4.fi} + d_3 + d_2 + \frac{d_1}{2}\right)}{d_{4.fi} \cdot b_{d4} + d_3 \cdot b_{d3} + d_2 \cdot b_{d2} + d_1 \cdot b_{d1}}$$

$y_0 = 75.439\ mm$

$y := h_{ef.fi} - y_0$ $\quad$ $y = 55.561\ mm$

Treghetsmoment, I

$$I := \sum_{i=1}^{n} \frac{1}{12} \cdot b \cdot d^3 + A \cdot d^2$$

$$I_{eff.1} := \left(\frac{1}{12} \cdot b_{d1} \cdot d_1^{\ 3} + \left((b_{d1} \cdot d_1) \cdot \left(y - \frac{d_1}{2}\right)^2\right)\right) + \left(\frac{1}{12} \cdot b_{d2} \cdot d_2^{\ 3} + \left((b_{d2} \cdot d_2) \cdot \left(y - d_1 - \frac{d_2}{2}\right)^2\right)\right)$$

$$I_{eff.2} := \left(\frac{1}{12} \cdot b_{d3} \cdot d_3^{\ 3} + \left((b_{d3} \cdot d_3) \cdot \left(y_0 - d_{4.fi} - \frac{d_3}{2}\right)^2\right)\right) + \left(\frac{1}{12} \cdot b_{d4} \cdot d_{4.fi}^{\ 3} + \left((b_{d4} \cdot d_{4.fi}) \cdot \left(y_0 - \frac{d_{4.fi}}{2}\right)^2\right)\right)$$

$I_{eff} := I_{eff.1} + I_{eff.2}$ $\quad$ $I_{eff} = (1.117 \cdot 10^8)\ mm^4$

Første arealmoment, S

Restarealet til d2 ovenfor y0: $\quad$ $d_{2.rest} := y - d_1$ $\quad$ $d_{2.rest} = 15.561\ mm$

$$S := \sum_{i=1}^{n} A_i \cdot y_i$$

$$S_{eff} := d_1 \cdot b_{d1} \cdot \left(y_0 - \frac{d_1}{2}\right) + d_{2.rest} \cdot b_{d2} \cdot \left(y_0 - d_1 - \frac{d_{2.rest}}{2}\right)$$

$S_{eff} = (2.232 \cdot 10^6)\ mm^3$

Dimensjonerende moment og skjærkapasitet

$$M_{Rd.fi} := \frac{f_{m.d} \cdot I_{eff}}{y} \qquad V_{Rd.fi} := \frac{f_{v.Rd} \cdot I_{eff} \cdot b}{S_{eff}}$$

$M_{Rd.fi} = 38.615\ kN \cdot m$ $\quad$ $V_{Rd.fi} = 160.215\ kN$

Utnyttelse

$$\frac{M_{Ed.fi}}{M_{Rd.fi}} = 0.228 \quad < 1.0 \quad \text{OK}$$

$$\frac{V_{Ed.fi}}{V_{Rd.fi}} = 0.037 < 1.0 \quad \text{OK}$$

Opptredende trykk- og strekkraft på mest påkjente sjikt

$$F_{Ed.trykk} := \frac{M_{Ed.fi}}{y - \frac{d_1}{2}}$$

$$F_{Ed.trykk} = 247.897 \ kN$$

$$F_{Ed.strekk} := \frac{M_{Ed.fi}}{y_0 - \left(d_{4.fi} + \frac{d_3}{2}\right)}$$

$$F_{Ed.strekk} = 255.976 \ kN$$

Dimensjonerende trykk- og strekkapasitet til mest påkjente sjikt

$$F_{Rd.trykk} := \left(\frac{f_{c0k} \cdot 0.8}{1}\right) \cdot b \cdot d_1 \qquad F_{Rd.trykk} = 672 \ kN$$

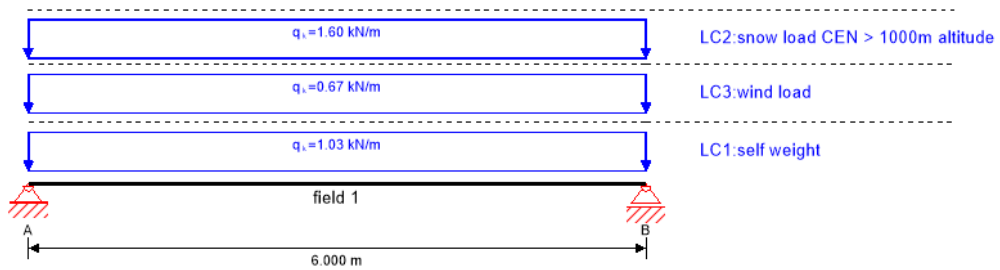
$$F_{Rd.strekk} := \left(\frac{f_{t0k} \cdot 0.8}{1}\right) \cdot b \cdot d_5 \qquad F_{Rd.strekk} = 448 \ kN$$

Utnyttelse

$$\frac{F_{Ed.trykk}}{F_{Rd.trykk}} = 0.369 \quad < 1.0 \quad \text{OK}$$

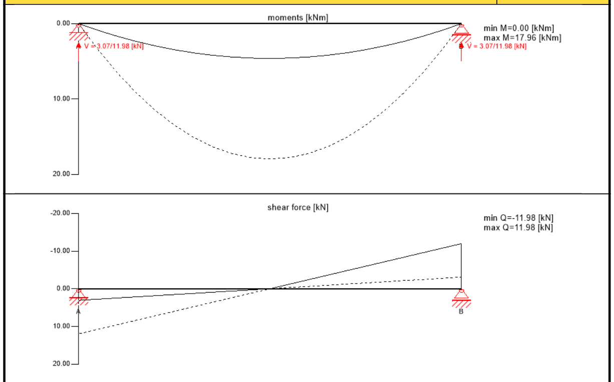
$$\frac{F_{Ed.strekk}}{F_{Rd.strekk}} = 0.571 \quad < 1.0 \quad \text{OK}$$

system



section: CLT 180 L5s; **material**: C24 spruce ETA (2019); **service class**: service class 1; **fire resistance class**: R 60

utilization 78 %



flexural stress analysis 21 %

$M_{y,d}$ =	17.96	kNm	$f_{m,k}$ =	24.00	N/mm²
$N_{t,d}$ =	0.00	kN	$f_{t,k}$ =	0.00	N/mm²
$\sigma_{t,d}$ =	0.00	N/mm²	$f_{t,d}$ =	10.08	N/mm²
$\sigma_{m,y,d}$ =	-3.96	N/mm² <	$f_{m,y,d}$ =	19.01	N/mm² ✓

shear stress analysis 3 %

V_d =	- 11.98	kN	$f_{v,k}$ =	4.00	N/mm²
$\tau_{v,d}$ =	0.09	N/mm² <	$f_{v,d}$ =	2.88	N/mm² ✓

rolling shear analysis 10 %

V_d =	-11.98	kN	$f_{r,k}$ =	1.15	N/mm²
$\tau_{r,d}$ =	0.08	N/mm² <	$f_{r,d}$ =	0.83	N/mm² ✓

flexural stress analysis fire 14 %

$M_{y,d}$ =	8.21	kNm	$f_{m,k}$ =	24.00	N/mm²
$N_{t,d}$ =	0.00	kN	$f_{t,k}$ =	0.00	N/mm²
$\sigma_{t,d}$ =	0.00	N/mm²	$f_{t,d}$ =	16.10	N/mm²
$\sigma_{m,y,d}$ =	-4.16	N/mm² <	$f_{m,y,d}$ =	30.36	N/mm² ✓

shear stress analysis fire 2 %

V_d =	-5.48	kN	$f_{v,k}$ =	4.00	N/mm²
$\tau_{v,d}$ =	0.07	N/mm² <	$f_{v,d}$ =	4.60	N/mm² ✓

rolling shear analysis fire 5 %

V_d =	-5.48	kN	$f_{r,k}$ =	1.15	N/mm²
$\tau_{r,d}$ =	0.07	N/mm² <	$f_{r,d}$ =	1.32	N/mm² ✓

w_{inst} = w[char]

field	k_{def}	limit	w_{limit}	$w_{calc.}$	ratio
		[-]	[mm]	[mm]	
1	0.8	L/400	15.0	11.4	76 %

w_{fin} = w[char] + w[q.p.]*kdef

field	k_{def}	limit	w_{limit}	$w_{calc.}$	ratio
		[-]	[mm]	[mm]	
1	0.8	L/300	20.0	15.5	78 %

$w_{net,fin}$ = w[q.p.] + w[q.p.]*kdef

field	k_{def}	limit	w_{limit}	$w_{calc.}$	ratio
		[-]	[mm]	[mm]	
1	0.8	L/350 L/350	17.1	9.1	53 %

support reaction

load case category	k_{mod}	A_V	B_V
		[kN]	
self weight	0.6	3.07	3.07
		3.07	3.07
snow load CEN > 1000m altitude	0.8	4.80	4.80
		0.00	0.00
wind load	0.9	2.01	2.01
		0.00	0.00

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VEDLEGG H2.1

ETASJESKILLER I MASSIVTRE

Referanser til standarder

[1] NS-EN 1995-1-1: Allmenne regler og regler for bygninger
[2] NS-EN 1990: Grunnlag for prosjektering av konstruksjoner
[3] NS-EN 338: Konstruksjonstrevirke - Fasthetsklasser

Prosjekteringsgrunnlag:

- 7-sjiktsmassivtre element
- 5 langsgående sjikt og 3 tverrgående sjikt
- Total tykkelse på elementet er 240 mm

Bruker Schubanalogieverfahren metoden for å regne på massivtre. Dette er en mer nøyaktig modell, da den tar hensyn til sjiktenes ulike elastisitesmoduler.

<u>Etasjeskiller med alle sjikt av kvalitet C24</u>

<u>FORUTSETNINGER OG ANTAKELSER</u>

$d_1 := 30\ mm$ $\quad d_3 := d_1 = 30\ mm$ $\quad d_5 := d_1 = 30\ mm$ $\quad d_7 := d_1 = 30\ mm$

$d_2 := 40\ mm$ $\quad d_4 := d_2 = 40\ mm$ $\quad d_6 := d_2 = 40\ mm$

$h := d_1 + d_2 + d_3 + d_4 + d_5 + d_6 + d_7 = 240\ mm$

$b := 1000\ mm$ (Ser på 1 m bredde av elemetet)

$L := 6\ m$

Avstand til elementet sitt tyngdepunkt fra hvert enkelt massivtreelement:

$z_1 := \frac{h}{2} - \frac{d_1}{2}$ $\quad z_2 := \frac{h}{2} - d_1 - \frac{d_2}{2}$ $\quad z_3 := \frac{h}{2} - d_1 - d_2 - \frac{d_3}{2}$ $\quad z_4 := 0\ mm$ $\quad z_6 := z_2 = 70\ mm$

$z_1 = 105\ mm$ $\quad z_2 = 70\ mm$ $\quad z_3 = 35\ mm$ $\quad z_5 := z_3 = 35\ mm$ $\quad z_7 := z_1 = 105\ mm$

[1] NA.2.3 Partialfaktor: $\gamma_M := 1.25$

[1] Tab. 3.1 $k_{mod} := 0.8$

[3] E-modul til sjiktene: $E_0 := 11000 \cdot \frac{N}{mm^2}$ $\quad E_{90} := \frac{E_0}{30}$

Bøyefasthet: $f_{mk.C24} := 24\ \frac{N}{mm^2}$

<u>LASTER</u>

Egenvekt: $g_k := 2.595\ \frac{kN}{m^2} \cdot b$ $\quad g_{ed} := 1.2 \cdot g_k$

Nyttelast: $q_k := 2.0\ \frac{kN}{m^2} \cdot b$ $\quad q_{ed} := 1.5 \cdot q_k$

Opptredende moment: $M_{Ed} := \frac{(g_{ed} + q_{ed}) \cdot L^2}{8}$ $\quad M_{Ed} = 27.513\ kN \cdot m$

BØYESTIVHET

Bøyestivhet til bjelke A

$$EI_A := \frac{b}{12} \cdot \left(4 \cdot \left(E_0 \cdot d_1^{\,3}\right) + 3\ \left(E_{90} \cdot d_2^{\,3}\right)\right)$$

$$EI_A = \left(1.049 \cdot 10^{11}\right)\ N \cdot mm^2$$

Bøyestivhet til bjelke B

$$EI_B := b \cdot \left(2 \cdot \left(E_0 \cdot d_1 \cdot z_1^{\,2}\right) + 2 \cdot \left(E_0 \cdot d_3 \cdot z_3^{\,2}\right) + 2 \cdot \left(E_{90} \cdot d_2 \cdot z_2^{\,2}\right)\right)$$

$$EI_B = \left(8.229 \cdot 10^{12}\right)\ N \cdot mm^2$$

Total bøyestivhet

$$EI_{ef} := EI_A + EI_B \qquad EI_{ef} = \left(8.334 \cdot 10^{12}\right)\ N \cdot mm^2$$

Effektiv E-modul

$$E_{element} := \frac{EI_{ef}}{\frac{1}{12} \cdot b \cdot h^3} \qquad E_{element} = \left(7.234 \cdot 10^3\right)\ \frac{N}{mm^2}$$

MOMENTFORDELING

Momentfordeling mellom bjelke A og B

$$M_A := \frac{M_{Ed}}{EI_{ef}} \cdot EI_A \qquad M_A = 0.346\ kN \cdot m$$

$$M_B := \frac{M_{Ed}}{EI_{ef}} \cdot EI_B \qquad M_B = 27.167\ kN \cdot m$$

Momentfordeling til hvert sjikt

$$I_1 := \frac{1}{12} \cdot b \cdot d_1^{\,3} \qquad I_2 := \frac{1}{12} \cdot b \cdot d_2^{\,3}$$

$$M_{A1} := \frac{E_0 \cdot I_1}{EI_A} \cdot M_A \qquad M_{A1} = 0.082\ kN \cdot m$$

$$M_{A2} := \frac{E_{90} \cdot I_2}{EI_A} \cdot M_A \qquad M_{A2} = 0.006\ kN \cdot m$$

$M_{A3} := M_{A1} = 0.082\ kN \cdot m$ $\qquad M_{A4} := M_{A2} = 0.006\ kN \cdot m$

$M_{A5} := M_{A1} = 0.082\ kN \cdot m$ $\qquad M_{A6} := M_{A2} = 0.006\ kN \cdot m$ $\qquad M_{A7} := M_{A1} = 0.082\ kN \cdot m$

NORMALKRAFTFORDELING TIL HVERT SJIKT

$$N_{B1} := \frac{E_0 \cdot (b \cdot d_1) \cdot z_1}{EI_B} \cdot M_B \qquad N_{B1} = 114.395\ kN$$

$$N_{B2} := \frac{E_{90} \cdot (b \cdot d_2) \cdot z_2}{EI_B} \cdot M_B \qquad N_{B2} = 3.389\ kN$$

$$N_{B3} := \frac{E_0 \cdot (b \cdot d_3) \cdot z_3}{EI_B} \cdot M_B \qquad N_{B3} = 38.132\ kN$$

$$N_{B4} := \frac{E_{90} \cdot (b \cdot d_4) \cdot z_4}{EI_B} \cdot M_B \qquad N_{B4} = 0\ N$$

$N_{B5} := N_{B3} = 38.132\ kN$ $\qquad N_{B6} := N_{B2} = 3.389\ kN$ $\qquad N_{B7} := N_{B1} = 114.395\ kN$

BØYESPENNING TIL HVERT SJIKT UT FRA MOMENTFORDELING FRA BJELKE A

$$\sigma_{M1} := \frac{M_{A1}}{I_1} \cdot \left(\frac{d_1}{2}\right) \qquad \sigma_{M1} = 0.545 \ \frac{N}{mm^2}$$

$$\sigma_{M2} := \frac{M_{A2}}{I_2} \cdot \left(\frac{d_2}{2}\right) \qquad \sigma_{M2} = 0.024 \ \frac{N}{mm^2}$$

$$\sigma_{M3} := \frac{M_{A3}}{I_1} \cdot \left(\frac{d_3}{2}\right) \qquad \sigma_{M3} = 0.545 \ \frac{N}{mm^2}$$

$$\sigma_{M4} := \frac{M_{A4}}{I_2} \cdot \left(\frac{d_4}{2}\right) \qquad \sigma_{M4} = 0.024 \ \frac{N}{mm^2}$$

$$\sigma_{M5} := \frac{M_{A5}}{I_1} \cdot \left(\frac{d_5}{2}\right) \qquad \sigma_{M5} = 0.545 \ \frac{N}{mm^2}$$

$$\sigma_{M6} := \frac{M_{A6}}{I_2} \cdot \left(\frac{d_6}{2}\right) \qquad \sigma_{M6} = 0.024 \ \frac{N}{mm^2}$$

$$\sigma_{M7} := \frac{M_{A7}}{I_1} \cdot \left(\frac{d_7}{2}\right) \qquad \sigma_{M7} = 0.545 \ \frac{N}{mm^2}$$

NORMALSPENNING TIL HVERT SJIKT UT FRA NORMALFORDELING FRA BJELKE B

$$\sigma_{N1} := \frac{N_{B1}}{b \cdot d_1} \qquad \sigma_{N1} = 3.813 \ \frac{N}{mm^2}$$

$$\sigma_{N2} := \frac{N_{B2}}{b \cdot d_2} \qquad \sigma_{N2} = 0.085 \ \frac{N}{mm^2}$$

$\sigma_{N3} := \frac{N_{B3}}{b \cdot d_3}$ $\sigma_{N3} = 1.271 \frac{N}{mm^2}$

$\sigma_{N4} := \frac{N_{B4}}{b \cdot d_4}$ $\sigma_{N4} = 0 \frac{N}{mm^2}$

$\sigma_{N5} := \frac{N_{B5}}{b \cdot d_5}$ $\sigma_{N5} = 1.271 \frac{N}{mm^2}$

$\sigma_{N6} := \frac{N_{B6}}{b \cdot d_6}$ $\sigma_{N6} = 0.085 \frac{N}{mm^2}$

$\sigma_{N7} := \frac{N_{B7}}{b \cdot d_7}$ $\sigma_{N7} = 3.813 \frac{N}{mm^2}$

SUM SPENNINGER I OVERGANG MELLOM HVERT SJIKT

Sjikt 1: $\sigma_{N1} = 3.813 \frac{N}{mm^2}$

$\sigma_{N1} + \sigma_{M1} = 4.358 \frac{N}{mm^2}$

$\sigma_{N1} - \sigma_{M1} = 3.268 \frac{N}{mm^2}$

Sjikt 2: $\sigma_{N2} = 0.085 \frac{N}{mm^2}$

$\sigma_{N2} + \sigma_{M2} = 0.109 \frac{N}{mm^2}$

$\sigma_{N2} - \sigma_{M2} = 0.061 \frac{N}{mm^2}$

Sjikt 3:

$$\sigma_{N3} = 1.271 \ \frac{N}{mm^2}$$

$$\sigma_{N3} + \sigma_{M3} = 1.816 \ \frac{N}{mm^2}$$

$$\sigma_{N3} - \sigma_{M3} = 0.726 \ \frac{N}{mm^2}$$

Sjikt 4:

$$\sigma_{N4} = 0 \ \frac{N}{mm^2}$$

$$\sigma_{N4} + \sigma_{M4} = 0.024 \ \frac{N}{mm^2}$$

$$\sigma_{N4} - \sigma_{M4} = -0.024 \ \frac{N}{mm^2}$$

Sjikt 5:

$$\sigma_{N5} = 1.271 \ \frac{N}{mm^2}$$

$$\sigma_{N5} + \sigma_{M5} = 1.816 \ \frac{N}{mm^2}$$

$$\sigma_{N5} - \sigma_{M5} = 0.726 \ \frac{N}{mm^2}$$

Sjikt 6:

$$\sigma_{N6} = 0.085 \ \frac{N}{mm^2}$$

$$\sigma_{N6} + \sigma_{M6} = 0.109 \ \frac{N}{mm^2}$$

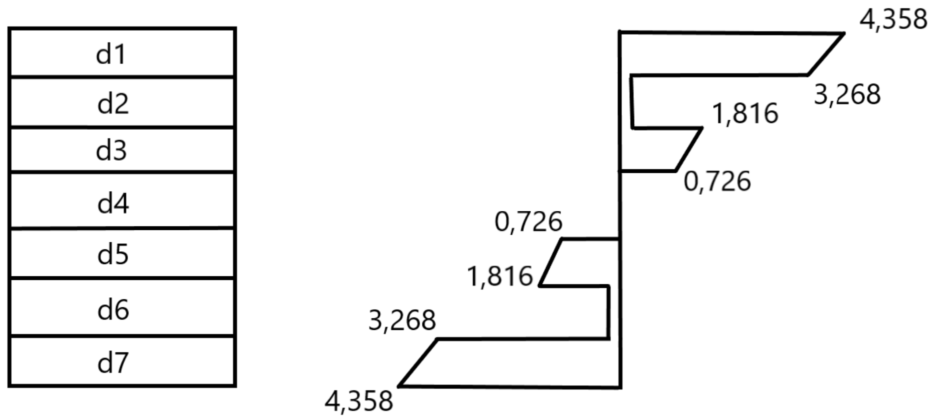
$$\sigma_{N6} - \sigma_{M6} = 0.061 \ \frac{N}{mm^2}$$

