STRUCTURAL CALCULATIONS

Revised August 2018

Project:
Blake Kingsbury and Merrit Chesson
Summit Powder Mountain, Lot #70
8492 E. Spring Park, Utah

Project Number: 8332

Prepared For:
Scandinavian
6410 N. Business Park Loop Rd. Unit E
Park City, Utah 84098

Date:
June 2018

Prepared By:
Alex Hawkins, PE

Reviewed By:
David A. Jenkins, PE, SE

PLAN REVIEW ACCEPTANCE
FOR COMPLIANCE WITH THE APPLICABLE CONSTRUCTION CODES IDENTIFIED BELOW.
[X] BUILDING [X] STRUCTURAL
[X] MECHANICAL [X] PLUMBING
[X] ELECTRICAL [X] ENERGY
[ ] ACCESSIBILITY [ ] FIRE
PLAN REVIEW ACCEPTANCE OF DOCUMENTS DOES NOT AUTHORIZE CONSTRUCTION TO PROCEED IN VIOLATION OF ANY FEDERAL, STATE, OR LOCAL REGULATIONS.
BY: MEM DATE: 08/15/18
WEST COAST CODE CONSULTANTS, INC.

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08/07/2018
10367643
ALEX
HAWKINS
STATE OF UTAH

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Project:	Blake Kingsbury and Merrit Chesson	Project No.:	8332
By:	Alex Hawkins, PE	Checked By:	DAJ
Date:	June 2018		

GENERAL PROJECT INFORMATION

Project: **Blake Kingsbury and Merrit Chesson**

Project Address: **Summit Powder Mountain, Lot #70**
8492 E. Spring Park, Utah

Latitude:	41.380	North (Approximate)
Longitude:	-111.781	West (Approximate)
Elevation:	8570	ft

Client: **Scandinavian**

PROJECT DESCRIPTION

Provide structural calculations for Scandinavian Log Home

GENERAL DESIGN CRITERIA

Structure Type:	Structure Type
Design Code:	2015 IBC
Risk Category:	II

DESIGN LOADS

Dead Loads:

Roof DL:

Roofing:	6	psf
Insulation:	3	psf
Sheathing:	2.5	psf
Framing:	4	psf
MPE:	1.5	psf
Sprinklers:	1.5	psf
Miscellaneous:	1.5	psf
Total Roof DL:	**20**	**psf**

Floor DL:

Flooring:	3	psf
Sheathing:	2.5	psf
Framing:	10	psf
MPE:	1.5	psf
Sprinklers:	1.5	psf
Miscellaneous:	1.5	psf
Total Floor DL:	**20**	**psf**

Wall DL:

Exterior Walls:	**20**	**psf**
Interior Bearing Walls:	**15**	**psf**
Log Walls:	**30**	**psf**

Live Loads:

Roof Live:	**20**	**psf**
Floor Live:	**40**	**psf**
Main Floor Corridor / Stair:	**40**	**psf**
Corridors above Main Floor:	**40**	**psf**
Balconies:	**60**	**psf**

Snow Loads:

Ground Snow Load, p_g:	261	psf
Exposure Factor, C_e:	1.0	
Thermal Factor, C_t:	1.0	
Importance Factor, I_s:	1.0	
Roof Snow Load, p_f:	**183**	**psf**

Wind Loads:

Wind Speed:	115	mph - 3 second gust
Exposure:	C	

Seismic Loads:

S_S:	0.853	g
S_1:	0.285	g
Site Soil Class:	D	
Importance Factor, I_E:	1.00	

DEFLECTION LIMITS:

		Total Load	Live Load
Roof:	L /	240	360
Floor:	L /	360	480
Horizontal:	L /	--	240

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FOUNDATION CRITERIA & SPECIFICATIONS

Soils Report:	Company:	Geostrata	
	Date:	July 11, 2018	
	Report / Project Number:	594-004	
	Contact:		
	Allowable Bearing Pressure:	1500	psf
	Passive Pressure:	1074	psf
	Active Pressure:	31	psf
	At Rest Pressure:	53	psf
	Coefficient of Friction, μ:	0.35	
Foundation Type:			
	Footing Type:	Concrete Spread Footing	
	Min. Depth to Frost:	30	in

MATERIAL SPECIFICATIONS

CONCRETE & REINFORCING STEEL SPECIFICATIONS:

Concrete Strength, f'_c:			
	Footings / Foundation Walls:	3,000	psi
	Grade Beams:	4,000	psi
	Slab on Grade:	4,000	psi
	Bearing/Shear Walls:	4,000	psi

