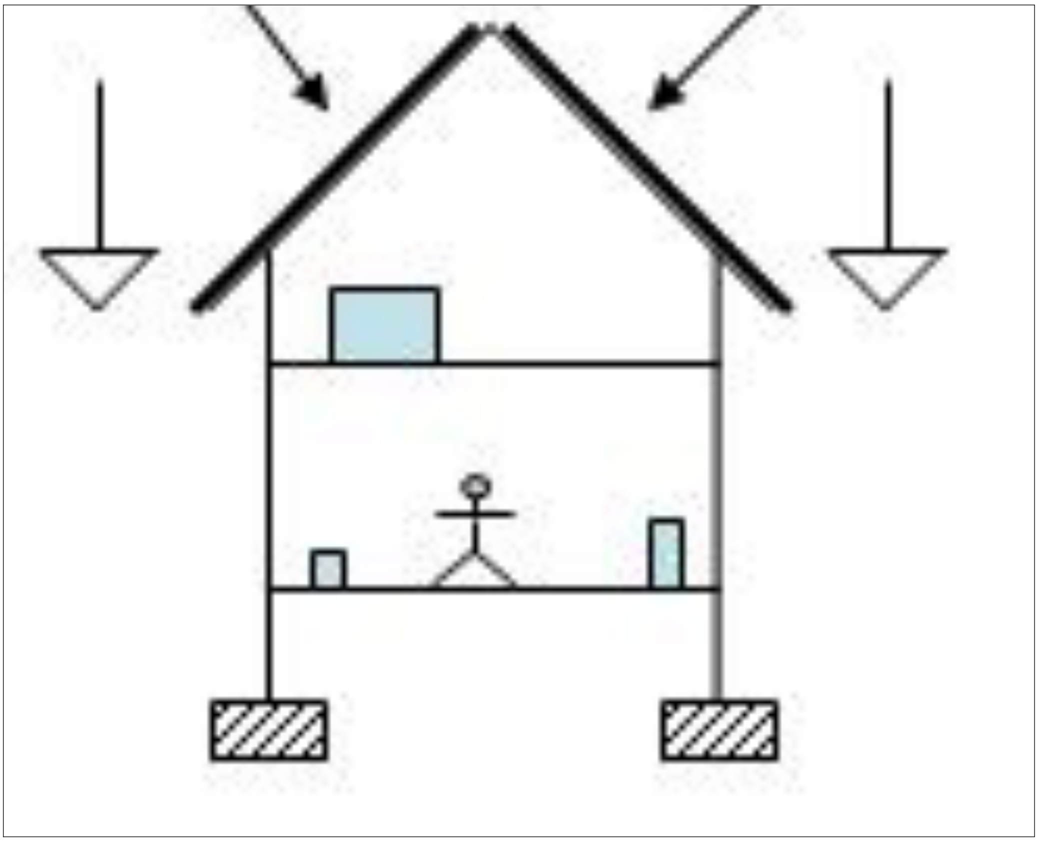
STATIC CALCULATIONS



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Class: Bkai0216
Semester: 2
Assignment detailing: This static calculation has been prepared for the Villa M house. I created calculations for the following: timber rafter, concrete storey partition and steel beam, foundation width.

INTRODUCTION

In our project we are using large amount of different beams. House has a steel beam for balcony, common rafters(timber) as roof construction and storey partition, and, lastly, we have glue laminated beam that holds parts of the roof. To know the necessary size to provide deflection that would be acceptable and safe.

The calculations will be done in 4 parts - timber rafter, concrete storey partition and steel beam, foundation width. For timber beam, since it is holding the roof in several places with different spans, only the largest span will be calculated as deflection will be the highest there.

Since we have chosen the beam size from the tables provided in Compendium in Scheme Design phase, we will have to reflect on how precisely the tables are for our case and see if we need to change the size in our drawings.