Sjikt 7: $\sigma_{N7} = 3.813\ \frac{N}{mm^2}$

$\sigma_{N7} + \sigma_{M7} = 4.358\ \frac{N}{mm^2}$

$\sigma_{N7} - \sigma_{M7} = 3.268\ \frac{N}{mm^2}$

Spenningsfordeling i kNm over tverrsnittet til et 7-sjikts element med alle sjikt av kvalitet C24 pga. et ytre bøyemoment på 27.513 kNm



KONTROLL KAPASITET

$$f_{md} := \frac{f_{mk.C24} \cdot k_{mod}}{\gamma_M} = 15.36\ \frac{N}{mm^2}$$

Utnyttelse

$$\frac{\sigma_{N7} + \sigma_{M7}}{f_{md}} = 0.284$$ < 1.0 OK

BRUKSGRENSEKONTROLL

Nedbøyning

[2] NA.A1 $d_{maks} := \frac{L}{250}$

$k_{def} := 0.6$ (Klimaklasse 1)

[1] Tab. NA.A1.1 $\psi_{0.q} := 0.7$ $\psi_{1.q} := 0.5$ $\psi_{2.q} := 0.3$

Øyeblikkelig nedbøyning

$\delta_g := \frac{5}{384} \cdot \frac{g_k \cdot L^4}{EI_{ef}}$ $\delta_g = 5.255\ mm$

$\delta_q := \frac{5}{384} \cdot \frac{q_k \cdot L^4}{EI_{ef}}$ $\delta_q = 4.05\ mm$

$w_{inst} := \delta_g + \delta_q$ $w_{inst} = 9.305\ mm$ < $\frac{L}{400} = 15\ mm$

Langtidsnedbøyning

$\delta_{gfin} := \delta_g \cdot \left(1 + k_{def}\right)$ $\delta_{gfin} = 8.408\ mm$

$\delta_{qfin} := \delta_q \cdot \left(1 + \psi_{2.q} \cdot k_{def}\right)$ $\delta_{qfin} = 4.779\ mm$

$w_{fin} := \delta_{gfin} + \delta_{qfin}$ $w_{fin} = 13.186\ mm$ < $\frac{L}{250} = 24\ mm$

BRANN DIMENSJONERING MASSIVTREDEKKE

Sjiktdata, total bøyestivhet og E-modul er likt som for bruddgrense

Referanser til standarer

[1] NS-EN 1995-1-2: Brannteknisk dimensjonering
[2] NS-EN 1990: Grunnlag for prosjektering av konstruksjoner
[3] Byggeteknisk forskrift (TEK17)
[4] NS-EN 1995-1-1: Allmenne regler og regler for bygninger

Etasjeskiller med alle sjikt av kvalitet C24

FORUTSETNINGER OG ANTAKELSER

[1] Tab 3.1 Dimensjonerende forkullingshastighet for gran: $\beta_n := 0.7 \frac{mm}{min}$

[3] § 11-3 Brannklasse 2 (Brann en side)

[3] § 11-4 Brannmotstandstid R60: $t := 60\ min$

Nyttelast, kategori A: $\psi_1 := 0.5$

Dim. last: $q_{brann} := g_k + \psi_1 \cdot q_k$ $q_{brann} = 3.595 \frac{kN}{m}$

[1] 4.2.2 EFFETIV FORKULLINGSDYBDE

$k_0 := 1$ for t > 20 min

$d_{char.n} := \beta_n \cdot t$ $d_0 := 7\ mm$

$d_{ef} := d_{char.n} + k_0 \cdot d_0$ $d_{ef} = 49\ mm$

DIMENSJONERENDE MOMENT- OG SKJÆRBELASTING

$$M_{Ed.fi} := \frac{q_{brann} \cdot L^2}{8} \qquad M_{Ed.fi} = 16.178 \ kN \cdot m \qquad V_{Ed.fi} := \frac{q_{brann} \cdot L}{2} \qquad V_{Ed.fi} = 10.785 \ kN$$

DIMENSJONERENDE FASTHETER

[4] Tab. 3.1 $k_{mod} := 0.8$

NS-EN 338 $f_m := 24 \ \frac{N}{mm^2}$ $f_v := 4 \ \frac{N}{mm^2}$ $f_{c0k} := 21 \ \frac{N}{mm^2}$ $f_{t0k} := 14 \ \frac{N}{mm^2}$

[4] Tab NA.2.3 Partialfaktor for bestandighet: $\gamma_{fi} := 1$

$$f_{m.d} := f_m \cdot \frac{k_{mod}}{\gamma_{fi}} \qquad f_{m.d} = 19.2 \ \frac{N}{mm^2} \qquad f_{v.Rd} := f_v \ \frac{k_{mod}}{\gamma_{fi}} \qquad f_{v.Rd} = 3.2 \ \frac{N}{mm^2}$$

RESTTVERSNITT ETTER BRANN

$$h_{ef.fi} := h - d_{ef} \qquad h_{ef.fi} = 191 \ mm$$

Sjikt d7

$$d_{7.fi} := d_7 - d_{ef} \qquad d_{7.fi} = -19 \ mm$$

Hele sjikt 7 forsvinner i brannen, i tillegg til 19 mm videre inn i sjikt 6.

Sjikt d6

$$d_{6.fi} := d_6 + d_{7.fi} \qquad d_{6.fi} = 21 \ mm$$

NØYTRALAKSE ETTER BRANN

Finner den nye nøytralaksen etter brann ved å vekte breddene med E-modulen.

Topplag d1

$b_{d1} := b = \left(1 \cdot 10^{3}\right)\ mm$

Tverrlag d2

$b_{d2} := \frac{b \cdot E_{90}}{E_0}$ $b_{d2} = 33.333\ mm$

Lengderetning d3

$b_{d3} := \frac{b \cdot E_0}{E_0}$ $b_{d3} = \left(1 \cdot 10^{3}\right)\ mm$

Midtlag d4

$b_{d4} := \frac{b \cdot E_{90}}{E_0}$ $b_{d4} = 33.333\ mm$

Lengderetning d5

$b_{d5} := \frac{b \cdot E_0}{E_0}$ $b_{d5} = \left(1 \cdot 10^{3}\right)\ mm$

Tverrlag d6

$b_{d6} := \frac{b \cdot E_{90}}{E_0}$ $b_{d6} = 33.333\ mm$

Nøytralakse

$$y0:=\sum_{i=1}^{n}\frac{(A_i\cdot y_i)}{A_i}$$

$$y_0:=\frac{b_{d6}\cdot d_{6.fi}\cdot\left(\frac{d_{6.fi}}{2}\right)+b_{d5}\cdot d_5\cdot\left(d_{6.fi}+\frac{d_5}{2}\right)+b_{d4}\cdot d_4\cdot\left(d_{6.fi}+d_5+\frac{d_4}{2}\right)+b_{d3}\cdot d_3\cdot\left(d_{6.fi}+d_5+d_4+\frac{d_3}{2}\right)+b_{d2}\cdot d_2\cdot\left(d_{6.fi}+d_5+d_4+d_3+\frac{d_2}{2}\right)+b_{d1}\cdot d_1\cdot\left(d_{6.fi}+d_5+d_4+d_3+d_2+\frac{d_1}{2}\right)}{b_{d6}\cdot d_{6.fi}+b_{d5}\cdot d_5+b_{d4}\cdot d_4+b_{d3}\cdot d_3+b_{d2}\cdot d_2+b_{d1}\cdot d_1}$$

$y_0=105.284\ mm$

$y:=h_{ef.fi}-y_0$ $\qquad y=85.716\ mm$

Treghetsmoment, I

$$I:=\sum_{i=1}^{n}\frac{1}{12}\cdot b\cdot d^3+A\cdot d^2$$

$$I_{eff.1}:=\left(\frac{1}{12}\cdot b_{d1}\cdot d_1^{\ 3}+\left((b_{d1}\cdot d_1)\cdot\left(y-\frac{d_1}{2}\right)^2\right)\right)+\left(\frac{1}{12}\cdot b_{d2}\cdot d_2^{\ 3}+\left((b_{d2}\cdot d_2)\cdot\left(y-d_1-\frac{d_2}{2}\right)^2\right)\right)$$

$$I_{eff.2}:=\left(\frac{1}{12}\cdot b_{d3}\cdot d_3^{\ 3}+\left((b_{d3}\cdot d_3)\cdot\left(y-d_1-d_2-\frac{d_3}{2}\right)^2\right)\right)+\left(\frac{1}{12}\cdot b_{d4}\cdot d_4^{\ 3}+\left((b_{d4}\cdot d_4)\cdot\left(y_0-\left(\frac{d_4}{2}+d_5+d_{6.fi}\right)\right)^2\right)\right)$$

$$I_{eff.3}:=\left(\frac{1}{12}\cdot b_{d5}\cdot d_3^{\ 3}+\left((b_{d5}\cdot d_5)\cdot\left(y_0-\left(\frac{d_5}{2}+d_{6.fi}\right)\right)^2\right)\right)+\left(\frac{1}{12}\cdot b_{d6}\cdot d_{6.fi}^{\ 3}+\left((b_{d6}\cdot d_{6.fi})\cdot\left(y_0-\frac{d_{6.fi}}{2}\right)^2\right)\right)$$

$I_{eff}:=I_{eff.1}+I_{eff.2}+I_{eff.3}$ $\qquad I_{eff}=\left(3.107\cdot 10^8\right)\ mm^4$

Første arealmoment, S

Restarealet til d3 ovenfor y0: $d_{3.rest} := y - d_1 - d_2$ $d_{3.rest} = 15.716\ mm$

$$S := \sum_{i=1}^{n} A_i \cdot y_i$$

$$S_{eff} := d_1 \cdot b_{d1} \cdot \left(y_0 - \frac{d_1}{2} \right) + d_2 \cdot b_{d2} \cdot \left(y_0 - d_1 - \frac{d_2}{2} \right) + d_{3.rest} \cdot b_{d3} \cdot \left(y_0 - d_1 - d_2 - \frac{d_{3.rest}}{2} \right)$$

$$S_{eff} = \left(3.213 \cdot 10^6\right)\ mm^3$$

Dimensjonerende moment og skjærkapasitet

$$M_{Rd.fi} := \frac{f_{m.d} \cdot I_{eff}}{y} \qquad V_{Rd.fi} := \frac{f_{v.Rd} \cdot I_{eff} \cdot b}{S_{eff}}$$

$$M_{Rd.fi} = 69.603\ kN \cdot m \qquad V_{Rd.fi} = 309.452\ kN$$

Utnyttelse

$\frac{M_{Ed.fi}}{M_{Rd.fi}} = 0.232 < 1.0$ OK

$\frac{V_{Ed.fi}}{V_{Rd.fi}} = 0.035 < 1.0$ OK

Opptredende trykk- og strekkraft på mest påkjente sjikt

$$F_{Ed.trykk} := \frac{M_{Ed.fi}}{y - \frac{d_1}{2}} \qquad F_{Ed.trykk} = 228.767\ kN$$

$$F_{Ed.strekk} := \frac{M_{Ed.fi}}{y_0 - \left(d_{6.fi} + \frac{d_5}{2}\right)}$$

$$F_{Ed.strekk} = 233.495 \ kN$$

Dimensjonerende trykk- og strekkapasitet til mest påkjente sjikt

$$F_{Rd.trykk} := \left(\frac{f_{c0k} \cdot 0.8}{1}\right) \cdot b \cdot d_1$$

$$F_{Rd.trykk} = 504 \ kN$$

$$F_{Rd.strekk} := \left(\frac{f_{t0k} \cdot 0.8}{1}\right) \cdot b \cdot d_5$$

$$F_{Rd.strekk} = 336 \ kN$$

Utnyttelse

$\frac{F_{Ed.trykk}}{F_{Rd.trykk}} = 0.454$ < 1.0 OK

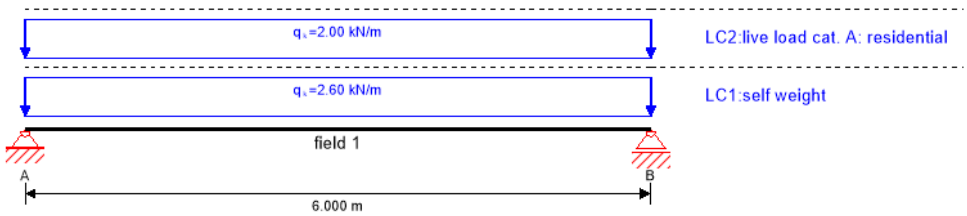
$\frac{F_{Ed.strekk}}{F_{Rd.strekk}} = 0.695$ < 1.0 OK

VEDLEGG H2.2

Marthe Elise Leirvåg

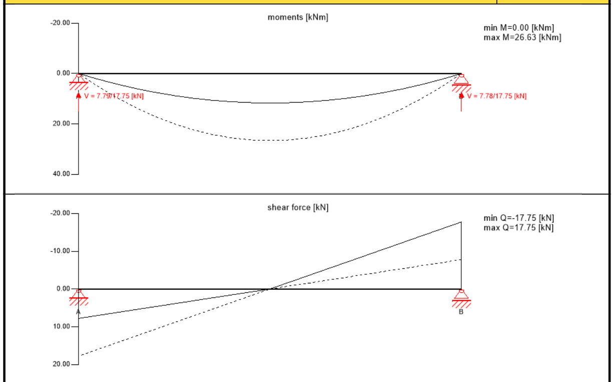
project	BYGG C	page	1
element	Etasjeskiller	date	25.05.2020

system



section: CLT 240 L7s; **material**: C24 spruce ETA (2019); **service class**: service class 1; **fire resistance class**: R 60

utilization 100 %



flexural stress analysis 25 %

$M_{y,d}$ =	26.63	kNm	$f_{m,k}$ =	24.00	N/mm²	
$N_{t,d}$ =	0.00	kN	$f_{t,k}$ =	0.00	N/mm²	
$\sigma_{t,d}$ =	0.00	N/mm²	$f_{t,d}$ =	8.96	N/mm²	
$\sigma_{m,y,d}$ =	4.30	N/mm² <	$f_{m,y,d}$ =	16.90	N/mm²	✓

shear stress analysis 4 %

V_d =	17.75	kN	$f_{v,k}$ =	4.00	N/mm²	
$\tau_{v,d}$ =	0.10	N/mm² <	$f_{v,d}$ =	2.56	N/mm²	✓

rolling shear analysis 18 %

V_d =	17.75	kN	$f_{r,k}$ =	0.86	N/mm²	
$\tau_{r,d}$ =	0.10	N/mm² <	$f_{r,d}$ =	0.55	N/mm²	✓

flexural stress analysis fire 15 %

$M_{y,d}$ =	16.18	kNm	$f_{m,k}$ =	24.00	N/mm²	
$N_{t,d}$ =	0.00	kN	$f_{t,k}$ =	0.00	N/mm²	
$\sigma_{t,d}$ =	0.00	N/mm²	$f_{t,d}$ =	16.10	N/mm²	
$\sigma_{m,y,d}$ =	-4.57	N/mm² <	$f_{m,y,d}$ =	30.36	N/mm²	✓

shear stress analysis fire 2 %

V_d =	- 10.79	kN	$f_{v,k}$ =	4.00	N/mm²	
$\tau_{v,d}$ =	0.08	N/mm² <	$f_{v,d}$ =	4.60	N/mm²	✓

rolling shear analysis fire 8 %

V_d =	-10.79	kN	$f_{r,k}$ =	0.86	N/mm²	
$\tau_{r,d}$ =	0.08	N/mm² <	$f_{r,d}$ =	0.99	N/mm²	✓

w_{inst} = w[char]

field	K_{def}	limit	w_{limit}	$w_{calc.}$	ratio
		[-]	[mm]	[mm]	
1	0.8	L/400	15.0	10.1	67 %

w_{fin} = w[char] + w[q.p.]*kdef

field	K_{def}	limit	w_{limit}	$w_{calc.}$	ratio
		[-]	[mm]	[mm]	
1	0.8	L/250	24.0	15.7	65 %

$w_{net,fin}$ = w[q.p.] + w[q.p.]*kdef

field	K_{def}	limit	w_{limit}	$w_{calc.}$	ratio
		[-]	[mm]	[mm]	
1	0.8	L/250 L/250	24.0	12.6	53 %

vibration analysis

criterion	calc.	cl. I	cl. II	cl. I	cl. II
frequency min	8.018	4.5	4.5	✓	✓
frequency	8.018	8.0	6.0	✓	✓
acceleration	0.076	0.05	0.1	×	✓
stiffness	0.107	0.25	0.5	✓	✓

support reaction

load case category	k_{mod}	A_V	B_V
		[kN]	
self weight	0.6	7.79	7.78
		7.79	7.78
live load cat. A: residential	0.8	6.00	6.00
		0.00	0.00

storaenso

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VEDLEGG H3.1

TAKTERRASSE I MASSIVTRE

Referanser til standarer

[1] NS-EN 1995-1-1: Allmenne regler og regler for bygninger
[2] NS-EN 1991-1-4: Laster på konstruksjoner
[3] NS-EN 338: Konstruksjonstrevirke - Fasthetsklasser

Prosjekteringsgrunnlag:

- 5-sjiktsmassivtre element
- 3 langsgående sjikt og 2 tverrgående sjikt
- Total tykkelse på elementet er 180mm

Bruker Schubanalogieverfahren metoden for å regne på massivtre. Dette er en mer nøyaktig modell, da den tar hensyn til sjiktenes ulike elastisitesmoduler.