Deformed Reinforcing Bars: ASTM A615 Grade 60
ASTM A706 Grade 60 Weldable Rebar is to be used where welds are specified on contract documents

Welded Wire Fabric: ASTM A185 - Flat sheets, not rolls

STEEL FRAMING SPECIFICATIONS

Structural Steel:	W-Shape:	ASTM A992, F_y = 50 ksi
	Tubing:	ASTM A500, Grade B, F_y = 46 ksi
	Channels, Plates and Angles:	ASTM A36, F_y = 36 ksi
	Pipe:	ASTM A53, Grade B, F_y = 35 ksi
	Machine Bolts:	ASTM A307
	High-strength Bolts:	ASTM A325 or A490

Welds: E70XX Electrodes, Comply with AWS D1.1

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WOOD FRAMING SPECIFICATIONS

Unless noted otherwise, the following species and grades of lumber shall be used.

Sawn Lumber:	Species:	Douglas Fir-Larch (North)
	2x4 studs up to 8'-0" long:	Stud Grade
	2x4 studs over 8'-0" long:	Grade #2
	Other studs:	Grade #2
	Posts:	Grade #1
	Joists:	Grade #2
	Beams:	Grade #2
	Headers:	Grade #2
	Subpurlins:	Grade #2
	Purlins:	Grade #2
Glue Laminated Beams:	Species:	Douglas Fir-Larch (North)
	Simple Spans:	24F-V4
	Continuous Spans:	24F-V8
Sheathing:		APA Rated OSB
Framing Hardware:		Simpson Strong-Tie Connectors
Structural Nails:		Common Wire Type (unless noted otherwise)
Bolts in Wood:		ASTM A307

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SNOW DRIFT ANALYSIS
Drift 1 - Roof

CHAPTER 7, ASCE 7-10 — *IBC 2015 / ASCE 7-10*

Design Parameters

Terrain Category	C
Roof Exposure	Partially Exposed
Thermal Conditions	All other structures
Snow Drift Analysis Required?	No

Ground Snow Load, p_g (psf)	261	*Utah Snow Load Study*	Snow Density, γ (pcf)	30.0	*Equation 7.7-1*
Exposure Factor, C_e	1.0	*Table 7-2*	Balance Snow Load Height, h_b (ft)	6.10	p_f / γ
Thermal Factor, C_t	1.0	*Table 7-3*	Adjacent Roof Height, h_r (ft)	6.5	
Importance Factor, I_s	1.0	*Table 1.5-2*	Length of Upper Roof, L_u (ft)	14	
Roof Snow Load, p_f (psf)	183	*Equation 7.3-1*	Length of Lower Roof, L_L (ft)	25	

Snow Drift Analysis

Windward Drift Height, $h_{d,wind}$ (ft)	2.70	*Figure 7-9*
Leeward Drift Height, $h_{d,lee}$ (ft)	3.24	*Figure 7-9*
h_c (ft)	0.40	$h_r - h_b$
Design Drift Height, h_d (ft)	**0.40**	*Section 7.7.1*
Design Drift Width, w (ft)	**3.24**	*Section 7.7.1*
Maximum Drift Surcharge Load, p_d (psf)	**12.13**	*Section 7.7.1*

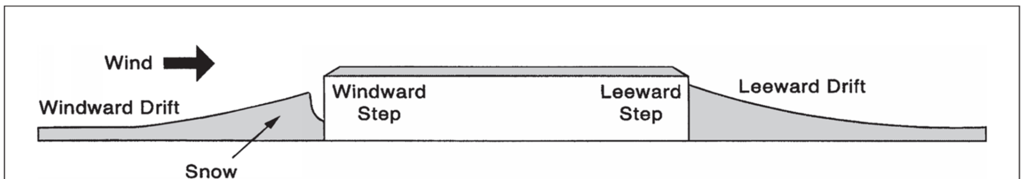

Figure 7-7 Drifts Formed at Windward and Leeward Steps

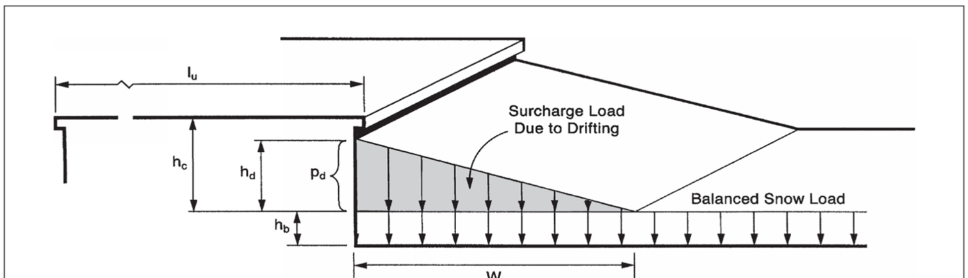

Figure 7-8 Configuration of Snow Drifts on Lower Roofs

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SNOW DRIFT ANALYSIS
Drift 1 - Family Room