CALCULATION ASSUMPTIONS

STANDARDS

CONSEQUENCE CLASS:

Since this is a single family house, it is in consequence class M (Medium).
All other classes such as load duration class, service class and material class for each element will be defined under the Static Calculations section within the report.

LITERATURE

In order to affectively complete these calculations I used the following documentation for guidance:

- Jnc 106 kompendie Load and safety.pdf
- Jnc 231 Example of calculating a steel beam A2016.pdf
- Jnc 203 Compendium for Load bearing constructions 2. Semester Timber and steel beams rev 01.09.2015.pdf
- Jnc 223 Example of the sizing a timber beam A2016.pdf
- Jnc 224a Exercise no 1 Solution proposal part one A2016.pdf

MATERIALS

ROOF - Light roof construction			
Material	**Density kN/m³**	**Calculation**	**Total kN/m²**
Common Rafter	5 kN/m³	5kN/m³ x (0.095 x 0,295) / 0.800	0.175
Asphalt paper(both layers			0.10
Plywood	7 kN/m³	7kn/m³ x (0.026)	0.09
Mineral wool insulation 300mm	0.5 kN/m³	0.5kn/m³ x (0.300)	0.15
Plasterboard 2 x 13mm	9 kN/m³	9kn/m³ x (0.013) = 0.117 x 2	0.23
		Sum **Self weight**	**0.745**

Heaviest wall			
Material	**Density kN/m³**	**Calculation**	**Total kN/m²**
Lecarille block part	6 kN/m³	6 kN/m³ x 0.39	2.34
Cavity brick wall	-	-	4
Insulation 190mm	1 kN/m³	1 kN/m³ x 0.19	0.2

Other			
Material	**Density kN/m³**	**Calculation**	**Total kN/m²**
Floor slab	17.5	17.5 kN/m³ x 0.14 m	2.45
Ceiling and flooring			0.6
Steel beam(IPE120)			0.1*
Foundations	25 kN/m³	25 kn/m³ x 0.4	10
*Values are kN/m, because area load is irrelevant in these cases			

LOADS

SELF WEIGHT	
Element	**Total**
Roof	0.745 kN/m^2
Wall	2.94 kN/m^2
Floor slab	0.6 kN/m^2
Steel beam IPE120	0.1 kN/m
Foundations	10 kN/m

IMPOSED LOAD	
Element	**Total**
Floor slab	1.5 kN/m^2
Steel beam IPE120	1.5 kN/m^2

ENVIRONMENTAL ACTIONS - SNOW		
Element	**Snow load shape coefficient**	**Total**
Roof	-	0.8 kN/m^2

Table 5.2: Snow load shape coefficients taken from ‘jnc 106 kompendie load and safety pg. 23’.

Table 5.2: Snow load shape coefficients

Angle of pitch of roof α	$0° \leq \alpha \leq 30°$	$30° < \alpha < 60°$	$\alpha \geq 60°$
μ_1	0,8	0,8(60 - α)/30	0,0
μ_2	0,8 + 0,8 α/30	1,6	--

STATIC CALCULATIONS

RAFTER

Load duration class: K (as snow is a short term load)

Service class: 2 (Ventilated area)

Material class: C18

Common rafter dimension: 95 x 295

F_{md}: 12.0 MPa (bending)

F_{vd}: 2.27 MPa (Shear)

Fc 90_d: 1.47 MPa (compression)

E_o: C18 = 9000 (Rigidity factor for timber)

Line Load

Self weight - 0.745 kN/m2 x 0.8m = 0.6 kN/m

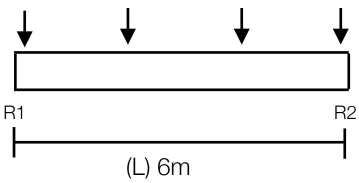
Snow load - 0.8 kN/m2 x 0.8m = 0.64 kN/m