<u>Tak med alle sjikt av kvalitet C24</u>

<u>FORUTSETNINGER OG ANTAKELSER</u>

$d_1 := 40\ mm$ $\quad d_3 := d_1 = 40\ mm$ $\quad d_5 := d_1 = 40\ mm$

$d_2 := 30\ mm$ $\quad d_4 := d_2 = 30\ mm$

$h := d_1 + d_2 + d_3 + d_4 + d_5 = 180\ mm$

$b := 1000\ mm$ (Ser på 1 m bredde av elemetet)

$L := 4.5\ m$

Tyngdepunkt til hvert enkelt massivtreelement:

$z_1 := \frac{h}{2} - \frac{d_1}{2}$ $\quad z_1 = 70\ mm$ $\quad z_5 := z_1 = 70\ mm$

$z_2 := \frac{h}{2} - d_1 - \frac{d_2}{2}$ $\quad z_2 = 35\ mm$ $\quad z_4 := z_2 = 35\ mm$

$z_3 := 0\ mm$

[1] NA.2.3 Partialfaktor: $\gamma_M := 1.25$

[1] Tab. 3.1 $k_{mod} := 0.8$ (Forenkler, dette er tss)

[3]

$E_0 := 11000 \cdot \frac{N}{mm^2}$ $E_{90} := \frac{E_0}{30}$

Bøyefasthet: $f_{mk.C24} := 24 \frac{N}{mm^2}$ $f_{mk.C14} := 14 \frac{N}{mm^2}$

<u>LASTER</u>

Egenvekt: $g_k := 1.025 \frac{kN}{m^2} \cdot b$

Snølast: $s_k := 3.57 \frac{kN}{m^2} \cdot b$

Vindlast: $q_{kast} := 1.34 \frac{kN}{m^2} \cdot b$

Nyttelast: $q_k := 4.0 \frac{kN}{m^2} \cdot b$

Formfaktor for flatt tak sug: $C_H := -0.7$

$q_H := q_{kast} \cdot (C_H - 0.2)$ $q_H = -1.206 \frac{kN}{m}$

Formfaktor for flatt tak trykk: $C_I := 0.2$

$q_I := q_{kast} \cdot (C_I + 0.3)$ $q_I = 0.67 \frac{kN}{m}$

Dimensjonerende verdier med kombinasjonsfaktor:

$q_1 := 1.0 \cdot g_k + 1.5 \cdot q_H$ $q_1 = -0.784 \frac{kN}{m}$ (sug)

$q_2 := 1.2 \cdot g_k + 1.5 \cdot q_k + 1.05 \cdot s_k + 1.05 \cdot q_I$ $q_2 = 11.682 \frac{kN}{m}$ (trykk)

Moment ved trykk:

$M_{Ed} := \frac{q_2 \cdot L^2}{8}$ $M_{Ed} = 29.57 \ kN \cdot m$

BØYESTIVHET

Bøyestivhet til bjelke A

$$EI_A := \frac{b}{12} \cdot \left(3 \cdot \left(E_0 \cdot {d_1}^3\right) + 2 \left(E_{90} \cdot {d_2}^3\right)\right)$$

$$EI_A = \left(1.777 \cdot 10^{11}\right) N \cdot mm^2$$

Bøyestivhet til bjelke B

$$EI_B := b \cdot \left(2 \cdot \left(E_0 \cdot d_1 \cdot {z_1}^2\right) + 2 \cdot \left(E_{90} \cdot d_2 \cdot {z_2}^2\right)\right)$$

$$EI_B = \left(4.339 \cdot 10^{12}\right) N \cdot mm^2$$

Total bøyestivhet

$$EI_{ef} := EI_A + EI_B \qquad EI_{ef} = \left(4.517 \cdot 10^{12}\right) N \cdot mm^2$$

Effektiv E-modul

$$E_{element} := \frac{EI_{ef}}{\frac{1}{12} \cdot b \cdot h^3} \qquad E_{element} = \left(9.293 \cdot 10^3\right) \frac{N}{mm^2}$$

MOMENTFORDELING

Momentfordeling mellom bjelke A og B

$$M_A := \frac{M_{Ed}}{EI_{ef}} \cdot EI_A \qquad M_A = 1.163\ kN \cdot m$$

$$M_B := \frac{M_{Ed}}{EI_{ef}} \cdot EI_B \qquad M_B = 28.407\ kN \cdot m$$

Momentfordeling til hvert sjikt

$$I_1 := \frac{1}{12} \cdot b \cdot {d_1}^3 \qquad I_2 := \frac{1}{12} \cdot b \cdot {d_2}^3$$

$$M_{A1} := \frac{E_0 \cdot I_1}{EI_A} \cdot M_A \qquad M_{A1} = 0.384 \ kN \cdot m$$

$$M_{A2} := \frac{E_{90} \cdot I_2}{EI_A} \cdot M_A \qquad M_{A2} = 0.005 \ kN \cdot m$$

$$M_{A3} := M_{A1} = 0.384 \ kN \cdot m$$

$$M_{A4} := M_{A2} = 0.005 \ kN \cdot m$$

$$M_{A5} := M_{A1} = 0.384 \ kN \cdot m$$

NORMALKRAFTFORDELING TIL HVERT SJIKT

$$N_{B1} := \frac{E_0 \cdot (b \cdot d_1) \cdot z_1}{EI_B} \cdot M_B \qquad N_{B1} = 201.647 \ kN$$

$$N_{B2} := \frac{E_{90} \cdot (b \cdot d_2) \cdot z_2}{EI_B} \cdot M_B \qquad N_{B2} = 2.521 \ kN$$

$$N_{B3} := \frac{E_0 \cdot (b \cdot d_3) \cdot z_3}{EI_B} \cdot M_B \qquad N_{B3} = 0 \ kN$$

$$N_{B4} := N_{B2} = 2.521 \ kN$$

$$N_{B5} := N_{B1} = 201.647 \ kN$$

BØYESPENNING TIL HVERT SJIKT UT FRA MOMENTFORDELING FRA BJELKE A

$$\sigma_{M1} := \frac{M_{A1}}{I_1} \cdot \left(\frac{d_1}{2}\right) \qquad \sigma_{M1} = 1.44 \ \frac{N}{mm^2}$$

$$\sigma_{M2} := \frac{M_{A2}}{I_2} \cdot \left(\frac{d_2}{2}\right) \qquad \sigma_{M2} = 0.036 \ \frac{N}{mm^2}$$

$$\sigma_{M3} := \frac{M_{A3}}{I_1} \cdot \left(\frac{d_3}{2}\right) \qquad \sigma_{M3} = 1.44 \ \frac{N}{mm^2}$$

$$\sigma_{M4} := \frac{M_{A4}}{I_2} \cdot \left(\frac{d_4}{2}\right) \qquad \sigma_{M4} = 0.036 \ \frac{N}{mm^2}$$

$$\sigma_{M5} := \frac{M_{A5}}{I_1} \cdot \left(\frac{d_5}{2}\right) \qquad \sigma_{M5} = 1.44 \ \frac{N}{mm^2}$$

NORMALSPENNING TIL HVERT SJIKT UT FRA NORMALFORDELING FRA BJELKE B

$$\sigma_{N1} := \frac{N_{B1}}{b \cdot d_1} \qquad \sigma_{N1} = 5.041 \ \frac{N}{mm^2}$$

$$\sigma_{N2} := \frac{N_{B2}}{b \cdot d_2} \qquad \sigma_{N2} = 0.084 \ \frac{N}{mm^2}$$

$$\sigma_{N3} := \frac{N_{B3}}{b \cdot d_3} \qquad \sigma_{N3} = 0 \ Pa$$

$$\sigma_{N4} := \frac{N_{B4}}{b \cdot d_4} \qquad \sigma_{N4} = 0.084 \ \frac{N}{mm^2}$$

$$\sigma_{N5} := \frac{N_{B5}}{b \cdot d_5} \qquad \sigma_{N5} = 5.041 \ \frac{N}{mm^2}$$

SUM SPENNINGER I OVERGANG MELLOM HVERT SJIKT

Sjikt 1: $\sigma_{N1} = 5.041 \ \frac{N}{mm^2}$

$$\sigma_{N1} + \sigma_{M1} = 6.482 \ \frac{N}{mm^2}$$

$$\sigma_{N1} - \sigma_{M1} = 3.601 \ \frac{N}{mm^2}$$

Sjikt 2: $\sigma_{N2} = 0.084 \ \frac{N}{mm^2}$

$$\sigma_{N2} + \sigma_{M2} = 0.12 \ \frac{N}{mm^2}$$

$$\sigma_{N2} - \sigma_{M2} = 0.048 \ \frac{N}{mm^2}$$

Sjikt 3: $\sigma_{N3}=0\ \frac{N}{mm^2}$

$\sigma_{N3}+\sigma_{M3}=1.44\ \frac{N}{mm^2}$

$\sigma_{N3}-\sigma_{M3}=-1.44\ \frac{N}{mm^2}$

Sjikt 4: $\sigma_{N4}=0.084\ \frac{N}{mm^2}$

$\sigma_{N4}+\sigma_{M4}=0.12\ \frac{N}{mm^2}$

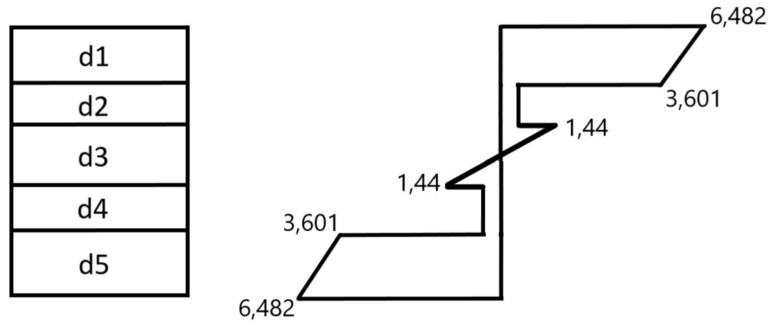
$\sigma_{N4}-\sigma_{M4}=0.048\ \frac{N}{mm^2}$

Sjikt 5: $\sigma_{N5}=5.041\ \frac{N}{mm^2}$

$\sigma_{N5}+\sigma_{M5}=6.482\ \frac{N}{mm^2}$

$\sigma_{N5}-\sigma_{M5}=3.601\ \frac{N}{mm^2}$

Spenningsfordeling i kNm over tverrsnittet til et 5-sjikts element med alle sjikt av kvalitet C24 pga. et ytre bøyemoment på 29.57kNm



KONTROLL

$$f_{md}:=\frac{f_{mk.C24}\cdot k_{mod}}{\gamma_M}=15.36\ \frac{N}{mm^2} > \quad \sigma_{N5}+\sigma_{M5}=6.482\ \frac{N}{mm^2}$$

Utnyttelse

$$\frac{\sigma_{N5}+\sigma_{M5}}{f_{md}}=0.422 \quad < 1.0 \quad \text{OK}$$

BRUKSGRENSEKONTROLL

Nedbøyning

$k_{def} := 0.6$ (Klimaklasse 1)

[1] Tab. NA.A1.1

$\psi_{0.q} := 0.7$ $\psi_{1.q} := 0.5$ $\psi_{2.q} := 0.3$

$\psi_{0.s} := 0.7$ $\psi_{1.s} := 0.5$ $\psi_{2.s} := 0.2$

$\psi_{0.v} := 0.6$ $\psi_{1.v} := 0.2$ $\psi_{2.v} := 0$

Tab 7.2 Øyeblikkelig nedbøyning

$$\delta_g := \frac{5}{384} \cdot \frac{g_k \cdot L^4}{EI_{ef}} \qquad \delta_g = 1.212\ mm$$

$$\delta_q := \frac{5}{384} \cdot \frac{q_k \cdot L^4}{EI_{ef}} \qquad \delta_q = 4.729\ mm$$

$$\delta_s := \frac{5}{384} \cdot \frac{s_k \cdot L^4}{EI_{ef}} \qquad \delta_s = 4.22\ mm$$

$$\delta_v := \frac{5}{384} \cdot \frac{q_I \cdot L^4}{EI_{ef}} \qquad \delta_v = 0.792\ mm$$

$$w_{inst} := \delta_g + \delta_q + \delta_s + \delta_v \qquad w_{inst} = 10.953\ mm \quad < \quad \frac{L}{350} = 12.857\ mm$$

Langtidsnedbøyning

$$\delta_{gfin} := \delta_g \cdot (1 + k_{def}) \qquad \delta_{gfin} = 1.939\ mm$$

$$\delta_{qfin} := \delta_q \cdot (1 + \psi_{2.q} \cdot k_{def}) \qquad \delta_{qfin} = 5.58\ mm$$

$$\delta_{sfin} := \delta_s \cdot (\psi_{0.s} + \psi_{2.s} \cdot k_{def}) \qquad \delta_{sfin} = 3.461\ mm$$

$$\delta_{vfin} := \delta_v \cdot (\psi_{0.v} + \psi_{2.v} \cdot k_{def}) \qquad \delta_{vfin} = 0.475\ mm$$

$$w_{fin} := \delta_{gfin} + \delta_{qfin} + \delta_{sfin} + \delta_{vfin} \qquad w_{fin} = 11.454\ mm \quad < \quad \frac{L}{250} = 18\ mm$$

BRANN DIMENSJONERING TAKTERRASSE
Sjiktdata, avstand fra tyngdepunkt og total
bøyestivhet er lik fra bruddgrense

Referanser til standarer
[1] NS-EN 1995-1-2: Brannteknisk dimensjonering
[2] NS-EN 1990: Grunnlag for prosjektering av konstruksjoner
[3] Byggeteknisk forskrift (TEK17)
[4] NS-EN 1995-1-1: Allmenne regler og regler for bygninger

Takterrasse med alle sjikt av kvalitet C24
FORUTSETNINGER OG ANTAKELSER

[1] Tab 3.1 Dimensjonerende forkullingshastighet for gran: $\beta_n := 0.7 \ \frac{mm}{min}$

[3] § 11-3 Brannklasse 2 (Brann en side)

[3] § 11-4 Brannmotstandstid R60: $t := 60 \ min$

[2] NA.A1.1 Nyttelast, kategori A: $\psi_1 := 0.5$ $\psi_2 := 0.2$

Dim. last: $q_{brann} := g_k + \psi_1 \cdot q_k + \psi_2 \cdot q_I + \psi_2 \cdot s_k$

$q_{brann} = 3.873 \ \frac{kN}{m}$

[1] 4.2.2 EFFETIV FORKULLINGSDYBDE

$k_0 := 1$ for t > 20 min

$d_{char.n} := \beta_n \cdot t$ $d_0 := 7 \ mm$

$d_{ef} := d_{char.n} + k_0 \cdot d_0$ $d_{ef} = 49 \ mm$

DIMENSJONERENDE MOMENT- OG SKJÆRBELASTING PÅ TAK

$M_{Ed.fi} := \frac{q_{brann} \cdot L^2}{8}$ $M_{Ed.fi} = 9.804 \ kN \cdot m$

$V_{Ed.fi} := \frac{q_{brann} \cdot L}{2}$ $V_{Ed.fi} = 8.714 \ kN$

DIMENSJONERENDE FASTHETER

[4] Tab. 3.1 $k_{mod} := 0.8$

NS-EN 338 $f_m := 24 \frac{N}{mm^2}$ $f_v := 4 \frac{N}{mm^2}$ $f_{c0k} := 21 \frac{N}{mm^2}$ $f_{t0k} := 14 \frac{N}{mm^2}$

[4] Tab NA.2.3 Partialfaktor for bestandighet: $\gamma_{fi} := 1$

$f_{m.d} := f_m \cdot \frac{k_{mod}}{\gamma_{fi}}$ $f_{m.d} = 19.2 \frac{N}{mm^2}$

$f_{v.Rd} := f_v \frac{k_{mod}}{\gamma_{fi}}$ $f_{v.Rd} = 3.2 \frac{N}{mm^2}$

<u>RESTTVERSNITT ETTER BRANN</u>

$h_{ef.fi} := h - d_{ef}$ $h_{ef.fi} = 131\ mm$

Sjikt d5

$d_{5.fi} := d_5 - d_{ef}$ $d_{5.fi} = -9\ mm$

Hele sjikt 5 forsvinner i brannen, i tillegg til 9 mm videre inn i sjikt 4.

Sjikt d4

$d_{4.fi} := d_4 + d_{5.fi}$ $d_{4.fi} = 21\ mm$

<u>NØYTRALAKSE ETTER BRANN</u>

Finner den nye nøytralaksen etter brann ved å vekte breddene med E-modulen.

Topplag d1

$b_{d1} := b = \left(1 \cdot 10^3\right)\ mm$

Tverrlag d2

$b_{d2} := \frac{b \cdot E_{90}}{E_0}$ $b_{d2} = 33.333\ mm$

Midtlag d3

$b_{d3} := \frac{b \cdot E_0}{E_0}$ $b_{d3} = \left(1 \cdot 10^3\right)\ mm$

Tverrlag d4

$b_{d4} := \frac{b \cdot E_{90}}{E_0}$ $b_{d4} = 33.333\ mm$

Nøytralakse

$$y0 := \sum_{i=1}^{n} \frac{(A_i \cdot y_i)}{A_i}$$

$$y_0 := \frac{d_{4.fi} \cdot b_{d4} \cdot \left(\frac{d_{4.fi}}{2}\right) + d_3 \cdot b_{d3} \cdot \left(d_{4.fi} + \frac{d_3}{2}\right) + d_2 \cdot b_{d2} \cdot \left(d_{4.fi} + d_3 + \frac{d_2}{2}\right) + d_1 \cdot b_{d1} \cdot \left(d_{4.fi} + d_3 + d_2 + \frac{d_1}{2}\right)}{d_{4.fi} \cdot b_{d4} + d_3 \cdot b_{d3} + d_2 \cdot b_{d2} + d_1 \cdot b_{d1}}$$

$y_0 = 75.439\ mm$

$y := h_{ef.fi} - y_0$ $\quad\quad$ $y = 55.561\ mm$

Treghetsmoment, I

$$I := \sum_{i=1}^{n} \frac{1}{12} \cdot b \cdot d^3 + A \cdot d^2$$

$$I_{eff.1} := \left(\frac{1}{12} \cdot b_{d1} \cdot d_1^{\,3} + \left((b_{d1} \cdot d_1) \cdot \left(y - \frac{d_1}{2}\right)^2\right)\right) + \left(\frac{1}{12} \cdot b_{d2} \cdot d_2^{\,3} + \left((b_{d2} \cdot d_2) \cdot \left(y - d_1 - \frac{d_2}{2}\right)^2\right)\right)$$

$$I_{eff.2} := \left(\frac{1}{12} \cdot b_{d3} \cdot d_3^{\,3} + \left((b_{d3} \cdot d_3) \cdot \left(y_0 - d_{4.fi} - \frac{d_3}{2}\right)^2\right)\right) + \left(\frac{1}{12} \cdot b_{d4} \cdot d_{4.fi}^{\,3} + \left((b_{d4} \cdot d_{4.fi}) \cdot \left(y_0 - \frac{d_{4.fi}}{2}\right)^2\right)\right)$$

$I_{eff} := I_{eff.1} + I_{eff.2}$ $\quad\quad$ $I_{eff} = (1.117 \cdot 10^8)\ mm^4$

Første arealmoment, S

Restarealet til d2 ovenfor y0: $\quad$ $d_{2.rest} := y - d_1$ $\quad$ $d_{2.rest} = 15.561\ mm$

$$S := \sum_{i=1}^{n} A_i \cdot y_i$$

$$S_{eff} := d_1 \cdot b_{d1} \cdot \left(y_0 - \frac{d_1}{2}\right) + d_{2.rest} \cdot b_{d2} \cdot \left(y_0 - d_1 - \frac{d_{2.rest}}{2}\right)$$

$S_{eff} = (2.232 \cdot 10^6)\ mm^3$

Dimensjonerende moment og skjærkapasitet

$$M_{Rd.fi} := \frac{f_{m.d} \cdot I_{eff}}{y} \quad\quad V_{Rd.fi} := \frac{f_{v.Rd} \cdot I_{eff} \cdot b}{S_{eff}}$$

$M_{Rd.fi} = 38.615\ kN \cdot m$ $\quad\quad$ $V_{Rd.fi} = 160.215\ kN$

Utnyttelse

$\frac{M_{Ed.fi}}{M_{Rd.fi}} = 0.254$ < 1.0 OK

$\frac{V_{Ed.fi}}{V_{Rd.fi}} = 0.054$ < 1.0 OK

Opptredende trykk- og strekkraft på mest påkjente sjikt

$$F_{Ed.trykk} := \frac{M_{Ed.fi}}{y - \frac{d_1}{2}}$$

$$F_{Ed.trykk} = 275.681 \; kN$$

$$F_{Ed.strekk} := \frac{M_{Ed.fi}}{y_0 - \left(d_{4.fi} + \frac{d_3}{2}\right)}$$

$$F_{Ed.strekk} = 284.665 \; kN$$

Dimensjonerende trykk- og strekkapasitet til mest påkjente sjikt

$$F_{Rd.trykk} := \left(\frac{f_{c0k} \cdot 0.8}{1}\right) \cdot b \cdot d_1$$

$$F_{Rd.trykk} = 672 \; kN$$

$$F_{Rd.strekk} := \left(\frac{f_{t0k} \cdot 0.8}{1}\right) \cdot b \cdot d_5$$

$$F_{Rd.strekk} = 448 \; kN$$

Utnyttelse

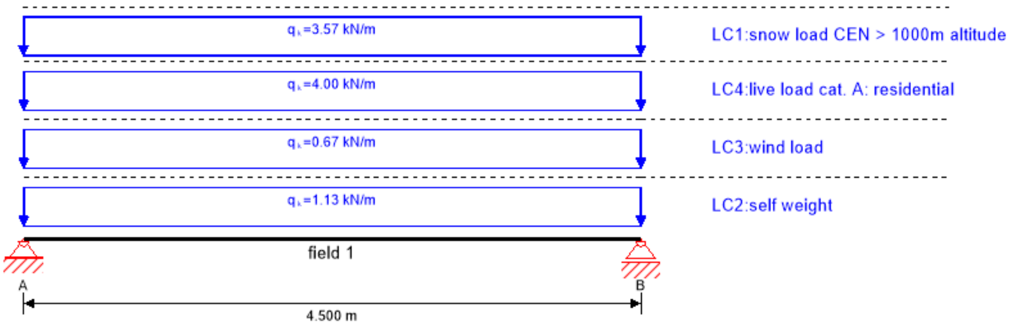
$\frac{F_{Ed.trykk}}{F_{Rd.trykk}} = 0.41$ < 1.0 OK