CHAPTER 7, ASCE 7-10 — *IBC 2015 / ASCE 7-10*

Design Parameters

Terrain Category	C
Roof Exposure	Partially Exposed
Thermal Conditions	All other structures
Snow Drift Analysis Required?	Yes

Ground Snow Load, p_g (psf)	261	*Utah Snow Load Study*	Snow Density, γ (pcf)	30.0	*Equation 7.7-1*
Exposure Factor, C_e	1.0	*Table 7-2*	Balance Snow Load Height, h_b (ft)	6.10	p_f / γ
Thermal Factor, C_t	1.0	*Table 7-3*	Adjacent Roof Height, h_r (ft)	10.25	
Importance Factor, I_s	1.0	*Table 1.5-2*	Length of Upper Roof, L_u (ft)	40.5	
Roof Snow Load, p_f (psf)	183	*Equation 7.3-1*	Length of Lower Roof, L_L (ft)	11.5	

Snow Drift Analysis

Windward Drift Height, $h_{d,wind}$ (ft)	2.43	*Figure 7-9*
Leeward Drift Height, $h_{d,lee}$ (ft)	4.49	*Figure 7-9*
h_c (ft)	4.15	$h_r - h_b$
Design Drift Height, h_d (ft)	**4.15**	*Section 7.7.1*
Design Drift Width, w (ft)	**19.43**	*Section 7.7.1*
Maximum Drift Surcharge Load, p_d (psf)	**124.63**	*Section 7.7.1*

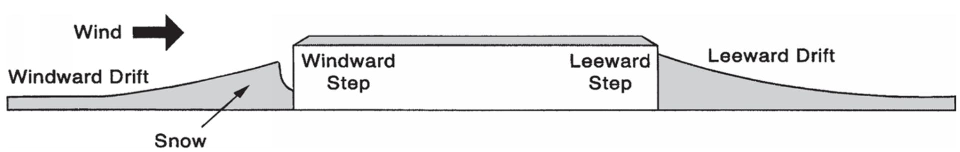


Figure 7-7 Drifts Formed at Windward and Leeward Steps

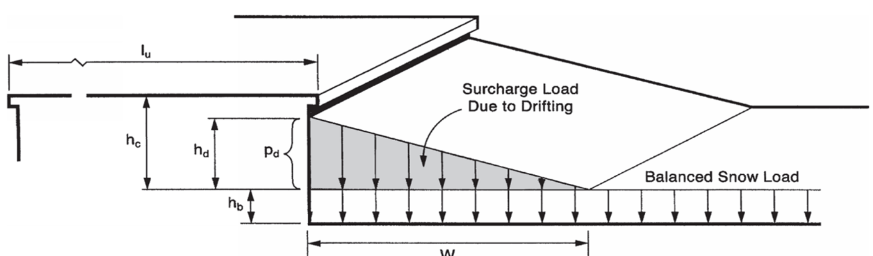


Figure 7-8 Configuration of Snow Drifts on Lower Roofs

USGS Design Maps Summary Report

User–Specified Input

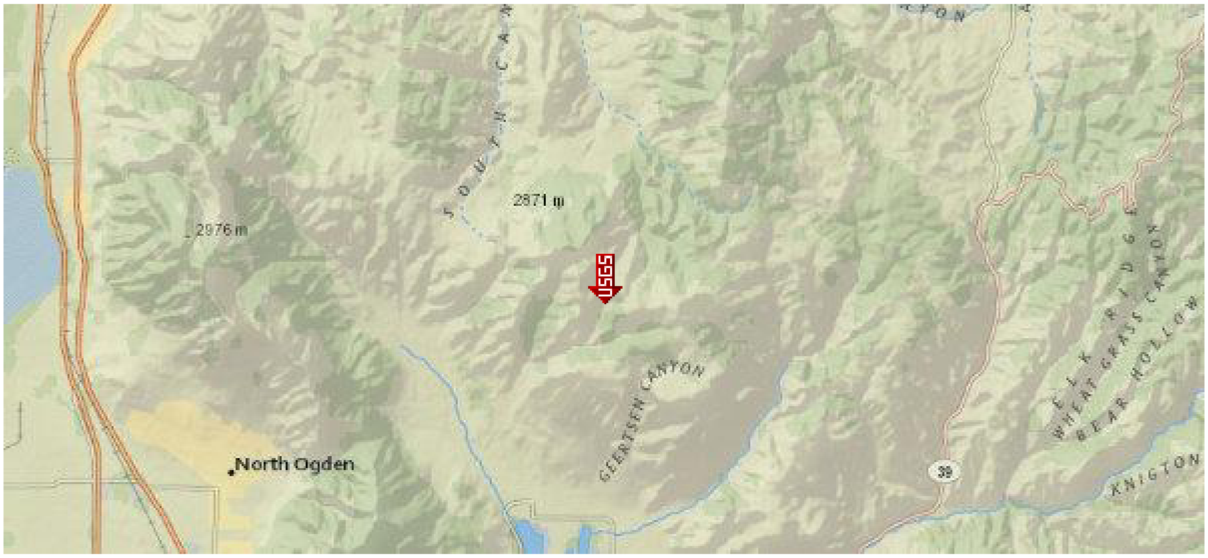
Report Title Powder Mountain
Tue June 5, 2018 23:24:18 UTC