Design Loads

Ed = γg x g + γs x s

Ed = 1.0 x 0.6 kN/m + 1.5 x 0.64 kN/m = 1.56 kN/m

Internal forces



(q) 1.56 kN/m

Mmax $= 1/8 \times q \times L^2$

$= 1/8 \times 1.56$ kN/m $\times 6^2$

= 7 kNm

Vd = R = 1/2 x q x L

= 1/2 x 1.56 x 6

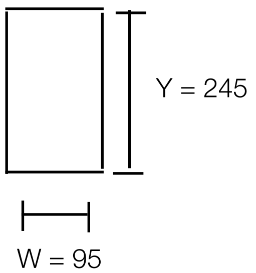
= 4.7 kN

Sectional modulus

Wmin = Mmax/$f_{m,d}$

$= 7 \times 10^6$ N/mm / 12 N/mm2 $= 583 \times 10^3$ N/mm^3

* Table 7.3.2.2 in 'jnc 203 Compendium for load bearing constructions 2' has been checked and the beam size 95 x 295 cannot be found within this table. Therefore the calculations created within this document is necessary as it is not possible to get a good understanding of the section properties of the timber beam from this table.



$Iy = 1/12 \times w \times h^3$

$= 1/12 \times (95 \times 295^3)$

$= 203 \times 10^6\ mm^4$

$Wy = 1/6 \times w \times h^2$

$= 1/6 \times 95 \times 295^2$

$= 1378 \times 10^3\ mm^3$

Wy >Wmin -> OK

Excepted deflection amount

L / 400

= 6000 / 400 = 15 mm

U inst: Selfweight

$5 \times q \times L^4 / 384 \times E \times I$

$= (5 \times 0.6 \times 6^4 \times 10^{12}) / (384 \times 9000 \times 203 \times 10^6)$

= 5.4 $<$ 15 (so it is well under the accepted deflection amount)

U inst: Snow load

$5 \times q \times L^4 / 384 \times E \times I$

$= (5 \times 0.64 \times 3850^4) / (384 \times 9000 \times 116 \times 10^6)$

= 5.64 $<$ 15 (so it is well under the accepted deflection amount)

Final Deflections

Ufin,G = Uinst,G (1 + kdef)

Ufin,G = 5.4 * (1+0.8) = 9.72 mm

Ufin,Q = 5.64 * (1+ 0) = 5.64 mm

Ufin = 9.72 + 5.64 = 15.36 mm

= 15.36 mm > 15mm (deflection is close enough to accept it, because it won't make a difference.

Bending stress

σb = M/W < fmd

σb = $(7 \times 10^6) / (950 \times 10^3)$

σb = 7.3 < 12 (fmd)

σb = 3.27 < 12 (so it is well under the maximum bending stress amount)

Shear stress

Rd = 1.5 x Vd/A < fvd

Rd = 1.5 x 4700 / 28025

Rd = 0.25 < 2.27

Shear stress calculation details:

4.7 has been changed from kN to N

28025 = 95 X 295 (Beam dimensions)

Rd = 0.25 < 2.27 (so it is well under the maximum shear stress amount)

Determining the min length of the bearing

Lmin = Vd / fc90d x W

Lmin = 4700 / 1.47 x 95

= 33.5mm (min amount to place on the wall plate)

Documentation

Strength classes : design values for softwoods in MPa												
Service class 1 and 2												
	C18							**C14**				
		P	L	M	K	Ø		P	L	M	K	Ø
Bending	$f_{m,d}$	8.0	9.3	10.7	12.0	14.7		6.2	7.3	8.3	9.3	11.4
Tension	$f_{t,0,d}$	4.9	5.7	6.5	7.3	9.0		3.6	4.1	4.7	5.3	6.5
	$f_{t,90,d}$	0.22	0.26	0.30	0.33	0.41		0.18	0.21	0.24	0.27	0.33
Compression	$f_{c,0d}$	8.0	9.3	10.7	12.0	14.7		7.1	8.3	9.49	10.67	13.4
	$f_{c,90,d}$	0.98	1.14	1.30	1.47	1.79		0.89	1.04	1.19	1.33	1.63
Shear	$f_{v,d}$	1.51	1.76	2.01	2.27	2.77		1.33	1.56	1.78	2.00	2.44

Table for Strength classes taken from 'jnc 203 Compendium for load bearing constructions 2' page 20.