$\frac{F_{Ed.strekk}}{F_{Rd.strekk}} = 0.635$ < 1.0 OK

VEDLEGG H3.2

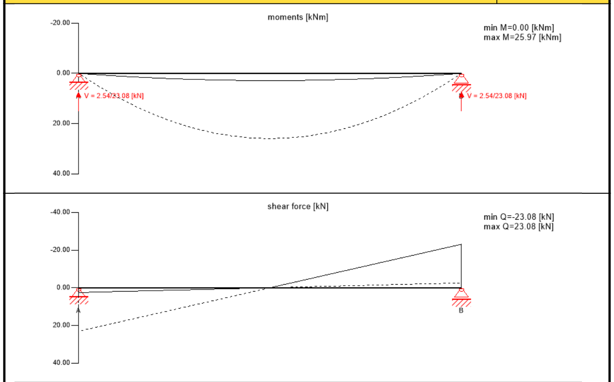
Marthe Elise Leirvåg	project	BYGG C	page	1
	element	Takterrasse	date	24.05.2020

system



section: CLT 180 L5s; **material**: C24 spruce ETA (2019); **service class**: service class 1; **fire resistance class**: R 60

utilization 80 %



flexural stress analysis 34 %

$M_{y,d}$ =	25.97	kNm	$f_{m,k}$ =	24.00	N/mm²
$N_{t,d}$ =	0.00	kN	$f_{t,k}$ =	0.00	N/mm²
$\sigma_{t,d}$ =	0.00	N/mm²	$f_{t,d}$ =	8.96	N/mm²
$\sigma_{m,y,d}$ =	-5.73	N/mm² <	$f_{m,y,d}$ =	16.90	N/mm² ✓

shear stress analysis 7 %

V_d =	23.08	kN	$f_{v,k}$ =	4.00	N/mm²
$\tau_{v,d}$ =	0.17	N/mm² <	$f_{v,d}$ =	2.56	N/mm² ✓

rolling shear analysis 22 %

V_d =	23.08	kN	$f_{r,k}$ =	1.15	N/mm²
$\tau_{r,d}$ =	0.16	N/mm² <	$f_{r,d}$ =	0.74	N/mm² ✓

flexural stress analysis fire 17 %

$M_{y,d}$ =	10.42	kNm	$f_{m,k}$ =	24.00	N/mm²
$N_{t,d}$ =	0.00	kN	$f_{t,k}$ =	0.00	N/mm²
$\sigma_{t,d}$ =	0.00	N/mm²	$f_{t,d}$ =	16.10	N/mm²
$\sigma_{m,y,d}$ =	-5.27	N/mm² <	$f_{m,y,d}$ =	30.36	N/mm² ✓

shear stress analysis fire 3 %

V_d =	-9.26	kN	$f_{v,k}$ =	4.00	N/mm²
$\tau_{v,d}$ =	0.12	N/mm² <	$f_{v,d}$ =	4.60	N/mm² ✓

rolling shear analysis fire 9 %

V_d =	-9.26	kN	$f_{r,k}$ =	1.15	N/mm²
$\tau_{r,d}$ =	0.12	N/mm² <	$f_{r,d}$ =	1.32	N/mm² ✓

w_{inst} = w[char]

field	K_{def}	limit	w_{limit}	$w_{calc.}$	ratio
		[-]	[mm]	[mm]	
1	0.8	L/350	12.9	10.3	80 %

w_{fin} = w[char] + w[q.p.]*kdef

field	K_{def}	limit	w_{limit}	$w_{calc.}$	ratio
		[-]	[mm]	[mm]	
1	0.8	L/250	18.0	13.4	74 %

$w_{net,fin}$ = w[q.p.] + w[q.p.]*kdef

field	K_{def}	limit	w_{limit}	$w_{calc.}$	ratio
		[-]	[mm]	[mm]	
1	0.8	L/300 L/300	15.0	7.0	47 %

support reaction

load case category	k_{mod}	A_V	B_V
		[kN]	
snow load CEN > 1000m altitude	0.8	8.03	8.03
		0.00	0.00
self weight	0.6	2.54	2.54
		2.54	2.54
wind load	0.9	1.51	1.51
		0.00	0.00
live load cat. A: residential	0.8	9.00	9.00

support reaction

load case category	k_{mod}	A_V	B_V
		[kN]	
		0.00	0.00

Disclaimer

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VEDLEGG I

Vegger

VEDLEGG I1.1

MASSIVTREVEGG 4-3 (LYDVEGG)

Referanse til standarder

[1] NS-EN 1995-1-1: Allmenne regler og regler for bygninger
[2] NS-EN 1990: Grunnlag for prosjektering av konstruksjoner
[3] NS-EN 338: Konstruksjonstrevirke - Fasthetsklasser
[4] NS-EN 1992-1-2: Brannteknisk dimensjonering
[5] Byggeteknisk forskrift (TEK17)

<u>FORUTSETNINGER OG ANTAKELSER</u>

Prosjekteringsgrunnlag:

- 3-sjiktsmassivtre element
- 2 langsgående sjikt og 1 tverrgående sjikt
- Total tykkelse på elementet er 100mm

Bruker Schubanalogieverfahren metoden for å regne på massivtre.

Betrakter lydveggen som to seperate massivtreelementer og ser derfor kun på et element med t=100 mm og halv lastbredde.

Alle sjikt er av kvalitet C24

Sjikt data: $d_1 := 30\ mm$ $d_2 := 40\ mm$ $d_3 := d_1 = 30\ mm$

Total tykkelse: $d := d_1 + d_2 + d_3 = 100\ mm$

Bredde: $b := 1000\ mm$ (Ser på 1 m bredde av elemetet)

Lengde: $h := 2.7\ m$

Lastbredd: $Lb := \dfrac{3.2\ m}{2}$

[1] NA.2.3 Partialfaktor: $\gamma_M := 1.25$

[1] NA.901 Klimaklasse: 1

[1] Tab. 2.1 Lastvarighetsklasse for snølast: Korttidslast

Lastvarighetsklasse for vindlasten: Øyeblikkslast $q_{kast} := 1.34\ \dfrac{kN}{m^2}$

Styrkeklasse C24:

[3] $f_{m.k} := 24 \frac{N}{mm^2}$ $f_{c0k} := 21 \frac{N}{mm^2}$ $E_0 := 12000 \cdot \frac{N}{mm^2}$

$E_{0.05} := 8040 \frac{N}{mm^2}$ $E_{90} := 370 \frac{N}{mm^2}$

LASTER

Egenlast tak: $g_{t.k} := 1.025 \frac{kN}{m^2} \cdot Lb$ $g_{t.k} = 1.64 \frac{kN}{m}$

Egenlast vegg: $g_{b.k} := 0.5 \frac{kN}{m^2} \cdot h$ $g_{b.k} = 1.35 \frac{kN}{m}$

Permanent last: $g_k := g_{t.k} + g_{b.k}$ $g_k = 2.99 \frac{kN}{m}$

Snølast: $s_k := 1.6 \frac{kN}{m^2} \cdot Lb$ $s_k = 2.56 \frac{kN}{m}$

Vindlast: $v_{t.k} := (0.2 + 0.3) \cdot q_{kast} \cdot Lb$ $v_{t.k} = 1.072 \frac{kN}{m}$

Dim. last $q_{ed} := 1.2 \cdot g_k + 1.5 \cdot s_k + 1.05 \cdot v_{t.k}$ $q_{ed} = 8.554 \frac{kN}{m}$

BRUDDGRENSE

Bøyestivhet

Tyngdepunkt til hvert enkelt massivtreelement:

$z_1 := \frac{d}{2} - \frac{d_1}{2}$ $z_1 = 35\ mm$

$z_2 := 0\ mm$

$z_3 := z_1$ $z_3 = 35\ mm$

$b_1 := 1000\ mm$

$b_2 := 1000\ mm \cdot \frac{E_{90}}{E_0}$ $b_2 = 30.833\ mm$

$$EI_A := \frac{b}{12} \cdot \left(2 \cdot E_0 \cdot d_1^{\,3} + 1 \cdot E_{90} \cdot d_2^{\,3}\right) \qquad EI_A = 55.973\ kN \cdot m^2$$

$$EI_B := b \cdot \left(2 \cdot E_0 \cdot d_1 \cdot z_1^{\,2} + 1 \cdot E_{90} \cdot d_2 \cdot z_2^{\,2}\right) \qquad EI_B = 882\ kN \cdot m^2$$

$$EI_{ef} := EI_A + EI_B \qquad EI_{ef} = 937.973\ kN \cdot m^2$$

$$EA_A := E_0 \cdot d_2 \cdot b \qquad EA_A = \left(4.8 \cdot 10^5\right)\ kN$$

$$EA_B := 2 \cdot E_0 \cdot d_1 \cdot b \qquad EA_B = \left(7.2 \cdot 10^5\right)\ kN$$

$$EA_{ef} := EA_A + EA_B \qquad EA_{ef} = \left(1.2 \cdot 10^6\right)\ kN$$

[1] 6.1.5 **Aksialkraft**

$k_{mod} := 1.0$ (forenkler her, har med vindlast)

$$f_{c0d} := f_{c0k} \cdot \frac{k_{mod}}{\gamma_M} \qquad f_{c0d} = 16.8\ \frac{N}{mm^2}$$

$$N_{ed} := q_{ed} \cdot b \qquad N_{ed} = 8.554\ kN$$

$$\sigma_{c0d} := \frac{N_{ed}}{d_1 \cdot b_1 \cdot 2 + d_2 \cdot b_2} \qquad \sigma_{c0d} = 0.14\ \frac{N}{mm^2}$$

[1] 6.3.2 **Knekking**

$$i := \sqrt{\frac{EI_{ef}}{EA_{ef}}} \qquad i = 27.958\ mm$$

Antar knekklengde: $L_{ky} := 1.0 \cdot h$

$$\lambda := \frac{L_{ky}}{i} \qquad \lambda = 96.574$$

$$\lambda_{rel} := \left(\frac{\lambda}{\pi}\right) \cdot \sqrt{\frac{f_{c0k}}{E_{0.05}}} \qquad \lambda_{rel} = 1.571$$

$$\beta_c := 0.2$$

$k_y := 0.5 \cdot \left(1 + \beta_c \cdot (\lambda_{rel} - 0.3) + \lambda_{rel}^{2}\right)$ $\qquad k_y = 1.861$

$k_{c.y} := \dfrac{1}{k_y + \sqrt{k_y^{2} - \lambda_{rel}^{2}}}$ $\qquad k_{c.y} = 0.35$

Utnyttelse

$\dfrac{\sigma_{c0d}}{k_{c.y} \cdot f_{c0d}} = 0.024$ $\quad$ < 1.0 $\quad$ OK

BRANNDIMENSJONERING

[1] Tab 3.1 Dimensjonerende forkullingshastighet for gran: $\beta_n := 0.7 \ \dfrac{mm}{min}$

[3] § 11-3 Brannklasse 2 (Brann på en side)

[3] § 11-4 Brannmotstandstid R60: $t := 60 \ min$

[1] Tab. NA.A1.1

Snølast: $\psi_{0.s} := 0.7$ $\quad \psi_{1.s} := 0.5$ $\quad \psi_{2.s} := 0.2$

Vindlast: $\psi_{0.v} := 0.6$ $\quad \psi_{1.v} := 0.2$ $\quad \psi_{2.v} := 0$

Dim. last: $q_{fi} := g_k + \psi_{1.s} \cdot s_k + \psi_{2.v} \cdot v_{t.k}$ $\qquad q_{fi} = 4.27 \ \dfrac{kN}{m}$

[1] 4.2.2 **Effektivt forkullingsdybde**

$k_0 := 1$ $\qquad$ for t > 20 min

$d_{char.n} := \beta_n \cdot t$ $\qquad d_0 := 7 \ mm$

$d_{ef} := d_{char.n} + k_0 \cdot d_0$ $\qquad d_{ef} = 49 \ mm$

Resttverrsnitt

$d_{ef.fi} := d - d_{ef}$ $\qquad d_{ef.fi} = 51 \ mm$

$d_{3.fi} := d_3 - d_{ef}$ $\quad d_{3.fi} = -19\ mm$ $\quad$ (Sjikt 3 er brent bort)

$d_{2.fi} := d_2 + d_{3.fi}$ $\quad d_{2.fi} = 21\ mm$ $\quad$ (Gjenstående av sjikt 2)

Nøytralakse etter brann

$$y := \sum_{i=1}^{3} \frac{(A_i \cdot y_i)}{A_i}$$

$$y_1 := d_{2.fi} + \frac{d_1}{2}$$

$$y := \frac{\left(d_1 \cdot b \cdot y_1 + d_{2.fi} \cdot b \cdot \frac{d_{2.fi}}{2}\right)}{d_1 \cdot b + d_{2.fi} \cdot b} \qquad y = 25.5\ mm$$

Bøyestivhet

Avstand til nøytralaksen:

$$z_{1.fi} := y - \frac{d_1}{2} \qquad z_{1.fi} = 10.5\ mm$$

$$z_{2.fi} := y - \frac{d_2}{2} \qquad z_{2.fi} = 5.5\ mm$$

$$EI_A := \frac{b}{12} \cdot \left(E_0 \cdot d_1^{\ 3} + E_{90} \cdot d_{2.fi}^{\ 3}\right) \qquad EI_A = 27.286\ kN \cdot m^2$$

$$EI_B := b \cdot \left(E_0 \cdot d_1 \cdot z_{1.fi}^{\ 2} + 1 \cdot E_{90} \cdot d_2 \cdot z_{2.fi}^{\ 2}\right) \qquad EI_B = 40.138\ kN \cdot m^2$$

$$EI_{ef} := EI_A + EI_B \qquad EI_{ef} = 67.423\ kN \cdot m^2$$

$$EA_A := E_0 \cdot d_{2.fi} \cdot b \qquad EA_A = \left(2.52 \cdot 10^5\right)\ kN$$

$$EA_B := E_0 \cdot d_1 \cdot b \qquad EA_B = \left(3.6 \cdot 10^5\right)\ kN$$

$$EA_{ef} := EA_A + EA_B \qquad EA_{ef} = \left(6.12 \cdot 10^5\right)\ kN$$

[1] 6.1.5 **Aksialkraft**

$k_{mod} := 1.0$ (forenkler her, har med vindlast)

$f_{c0d} := f_{c0k} \cdot \frac{k_{mod}}{\gamma_M}$ $f_{c0d} = 16.8 \frac{N}{mm^2}$

$N_{ed} := q_{fi} \cdot b$ $N_{ed} = 4.27\ kN$

$\sigma_{c0d} := \frac{N_{ed}}{d_1 \cdot b_1 + d_{2.fi} \cdot b_2}$ $\sigma_{c0d} = 0.139 \frac{N}{mm^2}$

[1] 6.3.2 **Knekking**

$i := \sqrt{\frac{EI_{ef}}{EA_{ef}}}$ $i = 10.496\ mm$

Antar knekklengde: $L_{ky} := 1.0 \cdot h$

$\lambda := \frac{L_{ky}}{i}$ $\lambda = 257.238$

$\lambda_{rel} := \left(\frac{\lambda}{\pi}\right) \cdot \sqrt{\frac{f_{c0k}}{E_{0.05}}}$ $\lambda_{rel} = 4.185$

$\beta_c := 0.2$

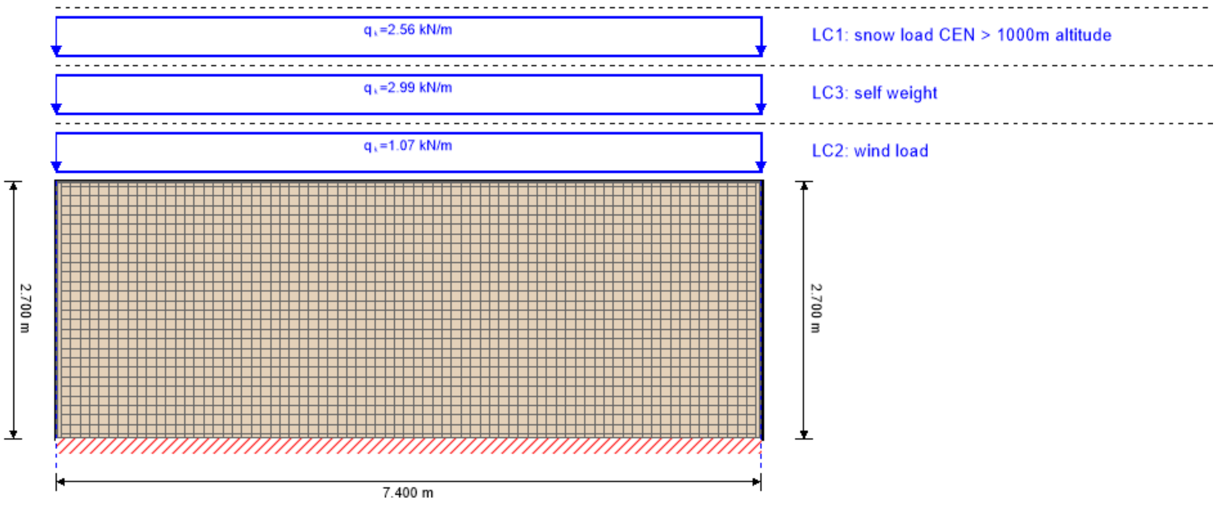
$k_y := 0.5 \cdot \left(1 + \beta_c \cdot \left(\lambda_{rel} - 0.3\right) + {\lambda_{rel}}^2\right)$ $k_y = 9.644$

$k_{c.y} := \frac{1}{k_y + \sqrt{{k_y}^2 - {\lambda_{rel}}^2}}$ $k_{c.y} = 0.055$

Utnyttelse

$\frac{\sigma_{c0d}}{k_{c.y} \cdot f_{c0d}} = 0.152$ < 1.0 OK

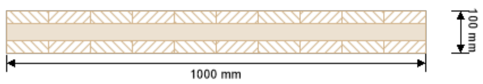
system



global utilization ratio 16 %

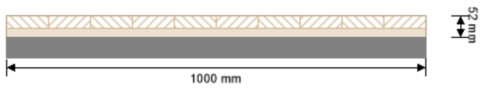
ULS	2 %	ULS fire	16 %	SLS	0 %

section: CLT 100 L3s



layer	thickness	orientation	material
1	30.0 mm	0°	C24 spruce ETA (2019)
2	40.0 mm	90°	C24 spruce ETA (2019)
3	30.0 mm	0°	C24 spruce ETA (2019)
t_{CLT}	**100.0 mm**		

section fire: CLT 100 L3s



layer	thickness	orientation	material
1	30.0 mm	0°	C24 spruce ETA (2019)
2	22.0 mm	90°	C24 spruce ETA (2019)
t_{CLT}	**52.0 mm**		
time	**60 min**		

fire resistance class:R 60

fire protection layering : no additional fire protection

k_0	d_0	$d_{char,0,h}$	$d_{ef,h}$	$d_{char,0,v}$	$d_{ef,v}$
[-]	[mm]	[mm]	[mm]	[mm]	[mm]
1	7	41.0	48.0	0.0	0.0

material values

material	$f_{m,k}$	$f_{t,0,k}$	$f_{t,90,k}$	$f_{c,0,k}$	$f_{c,90,k}$	$f_{v,k}$	$f_{r,k\ min}$	$E_{0,mean}$	G_{mean}	$G_{r,mean}$
	[N/mm²]	[N/mm²]	[N/mm²]	[N/mm²]	[N/mm²]	[N/mm²]	[N/mm²]	[N/mm²]	[N/mm²]	[N/mm²]
C24 spruce ETA (2019)	24.00	14.00	0.12	21.00	2.50	4.00	1.25	12,000.00	690.00	50.00

load

load case groups

	load case category	Typ	duration	Kmod	γ_{inf}	γ_{sup}	Ψ_0	Ψ_1	Ψ_2
LC1	snow load CEN > 1000m altitude	Q	medium term	0.8	0	1.5	0.7	0.5	0.2
LC2	wind load	Q	short term	0.9	0	1.5	0.6	0.2	0
LC3	self weight	G	permanent	0.6	1	1.35	1	1	1