Building Code Reference Document 2012/2015 International Building Code
(which utilizes USGS hazard data available in 2008)

Site Coordinates 41.38004°N, 111.78098°W

Site Soil Classification Site Class D – "Stiff Soil"

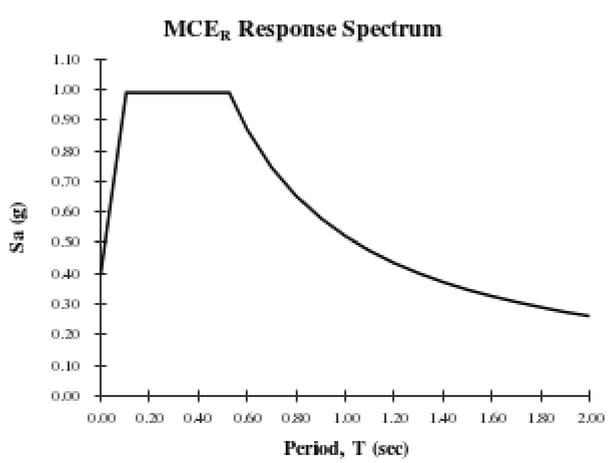
Risk Category I/II/III

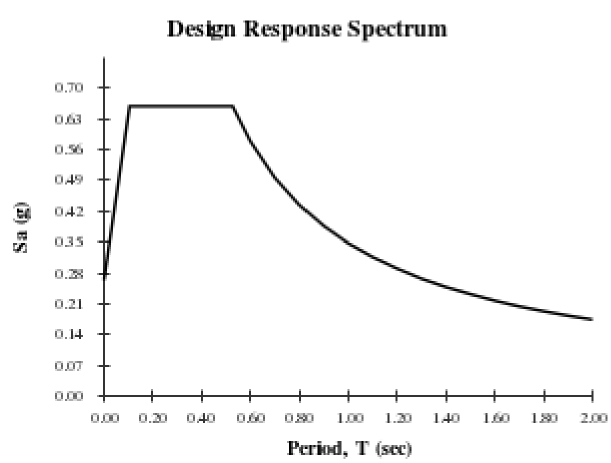


USGS–Provided Output

S_S =	0.853 g	S_{MS} =	0.989 g	S_{DS} =	0.659 g
S_1 =	0.285 g	S_{M1} =	0.521 g	S_{D1} =	0.347 g

For information on how the SS and S1 values above have been calculated from probabilistic (risk-targeted) and deterministic ground motions in the direction of maximum horizontal response, please return to the application and select the "2009 NEHRP" building code reference document.