Rigidity factor for timber

For calculating the instantaneous deflection u_{inst} we have to use the characteristic value of the regidity factor. See table below.

Strength clas		**C30**	**C24**	**C18**	**C14**	**GL32h**	**GL32c**	**GL28h**	**GL28c**	**GL24h**	**GL24c**
Elastic modulus	**E_0 MPa**	**12000**	**11000**	**9000**	**7000**	**14200**	**13500**	**12600**	**12500**	**11500**	**11000**

Table for rigidity factor for timber taken from 'jnc 203 Compendium for load bearing constructions 2' page 22.

Table 2.2 Single span rafters. Clear span L in meters

Heavy roof 0.55 kN/m^2											
Beam dimension		**Beam Distance from centre to centre**									
		0.4 m		0.6 m		0.8 m		1.0 m		1.2 m	
w	h	L_u	L_f	L_u	L_f	L_u	L_f	L_u	L_f	L_u	L_f
45	120	2.43	3.46	2.13	2.82	1.93	2.44	1.79	2.17	1.68	1.98
45	145	2.94	4.18	2.58	3.42	2.35	2.96	2.18	2.64	2.05	2.41
45	170	3.44	4.90	3.03	4.02	2.76	3.48	2.57	3.11	2.42	2.84
45	195	3.94	5.62	3.48	4.61	3.18	4.00	2.95	3.58	2.78	3.27
45	220	4.44	6.33	3.93	5.20	3.59	4.52	3.34	4.04	3.15	3.69
45	245	4.93	7.03	4.37	5.79	3.99	5.03	3.72	4.51	3.51	4.12
45	270	5.41	7.73	4.48	6.38	4.40	5.55	4.10	4.97	3.87	4.54
45	295	5.90	8.43	5.24	6.96	4.48	6.60	4.48	5.43	4.23	4.97
70	145	3.36	5.18	2.97	4.26	2.71	3.70	2.52	3.31	2.38	3.02
70	170	3.93	6.06	3.48	4.99	3.18	4.34	2.97	3.88	2.80	3.55
70	195	4.48	6.92	3.99	5.72	3.65	4.97	3.41	4.46	3.21	4.07
70	220	5.03	7.78	4.49	6.44	4.12	5.61	3.84	5.03	3.63	4.60
70	245	5.58	8.63	4.98	7.16	4.58	6.24	4.28	5.60	4.04	5.12
95	145	3.67	5.98	3.26	4.94	2.99	4.30	2.79	3.85	2.63	3.52
95	170	4.28	6.98	3.82	5.78	3.50	5.03	3.27	4.52	3.09	4.13
95	195	4.87	7.96	4.36	6.61	4.01	5.77	3.75	5.18	3.54	4.74
95	220	5.46	8.93	4.90	7.43	4.51	6.49	4.22	5.84	3.99	5.34
95	245	6.03	9.89	5.43	8.25	5.01	7.21	4.69	6.49	4.44	5.95
95	295	7.16	11.76	6.47	9.85	5.99	8.64	5.62	7.79	5.33	7.14

Table 2.2 single span rafters taken from 'jnc 205 Load bearing tables for use in scheme design' page 24.

Table 2.3. Factors r_u and r_f as the clear span L_u and L_F in Table 2.1 and 2.2 must be multiplied with, as a function of the slope of the beam.

β	$\geq 15^o$	20^o	25^o	30^o	35^o	40^o	45^o	50^o
r_u	1.00	0.97	0.94	0.89	0.84	0.79	0.72	0.66
r_f	1.00	0.98	0.95	0.92	0.92	0.91	0.89	0.86

Table 2.3 taken from 'jnc 205 Load bearing tables for use in scheme design' page 25.