LC1:snow load CEN > 1000m altitude

continuous load
q_k
[kN/m]
2.56

LC2:wind load

continuous load
q_k
[kN/m]
1.07

LC3:self weight

continuous load
q_k
[kN/m]
2.99

ULS combinations

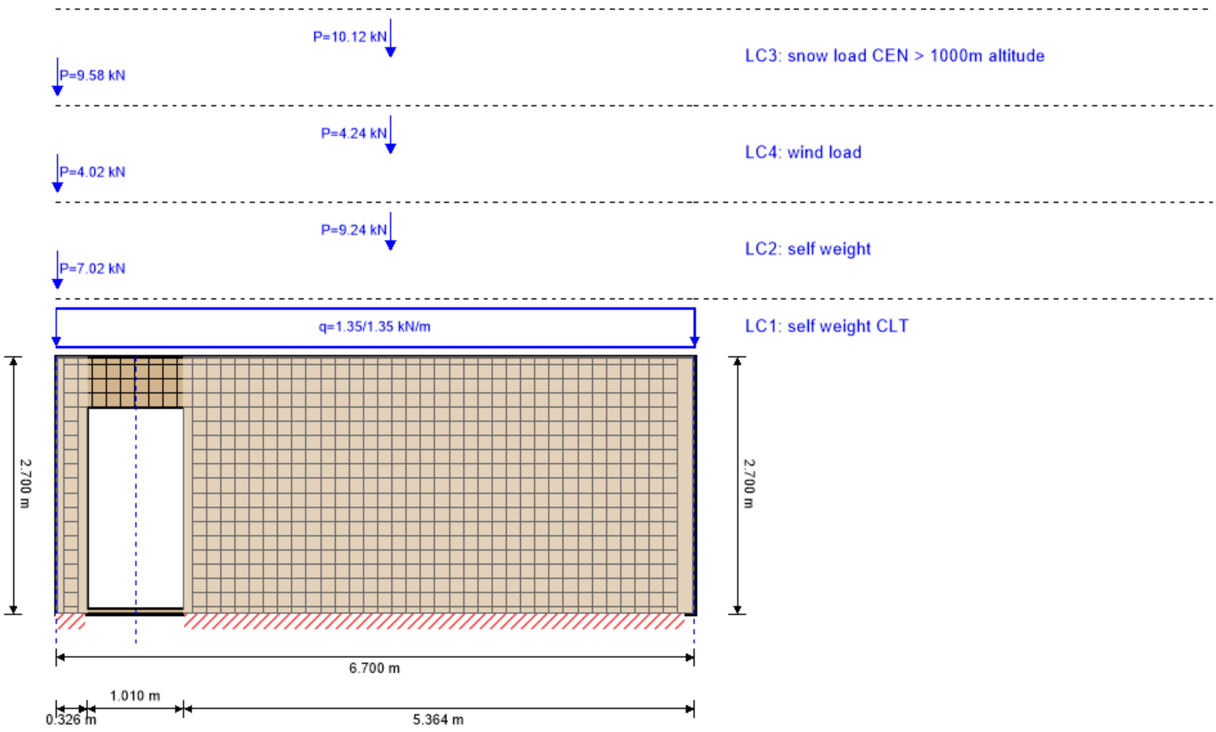
	combination rule
LCO1	1.23/1.00 * LC3
LCO2	1.23/1.00 * LC3 + 1.37/0.00 * LC1
LCO3	1.23/1.00 * LC3 + 1.37/0.00 * LC1 + 1.37/0.00 * 0.60 * LC2
LCO4	1.23/1.00 * LC3 + 1.37/0.00 * LC2
LCO5	1.23/1.00 * LC3 + 1.37/0.00 * LC2 + 1.37/0.00 * 0.70 * LC1

ULS combinations fire

	combination rule
LCO1	1.00/1.00 * LC3
LCO2	1.00/1.00 * LC3 + 1.00/0.00 * 0.50 * LC1
LCO3	1.00/1.00 * LC3 + 1.00/0.00 * 0.50 * LC1 + 1.00/0.00 * 0.00 * LC2
LCO4	1.00/1.00 * LC3 + 1.00/0.00 * 0.20 * LC2
LCO5	1.00/1.00 * LC3 + 1.00/0.00 * 0.20 * LC2 + 1.00/0.00 * 0.20 * LC1

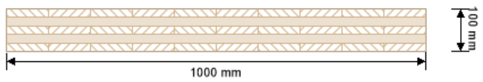
Ultimate limit state (ULS) - design results

system



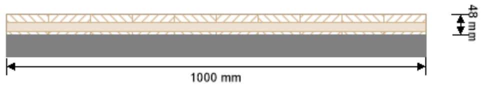
global utilization ratio						94 %
ULS	40 %	ULS fire	94 %	SLS	1 %	

section: CLT 100 L5s



layer	thickness	orientation	material
1	20.0 mm	0°	C24 spruce ETA (2019)
2	20.0 mm	90°	C24 spruce ETA (2019)
3	20.0 mm	0°	C24 spruce ETA (2019)
4	20.0 mm	90°	C24 spruce ETA (2019)
5	20.0 mm	0°	C24 spruce ETA (2019)
t_{CLT}	**100.0 mm**		

section fire: CLT 100 L5s



layer	thickness	orientation	material
1	20.0 mm	0°	C24 spruce ETA (2019)
2	20.0 mm	90°	C24 spruce ETA (2019)
3	8.0 mm	0°	C24 spruce ETA (2019)
t_{CLT}	**48.0 mm**		
time	**60 min**		

fire resistance class:R 60

fire protection layering : no additional fire protection

k_0	d_0	$d_{char,0,h}$	$d_{ef,h}$	$d_{char,0,v}$	$d_{ef,v}$
[-]	[mm]	[mm]	[mm]	[mm]	[mm]
1	7	45.0	52.0	0.0	0.0

material values

material	$f_{m,k}$	$f_{t,0,k}$	$f_{t,90,k}$	$f_{c,0,k}$	$f_{c,90,k}$	$f_{v,k}$	$f_{r,k\ min}$	$E_{0,mean}$	G_{mean}	$G_{r,mean}$
	[N/mm²]	[N/mm²]	[N/mm²]	[N/mm²]	[N/mm²]	[N/mm²]	[N/mm²]	[N/mm²]	[N/mm²]	[N/mm²]
C24 spruce ETA (2019)	24.00	14.00	0.12	21.00	2.50	4.00	1.25	12,000.00	690.00	50.00

load

load case groups

	load case category	Typ	duration	Kmod	γ_{inf}	γ_{sup}	Ψ_0	Ψ_1	Ψ_2
LC1	self weight CLT	G	permanent	0.6	1	1.35	1	1	1
LC2	self weight	G	permanent	0.6	1	1.35	1	1	1
LC3	snow load CEN > 1000m altitude	Q	medium term	0.8	0	1.5	0.7	0.5	0.2
LC4	wind load	Q	short term	0.9	0	1.5	0.6	0.2	0

LC1:self weight CLT

trapezoidal load

distance from start	$q_{k,a}$	load at end	load length
[m]	[kN/m]		[m]
0.000	1.35	1.35	6.700

LC2:self weight

point load

distance from start	P_k
[m]	[kN]
0.015	7.02
3.507	9.24

LC3:snow load CEN > 1000m altitude

point load

distance from start	P_k
[m]	[kN]
0.015	9.58
3.507	10.115

LC4:wind load

point load

distance from start	P_k
[m]	[kN]
0.015	4.015
3.507	4.235

ULS combinations

	combination rule
LCO1	1.23/1.00 * LC1 + 1.23/1.00 * LC2
LCO2	1.23/1.00 * LC1 + 1.23/1.00 * LC2 + 1.37/0.00 * LC3
LCO3	1.23/1.00 * LC1 + 1.23/1.00 * LC2 + 1.37/0.00 * LC3 + 1.37/0.00 * 0.60 * LC4
LCO4	1.23/1.00 * LC1 + 1.23/1.00 * LC2 + 1.37/0.00 * LC4
LCO5	1.23/1.00 * LC1 + 1.23/1.00 * LC2 + 1.37/0.00 * LC4 + 1.37/0.00 * 0.70 * LC3

ULS combinations fire

	combination rule
LCO1	1.00/1.00 * LC1 + 1.00/1.00 * LC2
LCO2	1.00/1.00 * LC1 + 1.00/1.00 * LC2 + 1.00/0.00 * 0.50 * LC3
LCO3	1.00/1.00 * LC1 + 1.00/1.00 * LC2 + 1.00/0.00 * 0.50 * LC3 + 1.00/0.00 * 0.00 * LC4
LCO4	1.00/1.00 * LC1 + 1.00/1.00 * LC2 + 1.00/0.00 * 0.20 * LC4
LCO5	1.00/1.00 * LC1 + 1.00/1.00 * LC2 + 1.00/0.00 * 0.20 * LC4 + 1.00/0.00 * 0.20 * LC3

VEDLEGG I3.1

MASSIVTREVEGG 3-3 (LYDVEGG)

Referanse til standarder

[1] NS-EN 1995-1-1: Allmenne regler og regler for bygninger
[2] NS-EN 1990: Grunnlag for prosjektering av konstruksjoner
[3] NS-EN 338: Konstruksjonstrevirke - Fasthetsklasser
[4] NS-EN 1992-1-2: Brannteknisk dimensjonering
[5] Byggeteknisk forskrift (TEK17)

FORUTSETNINGER OG ANTAKELSER

Prosjekteringsgrunnlag:

- 3-sjiktsmassivtre element
- 2 langsgående sjikt og 1 tverrgående sjikt
- Total tykkelse på elementet er 100mm

Bruker Schubanalogieverfahren metoden for å regne på massivtre.

Betrakter lydveggen som to seperate massivtreelementer og ser derfor kun på et element med t=100 mm og halv lastbredde.

Alle sjikt er av kvalitet C24

Sjikt data: $d_1 := 30\ mm$ $d_2 := 40\ mm$ $d_3 := d_1 = 30\ mm$

Total tykkelse: $d := d_1 + d_2 + d_3 = 100\ mm$

Bredde: $b := 1000\ mm$ (Ser på 1 m bredde av elemetet)

Lengde: $h := 2.7\ m$

Lastbredd: $Lb := \dfrac{3.2\ m}{2}$

[1] NA.2.3 Partialfaktor: $\gamma_M := 1.25$

[1] NA.901 Klimaklasse: 1

[1] Tab. 2.1 Lastvarighetsklasse for snølast: Korttidslast

Lastvarighetsklasse for vindlasten: Øyeblikkslast $q_{kast} := 1.34\ \dfrac{kN}{m^2}$

Styrkeklasse C24:

[3] $f_{m.k} := 24 \frac{N}{mm^2}$ $f_{c0k} := 21 \frac{N}{mm^2}$ $E_0 := 12000 \cdot \frac{N}{mm^2}$

$E_{0.05} := 8040 \frac{N}{mm^2}$ $E_{90} := 370 \frac{N}{mm^2}$

LASTER

Egenlast tak:	$g_{t.k} := 1.025 \frac{kN}{m^2} \cdot Lb$	$g_{t.k} = 1.64 \frac{kN}{m}$
Egenlast vegg:	$g_{b.k} := 2 \cdot 0.5 \frac{kN}{m^2} \cdot h$	$g_{b.k} = 2.7 \frac{kN}{m}$
Egenlast etasjeskiller:	$g_{e.k} := 2.595 \frac{kN}{m^2} \cdot Lb$	$g_{t.k} = 1.64 \frac{kN}{m}$
Permanent last:	$g_k := g_{t.k} + g_{b.k} + g_{e.k}$	$g_k = 8.492 \frac{kN}{m}$
Snølast:	$s_k := 1.6 \frac{kN}{m^2} \cdot Lb$	$s_k = 2.56 \frac{kN}{m}$
Vindlast:	$v_{t.k} := (0.2 + 0.3) \cdot q_{kast} \cdot Lb$	$v_{t.k} = 1.072 \frac{kN}{m}$
Nyttelast:	$q_k := 2 \frac{kN}{m^2} \cdot Lb$	$q_k = 3.2 \frac{kN}{m}$

Dim. last $q_{ed} := 1.2 \cdot g_k + 1.5 \cdot q_k + 1.05 \cdot s_k + 1.05 \cdot v_{t.k}$ $q_{ed} = 18.804 \frac{kN}{m}$

BRUDDGRENSE

Bøyestivhet

Tyngdepunkt til hvert enkelt massivtreelement:

$z_1 := \frac{d}{2} - \frac{d_1}{2}$ $z_1 = 35\ mm$

$z_2 := 0\ mm$

$z_3 := z_1$ $\quad z_3 = 35\ mm$

$b_1 := 1000\ mm$

$b_2 := 1000\ mm \cdot \frac{E_{90}}{E_0}$ $\quad b_2 = 30.833\ mm$

$EI_A := \frac{b}{12} \cdot \left(2 \cdot E_0 \cdot d_1^{\ 3} + 1 \cdot E_{90} \cdot d_2^{\ 3}\right)$ $\quad EI_A = 55.973\ kN \cdot m^2$

$EI_B := b \cdot \left(2 \cdot E_0 \cdot d_1 \cdot z_1^{\ 2} + 1 \cdot E_{90} \cdot d_2 \cdot z_2^{\ 2}\right)$ $\quad EI_B = 882\ kN \cdot m^2$

$EI_{ef} := EI_A + EI_B$ $\quad EI_{ef} = 937.973\ kN \cdot m^2$

$EA_A := E_0 \cdot d_2 \cdot b$ $\quad EA_A = \left(4.8 \cdot 10^5\right)\ kN$

$EA_B := 2 \cdot E_0 \cdot d_1 \cdot b$ $\quad EA_B = \left(7.2 \cdot 10^5\right)\ kN$

$EA_{ef} := EA_A + EA_B$ $\quad EA_{ef} = \left(1.2 \cdot 10^6\right)\ kN$

[1] 6.1.5 **Aksialkraft**

$k_{mod} := 1.0$ (forenkler her, har med vindlast)

$f_{c0d} := f_{c0k} \cdot \frac{k_{mod}}{\gamma_M}$ $\quad f_{c0d} = 16.8\ \frac{N}{mm^2}$

$N_{ed} := q_{ed} \cdot b$ $\quad N_{ed} = 18.804\ kN$

$\sigma_{c0d} := \frac{N_{ed}}{d_1 \cdot b_1 \cdot 2 + d_2 \cdot b_2}$ $\quad \sigma_{c0d} = 0.307\ \frac{N}{mm^2}$

[1] 6.3.2 **Knekking**

$i := \sqrt{\frac{EI_{ef}}{EA_{ef}}}$ $\quad i = 27.958\ mm$

Antar knekklengde: $\quad L_{ky} := 1.0 \cdot h$

$\lambda := \frac{L_{ky}}{i}$ $\lambda = 96.574$

$\lambda_{rel} := \left(\frac{\lambda}{\pi}\right) \cdot \sqrt{\frac{f_{c0k}}{E_{0.05}}}$ $\lambda_{rel} = 1.571$

$\beta_c := 0.2$

$k_y := 0.5 \cdot \left(1 + \beta_c \cdot (\lambda_{rel} - 0.3) + \lambda_{rel}^{2}\right)$ $k_y = 1.861$

$k_{c.y} := \frac{1}{k_y + \sqrt{k_y^{2} - \lambda_{rel}^{2}}}$ $k_{c.y} = 0.35$

Utnyttelse

$\frac{\sigma_{c0d}}{k_{c.y} \cdot f_{c0d}} = 0.052$ < 1.0 OK

BRANNDIMENSJONERING

[1] Tab 3.1 Dimensjonerende forkullingshastighet for gran: $\beta_n := 0.7 \frac{mm}{min}$

[3] § 11-3 Brannklasse 2 (Brann på en side)

[3] § 11-4 Brannmotstandstid R60: $t := 60\ min$

[1] Tab. NA.A1.1

Snølast: $\psi_{0.s} := 0.7$ $\psi_{1.s} := 0.5$ $\psi_{2.s} := 0.2$

Vindlast: $\psi_{0.v} := 0.6$ $\psi_{1.v} := 0.2$ $\psi_{2.v} := 0$

Nyttelast: $\psi_{0.q} := 0.7$ $\psi_{1.q} := 0.5$ $\psi_{2.q} := 0.3$

Dim. last: $q_{fi} := g_k + \psi_{1.q} \cdot q_k + \psi_{2.s} \cdot s_k + \psi_{2.v} \cdot v_{t.k}$ $q_{fi} = 10.604 \frac{kN}{m}$

[1] 4.2.2

Effektivt forkullingsdybde

$k_0 := 1$ for t > 20 min

$d_{char.n} := \beta_n \cdot t$ $d_0 := 7\ mm$

$d_{ef} := d_{char.n} + k_0 \cdot d_0$ $d_{ef} = 49\ mm$

Resttverrsnitt

$d_{ef.fi} := d - d_{ef}$ $d_{ef.fi} = 51\ mm$

$d_{3.fi} := d_3 - d_{ef}$ $d_{3.fi} = -19\ mm$ (Sjikt 3 er brent bort)

$d_{2.fi} := d_2 + d_{3.fi}$ $d_{2.fi} = 21\ mm$ (Gjenstående av sjikt 2)

Nøytralakse etter brann

$$y := \sum_{i=1}^{3} \frac{\left(A_i \cdot y_i\right)}{A_i}$$

$$y_1 := d_{2.fi} + \frac{d_1}{2}$$

$$y := \frac{\left(d_1 \cdot b \cdot y_1 + d_{2.fi} \cdot b \cdot \frac{d_{2.fi}}{2}\right)}{d_1 \cdot b + d_{2.fi} \cdot b} \qquad y = 25.5\ mm$$

Bøyestivhet

Avstand til nøytralaksen:

$z_{1.fi} := y - \frac{d_1}{2}$ $z_{1.fi} = 10.5\ mm$

$z_{2.fi} := y - \frac{d_2}{2}$ $z_{2.fi} = 5.5\ mm$

$$EI_A := \frac{b}{12} \cdot \left(E_0 \cdot d_1^{\,3} + E_{90} \cdot d_{2.fi}^{\,3}\right) \qquad EI_A = 27.286 \; kN \cdot m^2$$

$$EI_B := b \cdot \left(E_0 \cdot d_1 \cdot z_{1.fi}^{\,2} + 1 \cdot E_{90} \cdot d_2 \cdot z_{2.fi}^{\,2}\right) \qquad EI_B = 40.138 \; kN \cdot m^2$$

$$EI_{ef} := EI_A + EI_B \qquad EI_{ef} = 67.423 \; kN \cdot m^2$$

$$EA_A := E_0 \cdot d_{2.fi} \cdot b \qquad EA_A = \left(2.52 \cdot 10^5\right) \; kN$$

$$EA_B := E_0 \cdot d_1 \cdot b \qquad EA_B = \left(3.6 \cdot 10^5\right) \; kN$$

$$EA_{ef} := EA_A + EA_B \qquad EA_{ef} = \left(6.12 \cdot 10^5\right) \; kN$$

[1] 6.1.5 **Aksialkraft**

$k_{mod} := 1.0$ (forenkler her, har med vindlast)

$$f_{c0d} := f_{c0k} \cdot \frac{k_{mod}}{\gamma_M} \qquad f_{c0d} = 16.8 \; \frac{N}{mm^2}$$

$$N_{ed} := q_{fi} \cdot b \qquad N_{ed} = 10.604 \; kN$$

$$\sigma_{c0d} := \frac{N_{ed}}{d_1 \cdot b_1 + d_{2.fi} \cdot b_2} \qquad \sigma_{c0d} = 0.346 \; \frac{N}{mm^2}$$

[1] 6.3.2 **Knekking**

$$i := \sqrt{\frac{EI_{ef}}{EA_{ef}}} \qquad i = 10.496 \; mm$$

Antar knekklengde: $L_{ky} := 1.0 \cdot h$

$$\lambda := \frac{L_{ky}}{i} \qquad \lambda = 257.238$$

$$\lambda_{rel} := \left(\frac{\lambda}{\pi}\right) \cdot \sqrt{\frac{f_{c0k}}{E_{0.05}}} \qquad \lambda_{rel} = 4.185$$

$$\beta_c := 0.2$$

$$k_y := 0.5 \cdot \left(1 + \beta_c \cdot \left(\lambda_{rel} - 0.3\right) + {\lambda_{rel}}^2\right)$$ $$k_y = 9.644$$