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SEISMIC FORCE ANALYSIS - EQUIVALENT LATERAL FORCE PROCEDURE

CHAPTER 12 ASCE 7-10 — *IBC 2015 / ASCE 7-10*

Design Parameters

Risk Category	II	*Table 1604.5*	T_0 (sec)	0.106	*Section 11.4.5*
Building Height, h_n (ft)	29		T_S (sec)	0.528	*Section 11.4.5*
S_S (g)	0.853	*USGS*	T_L (sec)	8	*Section 11.4.5*
S_1 (g)	0.285	*USGS*	S_a (g)	N/A	*if $T<T_0$ (Equation 11.4-5)*
Site Class	D	*Geotech Report*	S_a (g)	0.659	*$T_0<T<T_S$ (Section 11.4.5-2)*
F_a	1.16	*Table 1613.3.3(1)*	S_a (g)	N/A	*$T_S<T<T_L$ (Equation 11.4-6)*
F_v	1.83	*Table 1613.3.3(2)*	C_t	0.02	*Table 12.8-2*
S_{MS} (g)	0.988	*$F_a S_S$*	x	0.75	*Table 12.8-2*
S_{M1} (g)	0.522	*$F_v S_1$*	T_a (sec)	0.250	*Equation 12.8-7*
S_{DS} (g)	0.659	*2/3(S_{MS})*	Response Modification Factor, R	2.5	*Table 12.2-1*
S_{D1} (g)	0.348	*2/3(S_{M1})*	Overstrength Factor, Ω_0	2.5	*Table 12.2-1*
Seismic Design Category	D	*Table 1613.3.5(1,2)*	$C_{S\ MAX}$	0.556	*Equation 12.8-3*
Importance Factor, I_E	1.00	*Table 1.5-2*	$C_{S\ MIN}$	0.029	*Equation 12.8-4*
Structure Type	All other structural systems		C_S	0.264	*Section 12.8.1.1*

Global Analysis

Component	Unit Weight (psf)	Area (ft^2)	Weight, w_i (kips)	Elevation, h_i (ft)	$w_i h_i^k$ (kip-ft)	F_i (kips)	$0.7F_i$ (kips)
Roof Level:			-		-	-	-
Roof	20	362	7.24	29	209.96	3.04	2.13
Walls	30	344	10.32	29	299.28	4.34	3.03
Snow	36.57	362	13.24	29	383.95	5.56	3.89
Roof	20	660	13.20	21	277.20	4.02	2.81
Walls	30	969	29.07	21	610.47	8.84	6.19
Hot Tub	100	88	8.80	21	184.80	2.68	1.87
Floor	20	362	7.24	21	152.04	2.20	1.54
Snow	36.57	660	24.14	21	506.91	7.34	5.14
Walls	30	1400	42.00	11	441.00	6.39	4.47
Floor	20	1145	22.90	11	240.45	3.48	2.44
Snow	36.57	229	8.38	11	87.94	1.27	0.89
			-		-	-	-
			-		-	-	-
			-		-	-	-
			-		-	-	-
			-		-	-	-
			-		-	-	-
			-		-	-	-
			-		-	-	-
			-		-	-	-
			-		-	-	-
			-		-	-	-
		Σw_i	187	$\Sigma w_i h_i^k$	3,394	V_x (kips)	49.17
Notes:				k	1	$0.7V_x$ (kips)	34.42

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SEISMIC FORCE ANALYSIS - DIAPHRAGM FORCES

CHAPTER 12 ASCE 7-10 *IBC 2015 / ASCE 7-10*

Design Parameters

Risk Category	II	*Table 1604.5*	S_{DS} (g)	0.659	*2/3(S_{MS})*
S_S (g)	0.853	*USGS*	S_{D1} (g)	0.348	*2/3(S_{M1})*
S_1 (g)	0.285	*USGS*	Seismic Design Category	D	*Table 1613.3.5(1,2)*
Site Class	D	*Geotech Report*	Importance Factor, I_E	1.00	*Table 1.5-2*

Diaphragm Design Forces

Level	F_i (k)	Sum F_i (k)	w_{px} (k)	Sum w_i (k)	F_{px} (k) *Eq. 12.10-1*	$F_{px,min}$ (k) *Eq. 12.10-2*	$F_{px,max}$ (k) *Eq. 12.10-3*	**$F_{px,design}$ (k)**	Scale Factor F_{px} / F_x
Roof	12.94	12.9	30.80	30.8	12.9	4.1	8.1	**8.1**	1.00
Rooftop Balcony	25.08	38.0	82.45	113.2	27.7	10.9	21.7	**21.7**	1.00
Upper	11.14	49.2	73.28	186.5	19.3	9.7	19.3	**19.3**	1.73
		-		-	-	-	-	-	-
		-		-	-	-	-	-	-
		-		-	-	-	-	-	-
		-		-	-	-	-	-	-
		-		-	-	-	-	-	-
		-		-	-	-	-	-	-
		-		-	-	-	-	-	-