Calculation:

800 mm distance between rafters

Lu 5.99 x 1 = 5.99m (which is what we need so it is fine)

Reflection:

Lf 8.64 x 1 = 8.64m (which is way beyond needed).

I feel that Lu is close enough to 6m not to make a difference, later in calculations we will see if it is or not.

Table A.3 - Construction materials-wood

Materials	Density γ [kN/m^3]
wood (see EN 338 for timber strength classes)	
timber strength class C14	3,5
timber strength class C16	3,7
timber strength class C18	3,8

Table A.3 taken from 'jnc 106 kompendie load and safety pg. 26'.

Reflection:

3.8kN/m^3 for wood was not used within these calculations as I followed the 5kn/m^3 as per usual standard and John's suggestion.

LIGHTWEIGHT FLOOR SLAB

Load duration class: P & M (Permanent and Medium Term loads
Service class: 1 (Moisture not added to the air)
Consequence class: CC2 medium
Span: 3.7m

Ultimate limit state(ULS):

$Sd = ɣg \times g + ɣq + Qk = 1{,}0 \times 0.6 + 1{,}5 \times 1.5 = 2.85\ kN/m^2$

Serviceability limit state(SLS):

$Sk = G + 0.5 \times Qk$

$Sk = 0.6\ kN/m^2 + 0.5 \times 1.5\ kN/m^2 = 1.35\ kN/m^2$

Fire limit state:

$Sk = G + w_1 * Qk$

$Sk = 0.6\ kN/m^2 + 0.3 \times 1.5\ kN/m^2 = 1.05\ kN/m^2$

Expan acoustics deck density 1750 kg/m² Load-bearing capacity

EXPAN Lyddæk rumvægt 1750 kg/m³ - bæreevnetabel 140 og 160 mm

Element type	Egen-last kN/m²	Vd max kN/m	Md max kNm/m	Vk R 60 A1 kN/m	Mk R 60 A1 kNm/m	1. Line Maximum load-bearing c 2. Line Load bearing capacity in 3. Line Maximum load-bearing c component, class R60 A1						
	Self weigh	Max. load-bearing capacity (design value) exclusive self weight of the slap				Clear span between the supports in mm						
						2800 3,8	3000 4,3	3200 4,9	3400 5,6	3600 6,2	3800 6,9	4000 7,7
140/30		45,23	17,64			**14,69**	**12,52**	**10,74**	**9,26**	**8,01**	**6,95**	**6,04**
						7,37	5,84	4,63	3,67	2,88	2,23	1,69
				60,67	*13,25*	*10,41*	*8,78*	*7,44*	*6,33*	*5,39*	*4,60*	*3,91*
140/31		45,60	20,30			**17,29**	**14,79**	**12,74**	**11,03**	**9,59**	**8,37**	**7,33**
						8,02	6,38	5,08	4,04	3,20	2,51	1,93
				45,50	*16,25*	*13,34*	*11,34*	*9,69*	*8,33*	*7,18*	*6,20*	*5,37*
140/32		47,13	26,50			**23,32**	**20,06**	**17,38**	**15,16**	**13,28**	**11,69**	**10,33**
						9,67	7,74	6,22	5,00	4,02	3,21	2,54
				60,67	*21,42*	*18,37*	*15,74*	*13,57*	*11,77*	*10,26*	*8,97*	*7,87*
140/33	2,50	49,52	36,35			**32,07**	**28,46**	**24,78**	**21,72**	**19,15**	**16,97**	**15,10**

Final comparison

As per EXPAN table in previous page, I chose to use 140/30 element, here is the comparison:

Ultimate limit state :

2.85 kN/m^2 < 6.95 kN/m^2 (very far from pushing the limits)

Serviceability limit state :

1.35 kN/m^2 < 2.23 kN/m^2 (similar situation here)

Fire limit state :

1.05 < 4.6 kN/m^2 (very safe here, too)

As all these values are very, very far from limits provided by Expan, we technically could use weaker slab but since this is the weakest they provide, I just have to use it. Other option would be searching for another company that makes thinner floor slabs but that would impact the price and availability because would have to import from other countries.