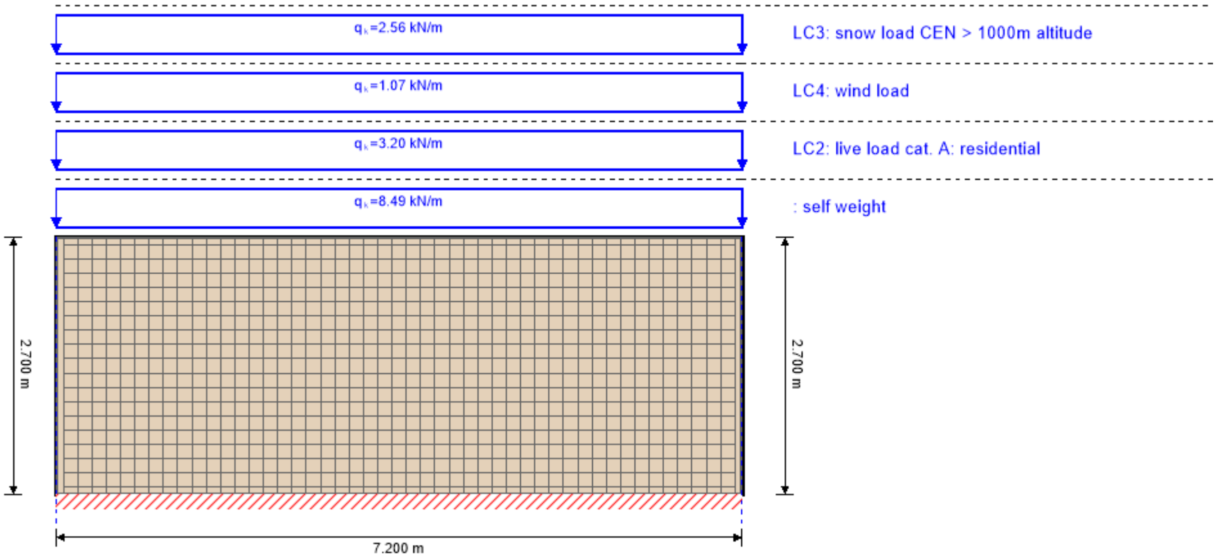
$$k_{c.y} := \frac{1}{k_y + \sqrt{{k_y}^2 - {\lambda_{rel}}^2}}$$ $$k_{c.y} = 0.055$$

Utnyttelse

$$\frac{\sigma_{c0d}}{k_{c.y} \cdot f_{c0d}} = 0.378$$ < 1.0 OK

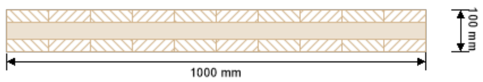
VEDLEGG I3.2

system



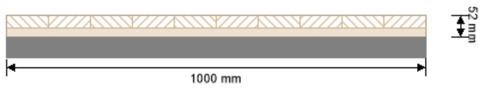
global utilization ratio						40 %
ULS	4 %	ULS fire	40 %	SLS	0 %	

section: CLT 100 L3s



layer	thickness	orientation	material
1	30.0 mm	0°	C24 spruce ETA (2019)
2	40.0 mm	90°	C24 spruce ETA (2019)
3	30.0 mm	0°	C24 spruce ETA (2019)
t_{CLT}	**100.0 mm**		

section fire: CLT 100 L3s



layer	thickness	orientation	material
1	30.0 mm	0°	C24 spruce ETA (2019)
2	22.0 mm	90°	C24 spruce ETA (2019)
t_{CLT}	**52.0 mm**		
time	**60 min**		

fire resistance class:R 60

fire protection layering : no additional fire protection

k_0	d_0	$d_{char,0,h}$	$d_{ef,h}$	$d_{char,0,v}$	$d_{ef,v}$
[-]	[mm]	[mm]	[mm]	[mm]	[mm]
1	7	41.0	48.0	0.0	0.0

material values

material	$f_{m,k}$	$f_{t,0,k}$	$f_{t,90,k}$	$f_{c,0,k}$	$f_{c,90,k}$	$f_{v,k}$	$f_{r,k\ min}$	$E_{0,mean}$	G_{mean}	$G_{r,mean}$
	[N/mm²]	[N/mm²]	[N/mm²]	[N/mm²]	[N/mm²]	[N/mm²]	[N/mm²]	[N/mm²]	[N/mm²]	[N/mm²]
C24 spruce ETA (2019)	24.00	14.00	0.12	21.00	2.50	4.00	1.25	12,000.00	690.00	50.00

load

load case groups

	load case category	Typ	duration	Kmod	γ_{inf}	γ_{sup}	Ψ_0	Ψ_1	Ψ_2
	self weight	G	permanent	0.6	1	1.35	1	1	1
LC2	live load cat. A: residential	Q	medium term	0.8	0	1.5	0.7	0.5	0.3
LC3	snow load CEN > 1000m altitude	Q	medium term	0.8	0	1.5	0.7	0.5	0.2
LC4	wind load	Q	short term	0.9	0	1.5	0.6	0.2	0

:self weight

continuous load
q_k
[kN/m]
8.49

LC2:live load cat. A: residential

continuous load
q_k
[kN/m]
3.2

LC3:snow load CEN > 1000m altitude

continuous load
q_k
[kN/m]
2.56

LC4:wind load

continuous load
q_k
[kN/m]
1.07

ULS combinations

	combination rule
LCO1	1.23/1.00 *
LCO2	1.23/1.00 * + 1.37/0.00 * LC2
LCO3	1.23/1.00 * + 1.37/0.00 * LC2 + 1.37/0.00 * 0.70 * LC3
LCO4	1.23/1.00 * + 1.37/0.00 * LC2 + 1.37/0.00 * 0.70 * LC3 + 1.37/0.00 * 0.60 * LC4
LCO5	1.23/1.00 * + 1.37/0.00 * LC3
LCO6	1.23/1.00 * + 1.37/0.00 * LC3 + 1.37/0.00 * 0.70 * LC2
LCO7	1.23/1.00 * + 1.37/0.00 * LC3 + 1.37/0.00 * 0.70 * LC2 + 1.37/0.00 * 0.60 * LC4
LCO8	1.23/1.00 * + 1.37/0.00 * LC4
LCO9	1.23/1.00 * + 1.37/0.00 * LC4 + 1.37/0.00 * 0.70 * LC2
LCO10	1.23/1.00 * + 1.37/0.00 * LC4 + 1.37/0.00 * 0.70 * LC2 + 1.37/0.00 * 0.70 * LC3

ULS combinations fire

	combination rule
LCO1	1.00/1.00 *
LCO2	1.00/1.00 * + 1.00/0.00 * 0.50 * LC2
LCO3	1.00/1.00 * + 1.00/0.00 * 0.50 * LC2 + 1.00/0.00 * 0.20 * LC3
LCO4	1.00/1.00 * + 1.00/0.00 * 0.50 * LC2 + 1.00/0.00 * 0.20 * LC3 + 1.00/0.00 * 0.00 * LC4
LCO5	1.00/1.00 * + 1.00/0.00 * 0.50 * LC3
LCO6	1.00/1.00 * + 1.00/0.00 * 0.50 * LC3 + 1.00/0.00 * 0.30 * LC2
LCO7	1.00/1.00 * + 1.00/0.00 * 0.50 * LC3 + 1.00/0.00 * 0.30 * LC2 + 1.00/0.00 * 0.00 * LC4
LCO8	1.00/1.00 * + 1.00/0.00 * 0.20 * LC4

ULS combinations fire

	combination rule 311
LCO9	1.00/1.00 * + 1.00/0.00 * 0.20 * LC4 + 1.00/0.00 * 0.30 * LC2
LCO10	1.00/1.00 * + 1.00/0.00 * 0.20 * LC4 + 1.00/0.00 * 0.30 * LC2 + 1.00/0.00 * 0.20 * LC3

VEDLEGG I4.1

MASSIVTREVEGG 2-3 (LYDVEGG)

Referanse til standarder

[1] NS-EN 1995-1-1: Allmenne regler og regler for bygninger
[2] NS-EN 1990: Grunnlag for prosjektering av konstruksjoner
[3] NS-EN 338: Konstruksjonstrevirke - Fasthetsklasser
[4] NS-EN 1992-1-2: Brannteknisk dimensjonering
[5] Byggeteknisk forskrift (TEK17)

FORUTSETNINGER OG ANTAKELSER

Prosjekteringsgrunnlag:

- 3-sjiktsmassivtre element
- 2 langsgående sjikt og 1 tverrgående sjikt
- Total tykkelse på elementet er 100mm

Bruker Schubanalogieverfahren metoden for å regne på massivtre.

Betrakter lydveggen som to seperate massivtreelementer og ser derfor kun på et element med t=100 mm og halv lastbredde.

Alle sjikt er av kvalitet C24

Sjikt data: $d_1 := 30\ mm$ $d_2 := 40\ mm$ $d_3 := d_1 = 30\ mm$

Total tykkelse: $d := d_1 + d_2 + d_3 = 100\ mm$

Bredde: $b := 1000\ mm$ (Ser på 1 m bredde av elemetet)

Lengde: $h := 2.7\ m$

Lastbredd: $Lb := \frac{3.2\ m}{2}$

[1] NA.2.3 Partialfaktor: $\gamma_M := 1.25$

[1] NA.901 Klimaklasse: 1

[1] Tab. 2.1 Lastvarighetsklasse for snølast: Korttidslast

Lastvarighetsklasse for vindlasten: Øyeblikkslast $q_{kast} := 1.34\ \frac{kN}{m^2}$

Styrkeklasse C24:

[3] $f_{m.k} := 24 \frac{N}{mm^2}$ $f_{c0k} := 21 \frac{N}{mm^2}$ $E_0 := 12000 \cdot \frac{N}{mm^2}$

$E_{0.05} := 8040 \frac{N}{mm^2}$ $E_{90} := 370 \frac{N}{mm^2}$

LASTER

Egenlast tak: $g_{t.k} := 1.025 \frac{kN}{m^2} \cdot Lb$ $g_{t.k} = 1.64 \frac{kN}{m}$

Egenlast vegg: $g_{b.k} := 3 \cdot 0.5 \frac{kN}{m^2} \cdot h$ $g_{b.k} = 4.05 \frac{kN}{m}$

Egenlast etasjeskiller: $g_{e.k} := 2 \cdot 2.595 \frac{kN}{m^2} \cdot Lb$ $g_{t.k} = 1.64 \frac{kN}{m}$

Permanent last: $g_k := g_{t.k} + g_{b.k} + g_{e.k}$ $g_k = 13.994 \frac{kN}{m}$

Snølast: $s_k := 1.6 \frac{kN}{m^2} \cdot Lb$ $s_k = 2.56 \frac{kN}{m}$

Vindlast: $v_{t.k} := (0.2 + 0.3) \cdot q_{kast} \cdot Lb$ $v_{t.k} = 1.072 \frac{kN}{m}$

Nyttelast: $q_k := 2 \cdot 2 \frac{kN}{m^2} \cdot Lb$ $q_k = 6.4 \frac{kN}{m}$

Dim. last $q_{ed} := 1.2 \cdot g_k + 1.5 \cdot q_k + 1.05 \cdot s_k + 1.05 \cdot v_{t.k}$ $q_{ed} = 30.206 \frac{kN}{m}$

BRUDDGRENSE

Bøyestivhet
Tyngdepunkt til hvert enkelt massivtreelement:

$z_1 := \frac{d}{2} - \frac{d_1}{2}$ $z_1 = 35\ mm$

$z_2 := 0\ mm$

$z_3 := z_1$ $z_3 = 35\ mm$

$b_1 := 1000\ mm$

$b_2 := 1000\ mm \cdot \dfrac{E_{90}}{E_0}$ $\quad b_2 = 30.833\ mm$

$EI_A := \dfrac{b}{12} \cdot \left(2 \cdot E_0 \cdot d_1^{\,3} + 1 \cdot E_{90} \cdot d_2^{\,3}\right)$ $\quad EI_A = 55.973\ kN \cdot m^2$

$EI_B := b \cdot \left(2 \cdot E_0 \cdot d_1 \cdot z_1^{\,2} + 1 \cdot E_{90} \cdot d_2 \cdot z_2^{\,2}\right)$ $\quad EI_B = 882\ kN \cdot m^2$

$EI_{ef} := EI_A + EI_B$ $\quad EI_{ef} = 937.973\ kN \cdot m^2$

$EA_A := E_0 \cdot d_2 \cdot b$ $\quad EA_A = \left(4.8 \cdot 10^5\right)\ kN$

$EA_B := 2 \cdot E_0 \cdot d_1 \cdot b$ $\quad EA_B = \left(7.2 \cdot 10^5\right)\ kN$

$EA_{ef} := EA_A + EA_B$ $\quad EA_{ef} = \left(1.2 \cdot 10^6\right)\ kN$

[1] 6.1.5 **Aksialkraft**

$k_{mod} := 1.0$ (forenkler her, har med vindlast)

$f_{c0d} := f_{c0k} \cdot \dfrac{k_{mod}}{\gamma_M}$ $\quad f_{c0d} = 16.8\ \dfrac{N}{mm^2}$

$N_{ed} := q_{ed} \cdot b$ $\quad N_{ed} = 30.206\ kN$

$\sigma_{c0d} := \dfrac{N_{ed}}{d_1 \cdot b_1 \cdot 2 + d_2 \cdot b_2}$ $\quad \sigma_{c0d} = 0.493\ \dfrac{N}{mm^2}$

[1] 6.3.2 **Knekking**

$i := \sqrt{\dfrac{EI_{ef}}{EA_{ef}}}$ $\quad i = 27.958\ mm$

Antar knekklengde: $L_{ky} := 1.0 \cdot h$

$\lambda := \dfrac{L_{ky}}{i}$ $\quad \lambda = 96.574$

$$\lambda_{rel} := \left(\frac{\lambda}{\pi}\right) \cdot \sqrt{\frac{f_{c0k}}{E_{0.05}}} \qquad \lambda_{rel} = 1.571$$

$$\beta_c := 0.2$$

$$k_y := 0.5 \cdot \left(1 + \beta_c \cdot \left(\lambda_{rel} - 0.3\right) + \lambda_{rel}^{\ 2}\right) \qquad k_y = 1.861$$

$$k_{c.y} := \frac{1}{k_y + \sqrt{k_y^{\ 2} - \lambda_{rel}^{\ 2}}} \qquad k_{c.y} = 0.35$$

Utnyttelse

$\frac{\sigma_{c0d}}{k_{c.y} \cdot f_{c0d}} = 0.084$ < 1.0 OK

BRANNDIMENSJONERING

[1] Tab 3.1 Dimensjonerende forkullingshastighet for gran: $\beta_n := 0.7 \frac{mm}{min}$

[3] § 11-3 Brannklasse 2 (Brann på en side)

[3] § 11-4 Brannmotstandstid R60: $t := 60\ min$

[1] Tab. NA.A1.1

Snølast:	$\psi_{0.s} := 0.7$	$\psi_{1.s} := 0.5$	$\psi_{2.s} := 0.2$
Vindlast:	$\psi_{0.v} := 0.6$	$\psi_{1.v} := 0.2$	$\psi_{2.v} := 0$
Nyttelast:	$\psi_{0.q} := 0.7$	$\psi_{1.q} := 0.5$	$\psi_{2.q} := 0.3$

Dim. last: $q_{fi} := g_k + \psi_{1.q} \cdot q_k + \psi_{2.s} \cdot s_k + \psi_{2.v} \cdot v_{t.k}$ $\qquad q_{fi} = 17.706 \frac{kN}{m}$

[1] 4.2.2 **Effektivt forkullingsdybde**

$k_0 := 1$ for t > 20 min

$d_{char.n} := \beta_n \cdot t$ $\quad$ $d_0 := 7\ mm$

$d_{ef} := d_{char.n} + k_0 \cdot d_0$ $\quad$ $d_{ef} = 49\ mm$

Resttverrsnitt

$d_{ef.fi} := d - d_{ef}$ $\quad$ $d_{ef.fi} = 51\ mm$

$d_{3.fi} := d_3 - d_{ef}$ $\quad$ $d_{3.fi} = -19\ mm$ $\quad$ (Sjikt 3 er brent bort)

$d_{2.fi} := d_2 + d_{3.fi}$ $\quad$ $d_{2.fi} = 21\ mm$ $\quad$ (Gjenstående av sjikt 2)

Nøytralakse etter brann

$$y := \sum_{i=1}^{3} \frac{\left(A_i \cdot y_i\right)}{A_i}$$

$$y_1 := d_{2.fi} + \frac{d_1}{2}$$

$$y := \frac{\left(d_1 \cdot b \cdot y_1 + d_{2.fi} \cdot b \cdot \frac{d_{2.fi}}{2}\right)}{d_1 \cdot b + d_{2.fi} \cdot b} \qquad y = 25.5\ mm$$

Bøyestivhet

Avstand til nøytralaksen:

$$z_{1.fi} := y - \frac{d_1}{2} \qquad z_{1.fi} = 10.5\ mm$$

$$z_{2.fi} := y - \frac{d_2}{2} \qquad z_{2.fi} = 5.5\ mm$$

$$EI_A := \frac{b}{12} \cdot \left(E_0 \cdot d_1^{\ 3} + E_{90} \cdot d_{2.fi}^{\ 3}\right) \qquad EI_A = 27.286\ kN \cdot m^2$$

$$EI_B := b \cdot \left(E_0 \cdot d_1 \cdot z_{1.fi}^{\ 2} + 1 \cdot E_{90} \cdot d_2 \cdot z_{2.fi}^{\ 2}\right) \qquad EI_B = 40.138\ kN \cdot m^2$$

$$EI_{ef} := EI_A + EI_B \qquad EI_{ef} = 67.423\ kN \cdot m^2$$

$EA_A := E_0 \cdot d_{2.fi} \cdot b$ $\quad EA_A = (2.52 \cdot 10^5)\ kN$

$EA_B := E_0 \cdot d_1 \cdot b$ $\quad EA_B = (3.6 \cdot 10^5)\ kN$

$EA_{ef} := EA_A + EA_B$ $\quad EA_{ef} = (6.12 \cdot 10^5)\ kN$

[1] 6.1.5 **Aksialkraft**

$k_{mod} := 1.0$ (forenkler her, har med vindlast)

$f_{c0d} := f_{c0k} \cdot \dfrac{k_{mod}}{\gamma_M}$ $\quad f_{c0d} = 16.8\ \dfrac{N}{mm^2}$

$N_{ed} := q_{fi} \cdot b$ $\quad N_{ed} = 17.706\ kN$

$\sigma_{c0d} := \dfrac{N_{ed}}{d_1 \cdot b_1 + d_{2.fi} \cdot b_2}$ $\quad \sigma_{c0d} = 0.578\ \dfrac{N}{mm^2}$

[1] 6.3.2 **Knekking**

$i := \sqrt{\dfrac{EI_{ef}}{EA_{ef}}}$ $\quad i = 10.496\ mm$

Antar knekklengde: $L_{ky} := 1.0 \cdot h$

$\lambda := \dfrac{L_{ky}}{i}$ $\quad \lambda = 257.238$

$\lambda_{rel} := \left(\dfrac{\lambda}{\pi}\right) \cdot \sqrt{\dfrac{f_{c0k}}{E_{0.05}}}$ $\quad \lambda_{rel} = 4.185$

$\beta_c := 0.2$

$k_y := 0.5 \cdot \left(1 + \beta_c \cdot (\lambda_{rel} - 0.3) + \lambda_{rel}^{\ 2}\right)$ $\quad k_y = 9.644$

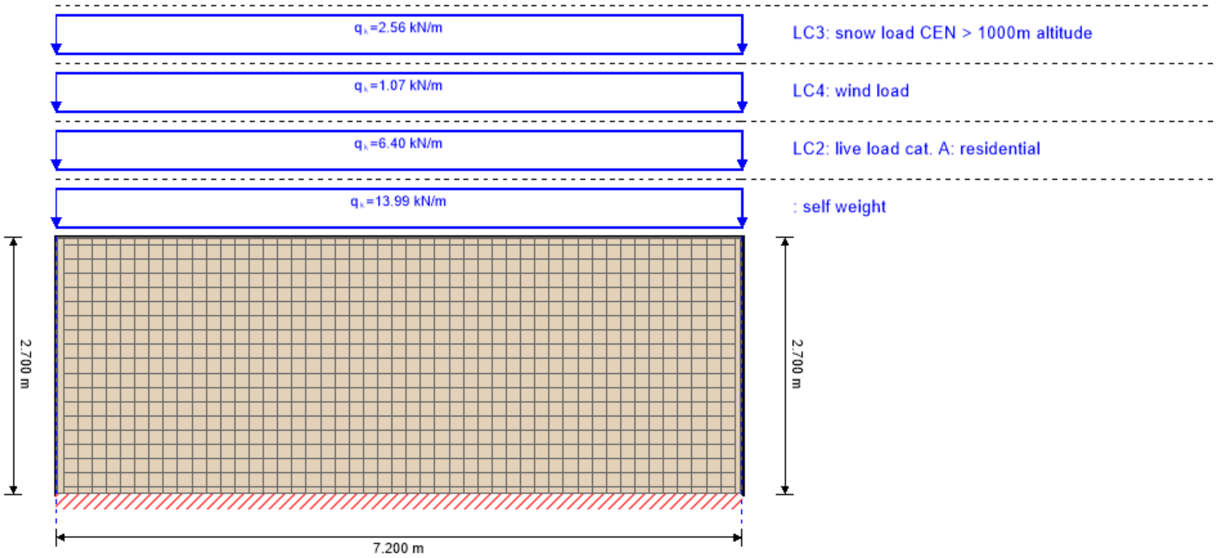
$k_{c.y} := \dfrac{1}{k_y + \sqrt{k_y^{\ 2} - \lambda_{rel}^{\ 2}}}$ $\quad k_{c.y} = 0.055$