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WIND FORCE ANALYSIS - DIRECTIONAL PROCEDURE

CHAPTER 27 (PART 1) ASCE 7-10 — *IBC 2015 / ASCE 7-10*

Design Parameters

Wind Speed, V (mph)	115	*Section 26.5*	Exposure Coefficient, K_h	1.024	*Table 27.3-1*
Exposure Category	C	*Section 26.7*	K_{zt} Applicable?	No	
Enclosure Classification	Enclosed		Height of Hill or Ridge, H (ft)	0	*Table 26.8-1*
Positive / Negative?	Positive		L_h (ft)	0	*Table 26.8-1*
nt. Pressure Coefficient, GC_{pi}	0.18	*Table 26.11-1*	H / L_h	0.00	
Mean Roof Height, h (ft)	36.5		x (ft)	0	*Table 26.8-1*
Building Length, L (ft)	48		Horizontal Attenuation, μ	0	*Table 26.8-1*
Building Width, B (ft)	24		Height Attenuation, γ	0	*Table 26.8-1*
L/B	2.00		K_1 / (H / L_h)	0	*Table 26.8-1*
h/L	0.76		K_1	0.00	*Table 26.8-1*
Roof Pitch	3.75	/12	K_2	0.00	*Table 26.8-1*
Roof Angle, θ	17.4		K_3	0.00	*Table 26.8-1*
Gust Effect Factor, G	0.85	*Section 26.9*	Topographic Factor, K_{zt} at h	1.00	*Section 26.8*
Terrain Constant, α	9.5	*Table 26.9-1*	Wind Directionality Factor, K_d	0.85	*Section 26.6*
Terrain Constant, z_g (ft)	900	*Table 26.9-1*	**Velocity Pressure, q_h (psf)**	**29.46**	*Equation 30.3-1*

MWFRS Wind Pressure Analysis

				Pressure Coefficients, C_p	*Wall*			*Parapet*	
					Windward	Leeward	Side	Windward	Leeward
					0.80	-0.50	-0.70	1.50	-1.00
Surface Mark	Surface Type	z (ft)	K_z	q_z (psf)	Wind Pressure, p (psf)				
1	Roof	40	1.044	30.0	-	-	-	-	-
2	Wall	38	1.032	29.7	14.90	-17.82	-22.83	-	-
3	Wall	32	0.996	28.7	14.18	-17.82	-22.83	-	-
4	Wall	21	0.911	26.2	12.53	-17.82	-22.83	-	-
5	Wall	10.5	0.849	24.4	11.31	-17.82	-22.83	-	-
6			-	-	-	-	-	-	-

Roof Type	Monoslope	*Roof*							
	Pressure Coefficients, C_p	Normal to Ridge for θ ≥ 10°			Parallel to Ridge for all θ				Windward Overhang
		Windward In	Windward Out	Leeward	0 to h/2	h/2 to h	h to 2h	> 2h	
		-0.18	-0.85	-0.60	-1.10	-0.70	-0.70	-0.70	0.80
Surface Mark	Surface Type	Wind Pressure, p (psf)							
1	Roof	-9.81	-26.59	-20.33	-32.85	-22.83	-22.83	-22.83	20.03
2	Wall	-	-	-	-	-	-	-	-
3	Wall	-	-	-	-	-	-	-	-
4	Wall	-	-	-	-	-	-	-	-
5	Wall	-	-	-	-	-	-	-	-
6		-	-	-	-	-	-	-	-

Diaphragm Forces

North-South, Positive Internal Pressure

Surface Mark	Surface Type	Projected Horizontal Pressure, p (psf)	Tributary Height (ft)	Unit Force (plf)	Diaphragm Width, W (ft)	Force (kips)
1	Roof	8.00	5	40.0	24	1.0
2	Wall	32.72	3	98.2	24	2.4
3	Wall	32.00	8	256.0	24	6.1
4	Wall	30.35	10.667	323.8	24	7.8
5	Wall	29.13	10.5	305.9	24	7.3
6		-		-		-
					Total Force (kips)	**24.6**

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WIND FORCE ANALYSIS - DIRECTIONAL PROCEDURE

CHAPTER 27 (PART 1) ASCE 7-10 — *IBC 2015 / ASCE 7-10*

Design Parameters

Wind Speed, V (mph)	115	*Section 26.5*	Exposure Coefficient, K_h	1.024	*Table 27.3-1*
Exposure Category	C	*Section 26.7*	K_{zt} Applicable?	No	
Enclosure Classification	Enclosed		Height of Hill or Ridge, H (ft)	0	*Table 26.8-1*
Positive / Negative?	Positive		L_h (ft)	0	*Table 26.8-1*
nt. Pressure Coefficient, GC_{pi}	0.18	*Table 26.11-1*	H / L_h	0.00	
Mean Roof Height, h (ft)	36.5		x (ft)	0	*Table 26.8-1*
Building Length, L (ft)	24		Horizontal Attenuation, μ	0	*Table 26.8-1*
Building Width, B (ft)	48		Height Attenuation, γ	0	*Table 26.8-1*
L/B	0.50		$K_1 / (H / L_h)$	0	*Table 26.8-1*
h/L	1.52		K_1	0.00	*Table 26.8-1*
Roof Pitch	3.75	/12	K_2	0.00	*Table 26.8-1*
Roof Angle, θ	17.4		K_3	0.00	*Table 26.8-1*
Gust Effect Factor, G	0.85	*Section 26.9*	Topographic Factor, K_{zt} at h	1.00	*Section 26.8*
Terrain Constant, α	9.5	*Table 26.9-1*	Wind Directionality Factor, K_d	0.85	*Section 26.6*
Terrain Constant, z_g (ft)	900	*Table 26.9-1*	**Velocity Pressure, q_h (psf)**	**29.46**	*Equation 30.3-1*