STEEL BEAM

Load duration class: P & M (Permanent and Medium Term loads

Service class: 3 (located outside)

Material class: S235

Inspection level: Normal

Cross section class: 1

E = 0,21*106 N/mm2 (page 30. compendium)

f_y = 235 N/mm2 (table 3.1 compendium)

y_{M0} = 1,1 (resistance of cross-section, 30. page of compendium)

IPE-profile

Load span = 0.5m

Span = 5.5m

Loads

Self-weight of wooden deck(density x loadspan x thickness):

5 kN/m^3 x 0.5 x 0.022 = 0.055 kN/m

Imposed load on flooring 1.5 kN/m^2 -> 0.5*1.5=0.75 kN/m

Design load

E_d=1.0 x 0.055 +1.5*0.75 = 1.18 kN/m

Load on each support

$Q_{d,max}$=0.5*1.18*5.5=3.25 kN

Moment

M_{max}=1.18*5.5^2/8=4.47 kNm

Acceptable U value

U_{acc}=5500/400=13.75 mm

Minimum section modulus

W_{min}=M_{max} y_m/f_y=4.47*1.1*10^6/235=21*10^3 mm^3

^ IPE120 profile has Wel value of 53 x 10^3 mm^3. I have to use Wel not Wpl due to Service class 3.

Design moment of elasticity

$M_{el,d}=W_{el}*f_y/\gamma_{M0}=53*10^3*235/1.1=11,3*10^6$ Nmm

Values from IPE table

$I_y=3.18*10^6 mm^4$

g=10.4 kg/m -> g=0.1 kN/m

Adjusting loads w/ self weight of the beam

$E_d=1.0*0.055 + 1.5*0.75+0.1=1.28$ kN/m

$Q_{d,max}=0.5*1.28*5.5=3.5$ kN

$M_{max}=1.28*5.5^2/8=4.84$ kNm

$W_{min}=M_{max}\ \gamma_m/f_y=4.84*1.1*10^6/235=23*10^3\ mm^3$

Self weight deflection

$u_{self}=5*0.1*5500^4/384*3.18*10^6*0.21*10^6=0.5*915*10^{12}/256*10^{12}=1.8$ mm

$u_{floor}=5*0.055*5500^4/384*3.18*10^6*0.21*10^6=251*10^{12}/256*10^{12}=1$ mm

$u_{imposed}=5*0.75*5500^4/384*3.18*10^6*0.21*10^6=3431*10^{12}/256*10^{12}=13$ mm

$u_{final}=1.8+1+13=15.8$ mm > Umax but imposed load is not going to be as high in real life(balcony) and deflection is acceptable since it wont affect anything else.

Bending stress

$\sigma=M_d/W_{el} < f_y/\gamma_{M0}$

$M_d/W_{el} = 4.8 \times 10^6/53 \times 10^3 = 90$

$f_y/\gamma_{M0} =213.6$

^ the difference between the two already says it all - not even close to the limit

Area of the beam, size taken from table

$A_v=A-2bt_f+(t_w+2r)t_f$

$A_v=1.32*10^3-2*64*6.3+(4.4+2*7)*6.3=1320 - 806 + 116 = 630\ mm^2$

Shear stress

$\tau_d=V_d*\sqrt{3}/A_v=4.84*10^3*1.73/630=13.3\ N/mm^2$

^value is less than 213.6 -> beam can withstand the shear stress.