[1] 6.1.5 **Utnyttelse**

$$\frac{\sigma_{c0d}}{k_{c.y} \cdot f_{c0d}} = 0.63 \qquad < 1.0 \qquad \text{OK}$$

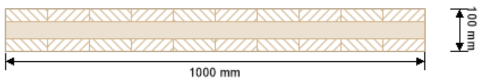
VEDLEGG I4.2

system



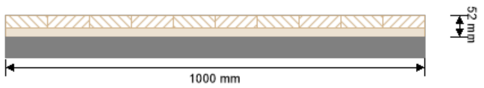
global utilization ratio						65 %
ULS	7 %	ULS fire	65 %	SLS	0 %	

section: CLT 100 L3s



layer	thickness	orientation	material
1	30.0 mm	0°	C24 spruce ETA (2019)
2	40.0 mm	90°	C24 spruce ETA (2019)
3	30.0 mm	0°	C24 spruce ETA (2019)
t_{CLT}	**100.0 mm**		

section fire: CLT 100 L3s



layer	thickness	orientation	material
1	30.0 mm	0°	C24 spruce ETA (2019)
2	22.0 mm	90°	C24 spruce ETA (2019)
t_{CLT}	**52.0 mm**		
time	**60 min**		

fire resistance class:R 60

fire protection layering : no additional fire protection

k_0	d_0	$d_{char,0,h}$	$d_{ef,h}$	$d_{char,0,v}$	$d_{ef,v}$
[-]	[mm]	[mm]	[mm]	[mm]	[mm]
1	7	41.0	48.0	0.0	0.0

material values

material	$f_{m,k}$	$f_{t,0,k}$	$f_{t,90,k}$	$f_{c,0,k}$	$f_{c,90,k}$	$f_{v,k}$	$f_{r,k\ min}$	$E_{0,mean}$	G_{mean}	$G_{r,mean}$
	[N/mm²]	[N/mm²]	[N/mm²]	[N/mm²]	[N/mm²]	[N/mm²]	[N/mm²]	[N/mm²]	[N/mm²]	[N/mm²]
C24 spruce ETA (2019)	24.00	14.00	0.12	21.00	2.50	4.00	1.25	12,000.00	690.00	50.00

load

load case groups

	load case category	Typ	duration	Kmod	γ_{inf}	γ_{sup}	Ψ_0	Ψ_1	Ψ_2
	self weight	G	permanent	0.6	1	1.35	1	1	1
LC2	live load cat. A: residential	Q	medium term	0.8	0	1.5	0.7	0.5	0.3
LC3	snow load CEN > 1000m altitude	Q	medium term	0.8	0	1.5	0.7	0.5	0.2
LC4	wind load	Q	short term	0.9	0	1.5	0.6	0.2	0

:self weight

continuous load
q_k
[kN/m]
13.99

LC2:live load cat. A: residential

continuous load
q_k
[kN/m]
6.4

LC3:snow load CEN > 1000m altitude

continuous load
q_k
[kN/m]
2.56

LC4:wind load

continuous load
q_k
[kN/m]
1.07

ULS combinations

	combination rule
LCO1	1.23/1.00 *
LCO2	1.23/1.00 * + 1.37/0.00 * LC2
LCO3	1.23/1.00 * + 1.37/0.00 * LC2 + 1.37/0.00 * 0.70 * LC3
LCO4	1.23/1.00 * + 1.37/0.00 * LC2 + 1.37/0.00 * 0.70 * LC3 + 1.37/0.00 * 0.60 * LC4
LCO5	1.23/1.00 * + 1.37/0.00 * LC3
LCO6	1.23/1.00 * + 1.37/0.00 * LC3 + 1.37/0.00 * 0.70 * LC2
LCO7	1.23/1.00 * + 1.37/0.00 * LC3 + 1.37/0.00 * 0.70 * LC2 + 1.37/0.00 * 0.60 * LC4
LCO8	1.23/1.00 * + 1.37/0.00 * LC4
LCO9	1.23/1.00 * + 1.37/0.00 * LC4 + 1.37/0.00 * 0.70 * LC2
LCO10	1.23/1.00 * + 1.37/0.00 * LC4 + 1.37/0.00 * 0.70 * LC2 + 1.37/0.00 * 0.70 * LC3

ULS combinations fire

	combination rule
LCO1	1.00/1.00 *
LCO2	1.00/1.00 * + 1.00/0.00 * 0.50 * LC2
LCO3	1.00/1.00 * + 1.00/0.00 * 0.50 * LC2 + 1.00/0.00 * 0.20 * LC3
LCO4	1.00/1.00 * + 1.00/0.00 * 0.50 * LC2 + 1.00/0.00 * 0.20 * LC3 + 1.00/0.00 * 0.00 * LC4
LCO5	1.00/1.00 * + 1.00/0.00 * 0.50 * LC3
LCO6	1.00/1.00 * + 1.00/0.00 * 0.50 * LC3 + 1.00/0.00 * 0.30 * LC2
LCO7	1.00/1.00 * + 1.00/0.00 * 0.50 * LC3 + 1.00/0.00 * 0.30 * LC2 + 1.00/0.00 * 0.00 * LC4
LCO8	1.00/1.00 * + 1.00/0.00 * 0.20 * LC4

ULS combinations fire

	combination rule
LCO9	1.00/1.00 * + 1.00/0.00 * 0.20 * LC4 + 1.00/0.00 * 0.30 * LC2
LCO10	1.00/1.00 * + 1.00/0.00 * 0.20 * LC4 + 1.00/0.00 * 0.30 * LC2 + 1.00/0.00 * 0.20 * LC3

VEDLEGG I5.1

MASSIVTREVEGG 1-3 (LYDVEGG)

Referanse til standarder

[1] NS-EN 1995-1-1: Allmenne regler og regler for bygninger
[2] NS-EN 1990: Grunnlag for prosjektering av konstruksjoner
[3] NS-EN 338: Konstruksjonstrevirke - Fasthetsklasser
[4] NS-EN 1992-1-2: Brannteknisk dimensjonering
[5] Byggeteknisk forskrift (TEK17)

FORUTSETNINGER OG ANTAKELSER

Prosjekteringsgrunnlag:

- 3-sjiktsmassivtre element
- 2 langsgående sjikt og 1 tverrgående sjikt
- Total tykkelse på elementet er 100mm

Bruker Schubanalogieverfahren metoden for å regne på massivtre.

Betrakter lydveggen som to seperate massivtreelementer og ser derfor kun på et element med t=100 mm og halv lastbredde.

Alle sjikt er av kvalitet C24

Sjikt data: $d_1 := 30\ mm$ $d_2 := 40\ mm$ $d_3 := d_1 = 30\ mm$

Total tykkelse: $d := d_1 + d_2 + d_3 = 100\ mm$

Bredde: $b := 1000\ mm$ (Ser på 1 m bredde av elemetet)

Lengde: $h := 2.7\ m$

Lastbredd: $Lb := \frac{3.2\ m}{2}$

[1] NA.2.3 Partialfaktor: $\gamma_M := 1.25$

[1] NA.901 Klimaklasse: 1

[1] Tab. 2.1 Lastvarighetsklasse for snølast: Korttidslast

Lastvarighetsklasse for vindlasten: Øyeblikkslast $q_{kast} := 1.34\ \frac{kN}{m^2}$

Styrkeklasse C24:

[3] $f_{m.k} := 24 \frac{N}{mm^2}$ $f_{c0k} := 21 \frac{N}{mm^2}$ $E_0 := 12000 \cdot \frac{N}{mm^2}$

$E_{0.05} := 8040 \frac{N}{mm^2}$ $E_{90} := 370 \frac{N}{mm^2}$

LASTER

Egenlast tak: $g_{t.k} := 1.025 \frac{kN}{m^2} \cdot Lb$ $g_{t.k} = 1.64 \frac{kN}{m}$

Egenlast vegg: $g_{b.k} := 4 \cdot 0.5 \frac{kN}{m^2} \cdot h$ $g_{b.k} = 5.4 \frac{kN}{m}$

Egenlast etasjeskiller: $g_{e.k} := 3 \cdot 2.595 \frac{kN}{m^2} \cdot Lb$ $g_{t.k} = 1.64 \frac{kN}{m}$

Permanent last: $g_k := g_{t.k} + g_{b.k} + g_{e.k}$ $g_k = 19.496 \frac{kN}{m}$

Snølast: $s_k := 1.6 \frac{kN}{m^2} \cdot Lb$ $s_k = 2.56 \frac{kN}{m}$

Vindlast: $v_{t.k} := (0.2 + 0.3) \cdot q_{kast} \cdot Lb$ $v_{t.k} = 1.072 \frac{kN}{m}$

Nyttelast: $q_k := 3 \cdot 2 \frac{kN}{m^2} \cdot Lb$ $q_k = 9.6 \frac{kN}{m}$

Dim. last $q_{ed} := 1.2 \cdot g_k + 1.5 \cdot q_k + 1.05 \cdot s_k + 1.05 \cdot v_{t.k}$ $q_{ed} = 41.609 \frac{kN}{m}$

BRUDDGRENSE

Bøyestivhet

Tyngdepunkt til hvert enkelt massivtreelement:

$z_1 := \frac{d}{2} - \frac{d_1}{2}$ $z_1 = 35\ mm$

$z_2 := 0\ mm$

$z_3 := z_1$ $\qquad z_3 = 35\ mm$

$b_1 := 1000\ mm$

$b_2 := 1000\ mm \cdot \frac{E_{90}}{E_0}$ $\qquad b_2 = 30.833\ mm$

$EI_A := \frac{b}{12} \cdot \left(2 \cdot E_0 \cdot d_1^{\ 3} + 1 \cdot E_{90} \cdot d_2^{\ 3}\right)$ $\qquad EI_A = 55.973\ kN \cdot m^2$

$EI_B := b \cdot \left(2 \cdot E_0 \cdot d_1 \cdot z_1^{\ 2} + 1 \cdot E_{90} \cdot d_2 \cdot z_2^{\ 2}\right)$ $\qquad EI_B = 882\ kN \cdot m^2$

$EI_{ef} := EI_A + EI_B$ $\qquad EI_{ef} = 937.973\ kN \cdot m^2$

$EA_A := E_0 \cdot d_2 \cdot b$ $\qquad EA_A = \left(4.8 \cdot 10^5\right)\ kN$

$EA_B := 2 \cdot E_0 \cdot d_1 \cdot b$ $\qquad EA_B = \left(7.2 \cdot 10^5\right)\ kN$

$EA_{ef} := EA_A + EA_B$ $\qquad EA_{ef} = \left(1.2 \cdot 10^6\right)\ kN$

[1] 6.1.5 **Aksialkraft**

$k_{mod} := 1.0$ (forenkler her, har med vindlast)

$f_{c0d} := f_{c0k} \cdot \frac{k_{mod}}{\gamma_M}$ $\qquad f_{c0d} = 16.8\ \frac{N}{mm^2}$

$N_{ed} := q_{ed} \cdot b$ $\qquad N_{ed} = 41.609\ kN$

$\sigma_{c0d} := \frac{N_{ed}}{d_1 \cdot b_1 \cdot 2 + d_2 \cdot b_2}$ $\qquad \sigma_{c0d} = 0.68\ \frac{N}{mm^2}$

[1] 6.3.2 **Knekking**

$i := \sqrt{\frac{EI_{ef}}{EA_{ef}}}$ $\qquad i = 27.958\ mm$

Antar knekklengde: $\qquad L_{ky} := 1.0 \cdot h$

[1] 4.2.2 **Effektivt forkullingsdybde**

$k_0 := 1$ for t > 20 min

$d_{char.n} := \beta_n \cdot t$ $d_0 := 7\ mm$

$d_{ef} := d_{char.n} + k_0 \cdot d_0$ $d_{ef} = 49\ mm$

Resttverrsnitt

$d_{ef.fi} := d - d_{ef}$ $d_{ef.fi} = 51\ mm$

$d_{3.fi} := d_3 - d_{ef}$ $d_{3.fi} = -19\ mm$ (Sjikt 3 er brent bort)

$d_{2.fi} := d_2 + d_{3.fi}$ $d_{2.fi} = 21\ mm$ (Gjenstående av sjikt 2)

Nøytralakse etter brann

$$y := \sum_{i=1}^{3} \frac{(A_i \cdot y_i)}{A_i}$$

$$y_1 := d_{2.fi} + \frac{d_1}{2}$$

$$y := \frac{\left(d_1 \cdot b \cdot y_1 + d_{2.fi} \cdot b \cdot \frac{d_{2.fi}}{2}\right)}{d_1 \cdot b + d_{2.fi} \cdot b} \qquad y = 25.5\ mm$$

Bøyestivhet

Avstand til nøytralaksen:

$z_{1.fi} := y - \frac{d_1}{2}$ $z_{1.fi} = 10.5\ mm$

$z_{2.fi} := y - \frac{d_2}{2}$ $z_{2.fi} = 5.5\ mm$

$EI_A := \frac{b}{12} \cdot \left(E_0 \cdot d_1^{\ 3} + E_{90} \cdot d_{2.fi}^{\ 3}\right)$ $EI_A = 27.286\ kN \cdot m^2$

$EI_B := b \cdot \left(E_0 \cdot d_1 \cdot z_{1.fi}^{\ 2} + 1 \cdot E_{90} \cdot d_2 \cdot z_{2.fi}^{\ 2}\right)$ $EI_B = 40.138\ kN \cdot m^2$

$EI_{ef} := EI_A + EI_B$ $EI_{ef} = 67.423\ kN \cdot m^2$

$EA_A := E_0 \cdot d_{2.fi} \cdot b$ $EA_A = \left(2.52 \cdot 10^5\right)\ kN$

$EA_B := E_0 \cdot d_1 \cdot b$ $EA_B = \left(3.6 \cdot 10^5\right)\ kN$

$EA_{ef} := EA_A + EA_B$ $EA_{ef} = \left(6.12 \cdot 10^5\right)\ kN$

[1] 6.1.5 **Aksialkraft**

$k_{mod} := 1.0$ (forenkler her, har med vindlast)

$f_{c0d} := f_{c0k} \cdot \frac{k_{mod}}{\gamma_M}$ $f_{c0d} = 16.8\ \frac{N}{mm^2}$

$N_{ed} := q_{fi} \cdot b$ $N_{ed} = 24.808\ kN$

$\sigma_{c0d} := \frac{N_{ed}}{d_1 \cdot b_1 + d_{2.fi} \cdot b_2}$ $\sigma_{c0d} = 0.809\ \frac{N}{mm^2}$

[1] 6.3.2 **Knekking**

$i := \sqrt{\frac{EI_{ef}}{EA_{ef}}}$ $i = 10.496\ mm$

Antar knekklengde: $L_{ky} := 1.0 \cdot h$

$\lambda := \frac{L_{ky}}{i}$ $\lambda = 257.238$

$\lambda_{rel} := \left(\frac{\lambda}{\pi}\right) \cdot \sqrt{\frac{f_{c0k}}{E_{0.05}}}$ $\lambda_{rel} = 4.185$

$\beta_c := 0.2$

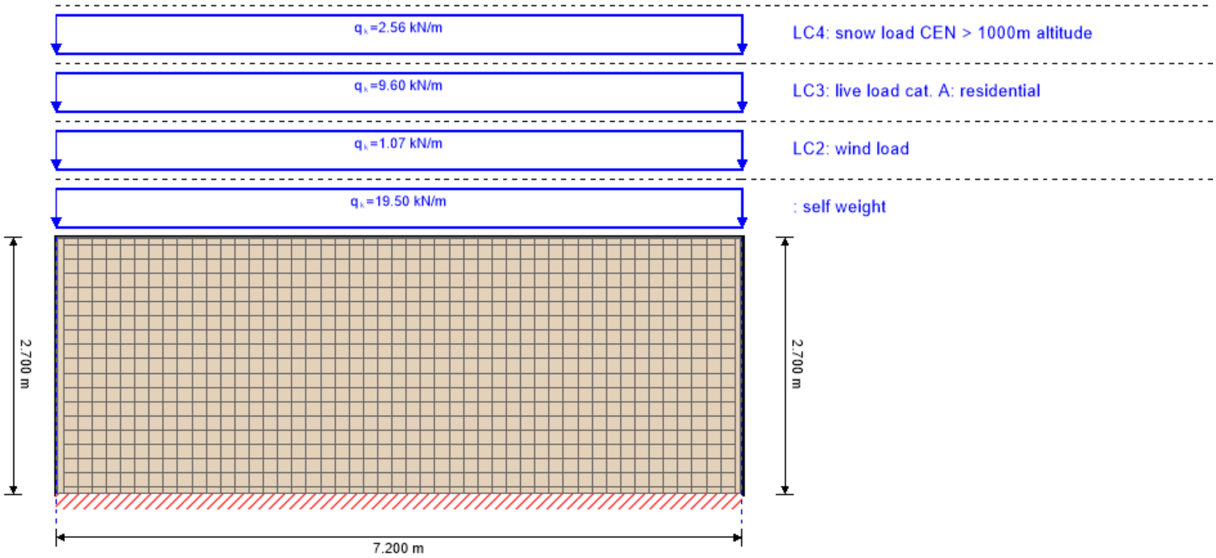
$$k_y := 0.5 \cdot \left(1 + \beta_c \cdot \left(\lambda_{rel} - 0.3\right) + \lambda_{rel}^{\ 2}\right) \qquad k_y = 9.644$$

$$k_{c.y} := \frac{1}{k_y + \sqrt{k_y^{\ 2} - \lambda_{rel}^{\ 2}}} \qquad k_{c.y} = 0.055$$

Utnyttelse

$$\frac{\sigma_{c0d}}{k_{c.y} \cdot f_{c0d}} = 0.883$$ < 1.0 OK

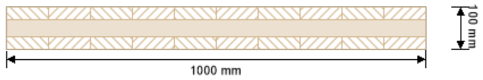
system



global utilization ratio 91 %

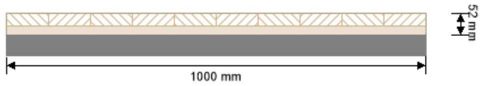
ULS	9 %	ULS fire	91 %	SLS	0 %

section: CLT 100 L3s



layer	thickness	orientation	material
1	30.0 mm	0°	C24 spruce ETA (2019)
2	40.0 mm	90°	C24 spruce ETA (2019)
3	30.0 mm	0°	C24 spruce ETA (2019)
t_{CLT}	**100.0 mm**		

section fire: CLT 100 L3s



layer	thickness	orientation	material
1	30.0 mm	0°	C24 spruce ETA (2019)
2	22.0 mm	90°	C24 spruce ETA (2019)
t_{CLT}	**52.0 mm**		
time	**60 min**		

fire resistance class:R 60
fire protection layering : no additional fire protection

k_0	d_0	$d_{char,0,h}$	$d_{ef,h}$	$d_{char,0,v}$	$d_{ef,v}$
[-]	[mm]	[mm]	[mm]	[mm]	[mm]
1	7	41.0	48.0	0.0	0.0

material values

material	$f_{m,k}$	$f_{t,0,k}$	$f_{t,90,k}$	$f_{c,0,k}$	$f_{c,90,k}$	$f_{v,k}$	$f_{r,k\ min}$	$E_{0,mean}$	G_{mean}	$G_{r,mean}$
	[N/mm²]	[N/mm²]	[N/mm²]	[N/mm²]	[N/mm²]	[N/mm²]	[N/mm²]	[N/mm²]	[N/mm²]	[N/mm²]
C24 spruce ETA (2019)	24.00	14.00	0.12	21.00	2.50	4.00	1.25	12,000.00	690.00	50.00

load

load case groups

	load case category	Typ	duration	Kmod	γ_{inf}	γ_{sup}	Ψ_0	Ψ_1	Ψ_2
	self weight	G	permanent	0.6	1	1.35	1	1	1
LC2	wind load	Q	short term	0.9	0	1.5	0.6	0.2	0
LC3	live load cat. A: residential	Q	medium term	0.8	0	1.5	0.7	0.5	0.3
LC4	snow load CEN > 1000m altitude	Q	medium term	0.8	0	1.5	0.7	0.5	0.2