MWFRS Wind Pressure Analysis

				Pressure Coefficients, C_p	*Wall*			*Parapet*	
					Windward	Leeward	Side	Windward	Leeward
					0.80	-0.50	-0.70	1.50	-1.00
Surface Mark	Surface Type	z (ft)	K_z	q_z (psf)	Wind Pressure, p (psf)				
1	Wall	5.5	0.849	24.4	11.31	-17.82	-22.83	-	-
2	Wall	10	0.849	24.4	11.31	-17.82	-22.83	-	-
3	Wall	10.667	0.849	24.4	11.31	-17.82	-22.83	-	-
4	Wall	10.5	0.849	24.4	11.31	-17.82	-22.83	-	-
5			-	-	-	-	-	-	-
6			-	-	-	-	-	-	-

Roof Type	Monosloped	*Roof*							
	Pressure Coefficients, C_p	Normal to Ridge for θ ≥ 10°			Parallel to Ridge for all θ				Windward Overhang
		Windward In	Windward Out	Leeward	0 to h/2	h/2 to h	h to 2h	> 2h	
		-0.18	-0.70	-0.30	-0.90	-0.90	-0.50	-0.30	0.80
Surface Mark	Surface Type	Wind Pressure, p (psf)							
1	Wall	-	-	-	-	-	-	-	-
2	Wall	-	-	-	-	-	-	-	-
3	Wall	-	-	-	-	-	-	-	-
4	Wall	-	-	-	-	-	-	-	-
5		-	-	-	-	-	-	-	-
6		-	-	-	-	-	-	-	-

Diaphragm Forces

East-West, Positive Internal Pressure

Surface Mark	Surface Type	Projected Horizontal Pressure, p (psf)	Tributary Height (ft)	Unit Force (plf)	Diaphragm Width, W (ft)	Force (kips)
1	Wall	29.13	6	174.8	18.5	3.2
2	Wall	29.13	10	291.3	36	10.5
3	Wall	29.13	10.667	310.7	42	16.1
4	Wall	29.13	10.5	305.9	42	12.8
5		-		-		-
6		-		-		-
					Total Force (kips)	**42.6**

ENSIGN THE STANDARD IN ENGINEERING	Project: Blake Kingsbury and Merrit Chesson	Project No.: 8332
	By: Alex Hawkins, PE	Checked By: DAJ
	Date: June 2018	

WIND FORCE ANALYSIS - COMPONENTS & CLADDING

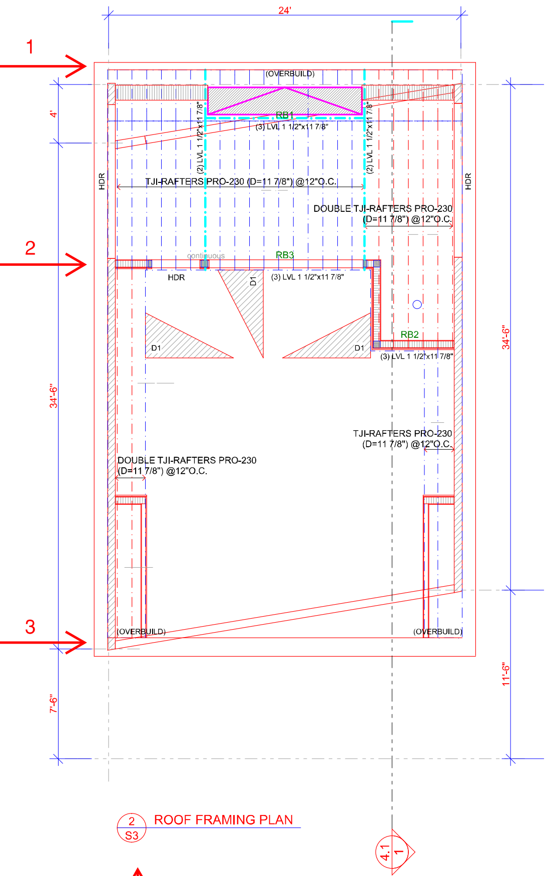
CHAPTER 30 ASCE 7-10 *IBC 2015 / ASCE 7-10*