Documentation

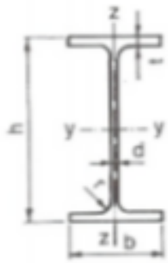
Table 3.1: Nominal values of yield strength f_y and ultimate tensile strength f_u for hot rolled structural steel

Standard and steel grade	Nominal thickness of the element t [mm]			
	$t \leq 40$ mm		$40 \text{ mm} < t \leq 80$ mm	
	f_y [N/mm^2]	f_u [N/mm^2]	f_y [N/mm^2]	f_u [N/mm^2]
EN 10025-2				
S 235	235	360	215	360
S 275	275	430	255	410
S 355	355	510	335	470
S 450	440	550	410	550

6.4.1.2 Tværsnitsklasse for valsede I-formede profiler

	Rent moment																							
profiltype	IPE						HEA						HEB						HEM					
stålstyrke	S235	S275	S355	S420	S450	S460	S235	S275	S355	S420	S450	S460	S235	S275	S355	S420	S450	S460	S235	S275	S355	S420	S450	S460
profil nr.																								
80	1	1	1	1	1	1																		
100	1	1	1	1	1	1	1	1	1	1	1	1	1	1	1	1	1	1	1	1	1	1	1	1
120	1	1	1	1	1	1	1	1	1	2	2	2	1	1	1	1	1	1	1	1	1	1	1	1
140	1	1	1	1	1	1	1	1	2	3	3	3	1	1	1	1	1	1	1	1	1	1	1	1
160	1	1	1	1	1	1	1	1	2	3	3	3	1	1	1	1	1	1	1	1	1	1	1	1
180	1	1	1	1	1	1	1	2	3	3	3	3	1	1	1	1	1	1	1	1	1	1	1	1
200	1	1	1	1	1	1	1	2	3	3	3	3	1	1	1	1	1	1	1	1	1	1	1	1
220	1	1	1	1	1	1	1	2	3	3	3	3	1	1	1	1	1	1	1	1	1	1	1	1
240	1	1	1	1	1	1	1	2	3	3	3	3	1	1	1	1	1	1	1	1	1	1	1	1
260							2	3	3	3	3	3	1	1	1	1	2	2	1	1	1	1	1	1
270	1	1	1	1	1	1																		
280							2	3	3	3	3	4	1	1	1	2	2	2	1	1	1	1	1	1
300	1	1	1	1	1	1	2	3	3	3	3	3	1	1	1	2	2	2	1	1	1	1	1	1
320							1	2	3	3	3	3	1	1	1	1	1	2	1	1	1	1	1	1
330	1	1	1	1	1	1																		

Efter Euronorm 19:1957
Normallængder 10, 12 og 14 m.



Tabelværdien skal multipliceres med de i tabellens hoved anførte faktorer.

profil nr.	h mm	b mm	d mm	t mm	r mm	A mm^2	u m^2/m	g kg/m	I_y mm^4	$W_{el,y}$ mm^3	i_y mm	I_z mm^4	$W_{el,z}$ mm^3	i_z mm	I_v mm^4	I_w mm^6	W_{pl} mm^3
faktor	1	1	1	1	1	10^3	1	1	10^6	10^3	1	10^6	10^3	1	10^3	10^9	10^3
80*	80	46	3,8	5,2	5	0,764	0,328	6,00	0,801	20,0	32,4	0,085	3,69	10,5	7,00	0,118	23,2
100*	100	55	4,1	5,7	7	1,03	0,400	8,10	1,71	34,2	40,7	0,159	5,79	12,4	12,1	0,351	39,4
120*	120	64	4,4	6,3	7	1,32	0,475	10,4	3,18	53,0	49,0	0,277	8,65	14,5	17,4	0,890	60,8
140*	140	73	4,7	6,9	7	1,64	0,551	12,9	5,41	77,3	57,4	0,449	12,3	16,5	24,5	1,98	88,4
160*	160	82	5,0	7,4	9	2,01	0,623	15,8	8,69	109	65,8	0,683	16,7	18,4	36,2	3,96	123,8
180*	180	91	5,3	8,0	9	2,39	0,698	18,8	13,2	146	74,2	1,01	22,2	20,5	48,0	7,43	166,4
200*	200	100	5,6	8,5	12	2,85	0,768	22,4	19,4	194	82,6	1,42	28,5	22,4	70,2	13,0	220