:self weight

continuous load
q_k
[kN/m]
19.5

LC2:wind load

continuous load
q_k
[kN/m]
1.07

LC3:live load cat. A: residential

continuous load
q_k
[kN/m]
9.6

LC4:snow load CEN > 1000m altitude

continuous load
q_k
[kN/m]
2.56

ULS combinations

	combination rule
LCO1	1.23/1.00 *
LCO2	1.23/1.00 * + 1.37/0.00 * LC2
LCO3	1.23/1.00 * + 1.37/0.00 * LC2 + 1.37/0.00 * 0.70 * LC3
LCO4	1.23/1.00 * + 1.37/0.00 * LC2 + 1.37/0.00 * 0.70 * LC3 + 1.37/0.00 * 0.70 * LC4
LCO5	1.23/1.00 * + 1.37/0.00 * LC3
LCO6	1.23/1.00 * + 1.37/0.00 * LC3 + 1.37/0.00 * 0.60 * LC2
LCO7	1.23/1.00 * + 1.37/0.00 * LC3 + 1.37/0.00 * 0.60 * LC2 + 1.37/0.00 * 0.70 * LC4
LCO8	1.23/1.00 * + 1.37/0.00 * LC4
LCO9	1.23/1.00 * + 1.37/0.00 * LC4 + 1.37/0.00 * 0.60 * LC2
LCO10	1.23/1.00 * + 1.37/0.00 * LC4 + 1.37/0.00 * 0.60 * LC2 + 1.37/0.00 * 0.70 * LC3

ULS combinations fire

	combination rule
LCO1	1.00/1.00 *
LCO2	1.00/1.00 * + 1.00/0.00 * 0.20 * LC2
LCO3	1.00/1.00 * + 1.00/0.00 * 0.20 * LC2 + 1.00/0.00 * 0.30 * LC3
LCO4	1.00/1.00 * + 1.00/0.00 * 0.20 * LC2 + 1.00/0.00 * 0.30 * LC3 + 1.00/0.00 * 0.20 * LC4
LCO5	1.00/1.00 * + 1.00/0.00 * 0.50 * LC3
LCO6	1.00/1.00 * + 1.00/0.00 * 0.50 * LC3 + 1.00/0.00 * 0.00 * LC2
LCO7	1.00/1.00 * + 1.00/0.00 * 0.50 * LC3 + 1.00/0.00 * 0.00 * LC2 + 1.00/0.00 * 0.20 * LC4
LCO8	1.00/1.00 * + 1.00/0.00 * 0.50 * LC4

ULS combinations fire

	combination rule
LCO9	1.00/1.00 * + 1.00/0.00 * 0.50 * LC4 + 1.00/0.00 * 0.00 * LC2
LCO10	1.00/1.00 * + 1.00/0.00 * 0.50 * LC4 + 1.00/0.00 * 0.00 * LC2 + 1.00/0.00 * 0.30 * LC3

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VEDLEGG J

Stabilitet

J. Stabilitet

- J1. Skjevstillingslaster
  - *J1.1 Mathcad*
  - *J1.2 Excel*
- J2. Global stabilitet
  - *J2.1 Mathcad*

VEDLEGG J1.1

BEREGNING AV HELNINGSVINKEL TIL SKJEVSTILLINGSLAST

Referanser til standarder

NS-EN 1992-1-1: Allmenne regler og regler for bygninger

- Kapittel 5: Punkt 5.2(5)

REDUKSJONSFAKTOR

Basisverdi for helning: $\theta_0 := \frac{1}{200}$

$h := 12.62$

Reduksjonsfaktor for høyden til konstruksjonen, h: $\alpha_h := \frac{2}{\sqrt{h}}$ $\alpha_h = 0.563$

Krav til reduksjonsfaktor: $\frac{2}{3} \le \alpha_h \le 1$

$\frac{2}{3} = 0.667$

Riktig reduksjonsfaktor blir da: $\alpha_h := 0.67$

ANTALL KONSTRUKSJONSDELER I AKSENE

Antall konstruksjonsdeler i y-akse: $m_y := 2$ (Ytre søylerad på vestsiden)

Antall konstruksjonsdeler i x-akse: $m_x := 3$ (To søyler og en vegg på nordsiden)

HELNINGSVINKEL FOR VERTIKALE BÆREELEMENTER

y-akse: $\alpha_{my} := \sqrt{0.5 \cdot \left(1 + \frac{1}{m_y}\right)}$ $\alpha_{my} = 0.866$

$\theta_{iy} := \theta_0 \cdot \alpha_h \cdot \alpha_{my}$ $\theta_{iy} = 0.003$

x-akse: $\alpha_{mx} := \sqrt{0.5 \cdot \left(1 + \frac{1}{m_x}\right)}$ $\alpha_{mx} = 0.816$

$\theta_{ix} := \theta_0 \cdot \alpha_h \cdot \alpha_{mx}$ $\theta_{ix} = 0.003$

Helningen blir lik 0.003 i begge retninger, både x- og y-retning.

VEDLEGG J1.2

Skjevstillingslaster

Etasje	Areal [m^2]	Egenlast g [kN/m^2]	Nyttelast p [kN/m^2]	Snølast s [kN/m^2]	Helning Ø i x- og y-retning	Horisontallast fra egenvekt Hg = Ø x g x areal	Horisontallast fra nyttelast Hq = Ø x p x areal	Horisontallast fra snølast Hs = Ø x s x areal
1	300	1,608	2	0	0,003	1,4472	1,8	0
2	310,5	1,608	2	0	0,003	1,497852	1,863	0
3	310,5	1,608	2	0	0,003	1,497852	1,863	0
4: Inne	250	1,608	2	0	0,003	1,206	1,5	0
4: Takterrasse	60,5	1,608	4	3,57	0,003	0,291852	0,726	0,647955
Takplan	250	1,125	0	1,6	0,003	0,84375	0	1,2

Etasje	Total horisontallast [kN]*	Lengde av dekke i y-retning [m]	Horisontallast per meter dekke y-retning [kN/m]	Lengde av dekke i x-retning [m]	Horisontallast per meter dekke x-retning [kN/m]
1	3,2472	13,5	0,240533333	27,2	0,119382353
2	3,360852	13,5	0,248952	27,2	0,123560735
3	3,360852	13,5	0,248952	27,2	0,123560735
4	4,371807	13,5	0,323837556	27,2	0,160728199
Takplan	2,04375	13,5	0,151388889	20,3	0,10067734

**gjelder for både x- og y-retning*
Lengder og areal er forenklet

VEDLEGG J2.1

BEREGNING AV GLOBAL STABILITET

Referanser til standarder

[1] NS-EN 1991-1-4: Laster på konstruksjoner - Almenne laster: Vindlaster

Prosjekteringsgrunnlag:

- Vindlast lo side, side D, trykk
- Vindlast le side, side E, sug

Beregning av veltende moment

VINDLASTER

$$q_D := 1120 \frac{N}{m^2}$$

$$q_E := 617.942 \frac{N}{m^2}$$

DIMENSJONER

[1] Fig. 7.5

$$h := 12.75\ m$$

$$h_{etasje} := 3.025\ m$$

$$b := 29.490\ m$$

$$d := 18.31\ m$$

REDUKSJON I KORRELASJON

[1] 7.2.2(3)

$$\frac{h}{d} = 0.696 \qquad \frac{h}{d} < 1 \qquad K_{red} := 0.85$$

VELTENDE MOMENT

$LF := 1.5$ (Lastfaktor lik 1,5)

$q_{VK} := (q_D + q_E) \cdot K_{red} \cdot b$ $q_{VK} = 43.564 \ \frac{kN}{m}$

$M_{VK} := q_{VK} \cdot \frac{h^2}{2}$ $M_{VK} = (3.541 \cdot 10^3) \ kN \cdot m$

$M_{Vd} := M_{VK} \cdot LF$ $M_{Vd} = (5.311 \cdot 10^3) \ kN \cdot m$

Stabiliserende moment

DIMENSJONERENDE KRAFT FRA HEIS- OG TRAPPESJAKT

$O_{sjakt} := (7 \ m + 2.8 \ m + 2.8 \ m + 7 \ m + 2.1 \ m + 2.1 \ m + 3 \ m)$

$O_{sjakt} = 26.8 \ m$ (Omkrets sjakt)

$t_{vegg} := 0.24 \ m$ (Tykkelse sjakt)

$A_{sjakt} := O_{sjakt} \cdot t_{vegg}$ $A_{sjakt} = 6.432 \ m^2$ (Areal sjakt)

$Egenvekt_{tre} := 350 \ \frac{kg}{m^3}$

$Tyngdetetthet_{tre} := Egenvekt_{tre} \cdot 9.81 \ \frac{m}{s^2}$

$Tyngdetetthet_{tre} = (3.434 \cdot 10^3) \ \frac{kg}{m^2 \cdot s^2}$

$H_{sjakt} := 5 \cdot 3.025 \ m$ $H_{sjakt} = 15.125 \ m$

$Volum_{sjakt} := A_{sjakt} \cdot H_{sjakt}$ $Volum_{sjakt} = 97.284 \ m^3$

$LF := 0.9$ (Lastfaktor lik 0,9)

$q_{k.sjakt} := Tyngdetetthet_{tre} \cdot Volum_{sjakt}$ $q_{k.sjakt} = 334.025\ kN$

$q_{d.sjakt} := q_{k.sjakt} \cdot LF$ $q_{d.sjakt} = 300.622\ kN$

DIMENSJONERENDE KRAFT FRA ØVRIGE VEGGER

$b := 6.6\ m$

$t_{vegg} = 0.24\ m$

$A_{vegg} := b \cdot t_{vegg}$ $A_{vegg} = 1.584\ m^2$

$H_{vegg} := 2.6\ m$

$Volum_{vegg} := A_{vegg} \cdot H_{vegg}$ $Volum_{vegg} = 4.118\ m^3$

$Tyngde_{vegg} := Tyngdetetthet_{tre} \cdot Volum_{vegg}$ $Tyngde_{vegg} = 14.141\ kN$

$stk := 10$

$Tyngde_{total} := Tyngde_{vegg} \cdot stk$ $Tyngde_{total} = 141.405\ kN$

$q_{k.vegger} := Tyngde_{total}$ $q_{k.vegger} = 141.405\ kN$

$q_{d.vegger} := q_{k.vegger} \cdot LF$ $q_{d.vegger} = 127.265\ kN$

STABILISERENDE MOMENT

$b_{bygg} := 29.49\ m$

$M_{Sd} := (q_{d.sjakt} + q_{d.vegger}) \cdot \frac{b_{bygg}}{2}$ $\qquad M_{Sd} = (6.309 \cdot 10^{3})\ kN \cdot m$

Konklusjon

$M_{Sd} = (6.309 \cdot 10^{3})\ kN \cdot m$ (Stabiliserende moment)

$M_{Vd} = (5.311 \cdot 10^{3})\ kN \cdot m$ (Veltende moment)

$M_{Sd} > M_{Vd}$ Stabiliserende moment er større enn veltende moment, global stabilitet ok.

VEDLEGG K

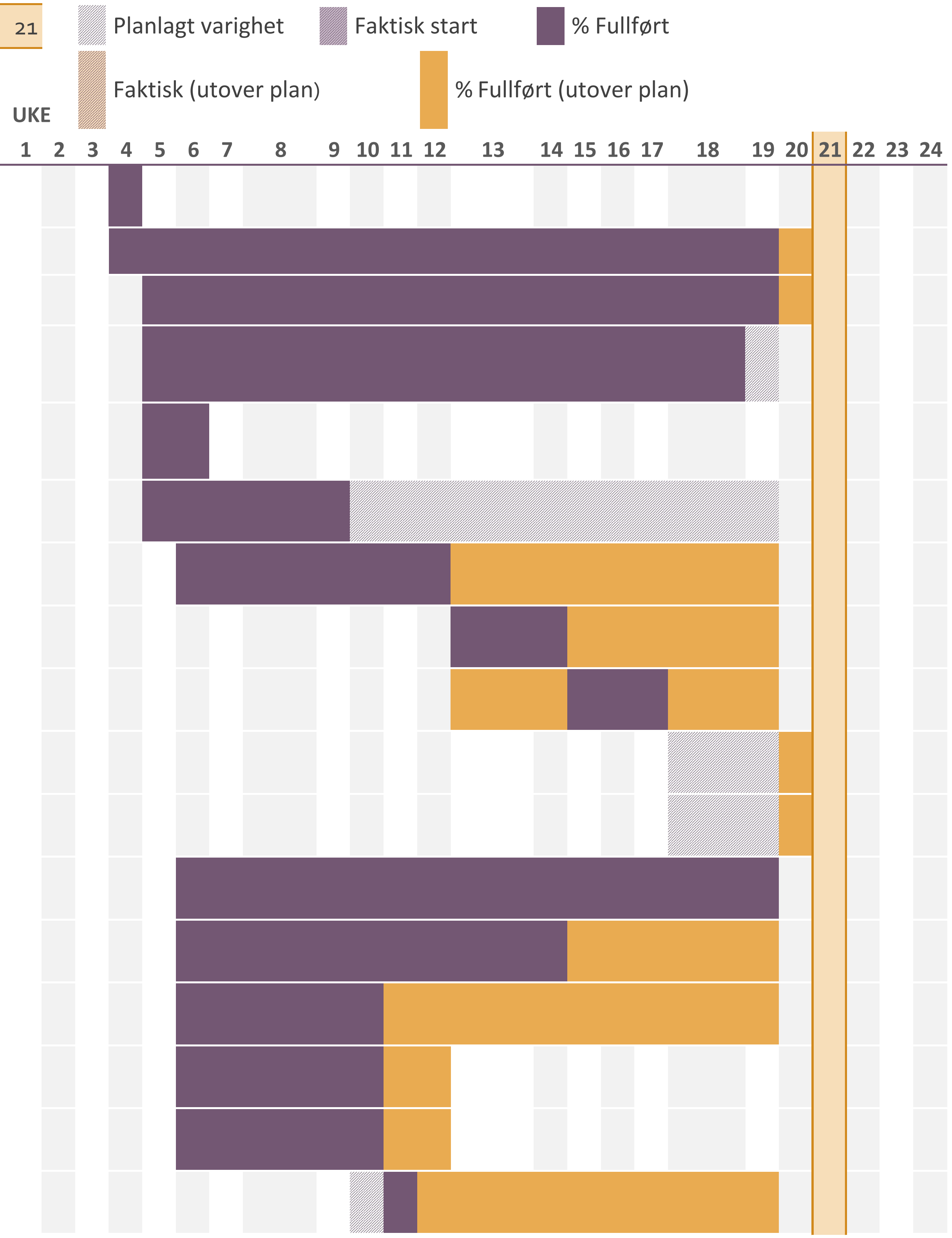
Fremdriftsplan

Fremdriftsplan bacheloroppgave 2020

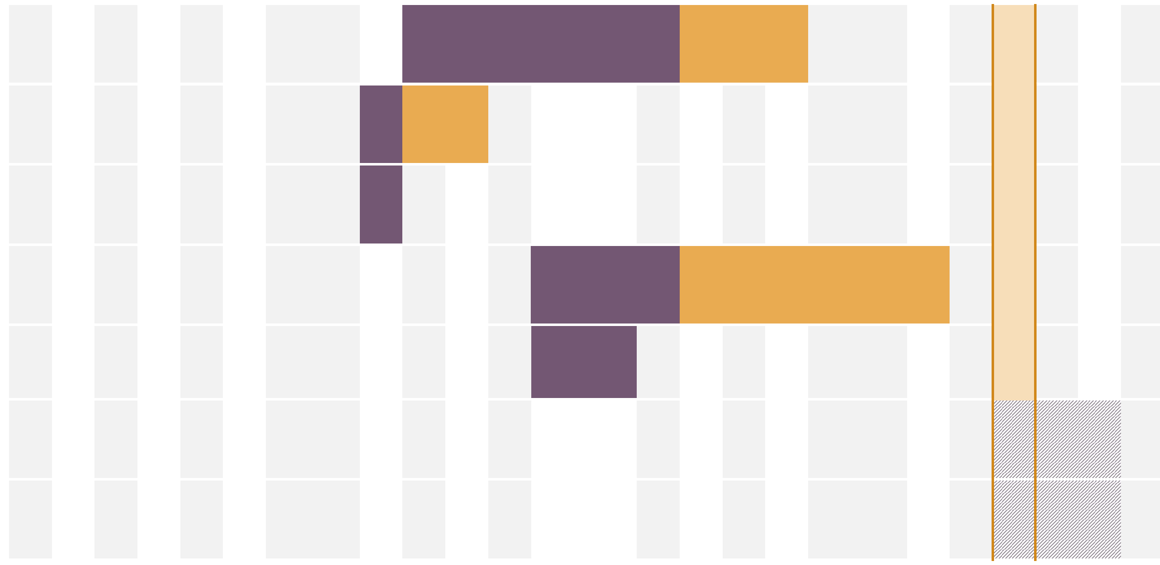
Marthe Elise Leirvåg, Nora Bjørvik Karlsen og Sofie Othilia Ngo

Period Highlight: 21

AKTIVITET	PLANLAGT START	PLANLAGT VARIGHET	FAKTISK START	FAKTISK VARIGHET	PROSENT FULLFØRT
1. Forprosjektrapport	4	1	4	1	100 %
2. Prosjektdagbok	4	16	4	17	100 %
3. Rapportskriving	5	15	5	16	100 %
3.1 Innledning og problemstilling	5	15	5	14	100 %
3.2 Beskrivelse av bygg	5	2	5	2	100 %
3.3 Metode	5	15	5	5	100 %
3.4 Laster	6	7	6	14	100 %
3.5 Dimensjonering	13	2	13	7	100 %
3.6 Drøfting	15	3	13	7	100 %
3.7 Konklusjon	18	2	20	1	100 %
3.8 Sammendrag	18	2	20	1	100 %
4. Utforming av oppgave	6	14	6	14	100 %
5. Beregninger	6	9	6	14	100 %
5.1 Laster	6	5	6	14	100 %
5.1.1 Snø	6	5	6	7	100 %
5.1.2 Vind	6	5	6	7	100 %
5.1.3 Seismisk	10	2	11	9	100 %



5.1.4 Egenlast	10	5	10	8	100 %
5.1.5 Jordtrykk	9	1	9	3	100 %
5.1.6 Nyttelast	9	1	9	1	100 %
5.2 Dimensjonering	13	2	13	7	100 %
6. Underveispresentasjon	13	1	13	1	100 %
7. Sluttpresentasjon	21	3			0 %
8. Videoinnlevering	21	3			0 %



VEDLEGG L

Timeliste

Timeliste				
Uke	**Dag**	**Marthe**	**Sofie**	**Nora**
4	Mandag	3	2	3
	Onsdag	2	2	2
	fredag	3	3	3
5	fredag	2	2	2
6	onsdag	2	2	2
	fredag	3,5		3,5
7	mandag	5		5
	onsdag	4	3	4
	fredag		0,5	0,5
	søndag	1		1
8	mandag	5		4,5
	tirsdag	3		3
	onsdag	4		4
	fredag	6		6
9	tirsdag	1		1
10	**eksamen**			
11	mandag	7		7
	tirsdag	7	7	7
	onsdag	7	7	7
	torsdag	4	4	4
	fredag	7	7	7
12	mandag	6		6
	tirsdag	8	7	7
	onsdag	7	7	6
	torsdag	7	7	7
	fredag	7	7	7
13	mandag	7	7	6
	tirsdag	7	7	7
	onsdag	7	7	7
	torsdag	5	5	4
	fredag	7	7	7
14	Mandag	5	5	5
	tirsdag	7	7	
	onsdag	8	8	8
	torsdag	7	7	7
	fredag	5	5	6
15	Mandag	7	7	7
	Tirsdag	6	6	6
	Onsdag	6,5	6,5	6,5
16	tirsdag	7	7	7
	onsdag	7	7	7
	torsdag	6	6	6
	fredag	6,5	6,5	6,5
17	mandag	7	4,5	6
	tirsdag	7	7	7
	onsdag	7	7	6,5
	torsdag	8	6	7,5
	fredag	8	8	8

18	mandag	8	8	8
	tirsdag	8	8	8
	onsdag	8	8	5
	torsdag	8	8	8
	fredag	7	7	7
	søndag	5	5	5
19	mandag	8	8	7,5
	tirsdag	8	8	8
	onsdag	8	8	6
	torsdag	8	7	8
	fredag	7	7	7
20	mandag	8	8	8
	tirsdag	8	8	8
	onsdag	8	8	8
	torsdag	11	11	11
	fredag	8	8	8
	lørdag	7,5	7,5	7,5
21	mandag	6	6	6
	tirsdag	10	10	10
	onsdag	8	8	8
	torsdag	9	9	9
	fredag	8	8	8
	lørdag	13,5	13,5	13,5
	søndag	14	14	14
SUM		461,5	412	444