- From the compendium page 35 the shear area A_v for IPE section is
- $A_v = A - 2bt_f + (t_w + 2r)t_f$

FOUNDATION WIDTH

Self weight

Structure	Load span (m)	Area load (kN/m²)	Line load (kN/m)
Roof	3	0.745	2.235
Cavity brick wall incl. insulation	2.6	4.2	11
Lecarille blokke basement wall	2.6	2.34	6
Concrete floor slab	1.85	2.45	4.5
Ceiling and flooring	1.85	0.6	1.11
Foundation	0.9	10	9
		Total	**33.85**

Imposed loads

Load	Load span (m)	Area load (kN/m²)	Line load (kN/m)
Snow load on roof	3	0.8	2.4
Imposed load on concrete floor slab	1.85	1.5	2.78
		Total	**5.18**

Load combinations

Formulas are taken from material given last semester. It is the same, but I don't have gangway so I substitute q_{loft} with 0. This calculation is carried 2 times because we have only 2 loads that can be reduced - snowload and imposed load.

$$E_{d,1} = \gamma_g \cdot \Sigma G + \gamma_q \cdot S_k + \gamma_q \cdot \psi_0 \cdot q_{loft} + \gamma_q \cdot \psi_0 \cdot q_{storey}$$

Load combinations

$E_{d,1}$ = 1 x 33.85 + 1.5 x 2.4 x 0.3 + 1.5 x 2.78 = 39.1 kN/m

$E_{d,2}$ = 1 x 33.85 + 1.5 x 2.4 + 1.5 x 2.78 x 0.5 = 39.55 kN/m

Thus we use 39.55 kN/m to size our foundations.

$\sigma = E_{d,max}/w$ $\sigma = 39.55/0.4 = 98.9\ kN/m^2 < 200\ kN/m^2$
^ for school situation we are using approximate maximum tension of soil 200 kN/m²

CONCLUSION

Doing this calculation was to give me some actual numbers and calculations to back up the choices we made in scheme design when chose the sizes of floor slab, rafters, balcony beam and foundation width.

Rafters final deflection turned out to be 0.36 mm more than suggested maximum but 1/3 of a millimetre will not change the quality and it will not affect any other structures negatively. So I decided to keep the previously used size. It is the most convenient as well, because of the fact that we have 300mm insulation and with suspended ceiling and a little big higher roof it is perfect fit for us.

Floor slab we chose from the tables before was, as expected, too much. But since it would be more expensive to find another company from different country that could provide a thinner slab, I think this is still a good choice.

Steel beam for balcony I chose just based on my experience and turned out to be spot on. I used service class 3 since it is exposed to environment. But I feel that the service class safety is a bit too much, because the beam would be zinc coated and thus not rust and the temperature swings in Denmark are not that harsh to affect strength of the beam.

Foundation width was not chosen by any tables but it is the only thing that was fixed by the fact that we have 408mm cavity brick walls that demand at least 390mm wide foundation. But, just to make sure, I did the calculations and as expected - turned out to be very safe. Of course, to be completely sure land should be examined to know the exact maximum tension of the soil but it more than certainly will be in range of 150-250, so our foundation size is safe.

Overall these calculations approved of the sizes and material classes we have chosen in scheme design, which means nothing needs to be changed in our drawings regarding sizes. But doing this calculation, I understood that depending on whether it's timber beam, steel, concrete slab or something else, the formatting and the way to structure the calculations, is very different. This lead me to think that if I would like to make future-proof template, I would make for each type of calculations a different template and include all the tables and everything in them.