Design Parameters

Wind Speed, V (mph)	115	*Section 26.5*	L_h (ft)	0	*Table 26.8-1*
Exposure Category	C	*Section 26.7*	H / L_h	0.00	
nt. Pressure Coefficient, GC_{pi}	0.18	*Table 26.11-1*	x (ft)	0	*Table 26.8-1*
Mean Roof Height, h (ft)	36.5		Horizontal Attenuation, μ	0	*Table 26.8-1*
Roof Pitch	3.75	/12	Height Attenuation, γ	0	*Table 26.8-1*
Roof Angle, θ	17.4		$K_1 / (H / L_h)$	0	*Table 26.8-1*
Gust Effect Factor, G	0.85	*Section 26.9*	K_1	0.00	*Table 26.8-1*
Terrain Constant, α	9.5	*Table 26.9-1*	K_2	0.00	*Table 26.8-1*
Terrain Constant, z_g (ft)	900	*Table 26.9-1*	K_3	0.00	*Table 26.8-1*
Exposure Coefficient, K_h	1.024	*Table 30.3-1*	Topographic Factor, K_{zt} at h	1.00	*Section 26.8*
K_{zt} Applicable?	No		Wind Directionality Factor, K_d	0.85	*Section 26.6*
Height of Hill or Ridge, H (ft)	0	*Table 26.8-1*	**Velocity Pressure, q_h (psf)**	**29.46**	*Equation 30.3-1*

Design Wind Pressure

Location			Tributary Area (ft^2) < 10	20	50	100	>500
Walls	Within 5 ft of building corner		-46.5	-43.6	-39.2	-36.2	-28.9
	All other areas		-37.7	-36.2	-34.8	-32.6	-28.9
	Positive Pressure		34.8	33.3	31.8	30.3	25.9
Roof	Within 5 ft of building corner		-87.8	-73.1	-52.4	-37.7	-37.7
	Within 5 ft of building edge		-58.3	-52.4	-43.6	-37.7	-37.7
	All other areas		-34.8	-34.0	-33.3	-31.8	-31.8
Parapet	Within 5 ft of building corner	A	111.9	95.7	73.6	57.4	53.0
		B	-70.7	-66.3	-60.4	-56.0	-44.2
	All other areas	A	82.5	75.1	64.8	57.4	53.0
		B	-61.9	-58.9	-56.0	-52.3	-44.2



2 S3 ROOF FRAMING PLAN

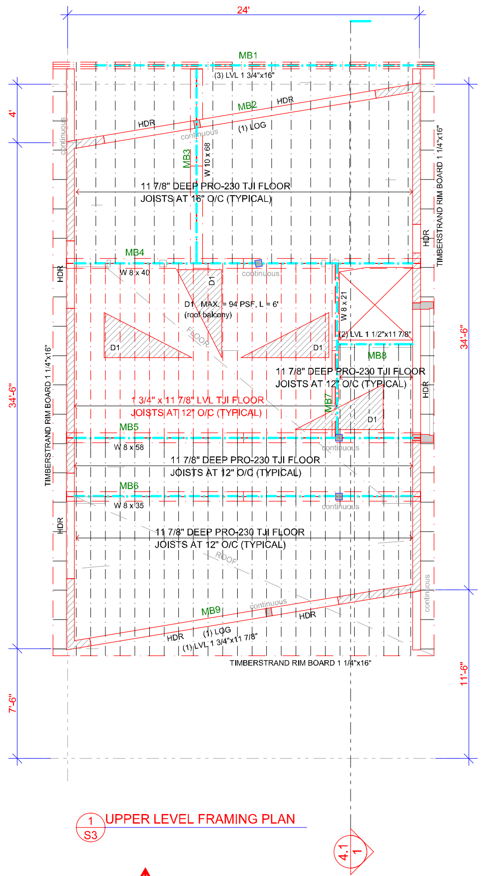
0.6 * W = 2.04 k
0.7 * EQ = 9.06 k

0.6 * W = 1.92 lbs
0.7 * EQ = 9.06 lbs

1 0.6 * W = 960 lbs
0.7 * EQ = 4530 lbs

2 0.6 * W = 960 lbs
0.7 * EQ = 4530 lbs

3 0.6 * W = 0 lbs
0.7 * EQ = 0 lbs



1 S3 UPPER LEVEL FRAMING PLAN

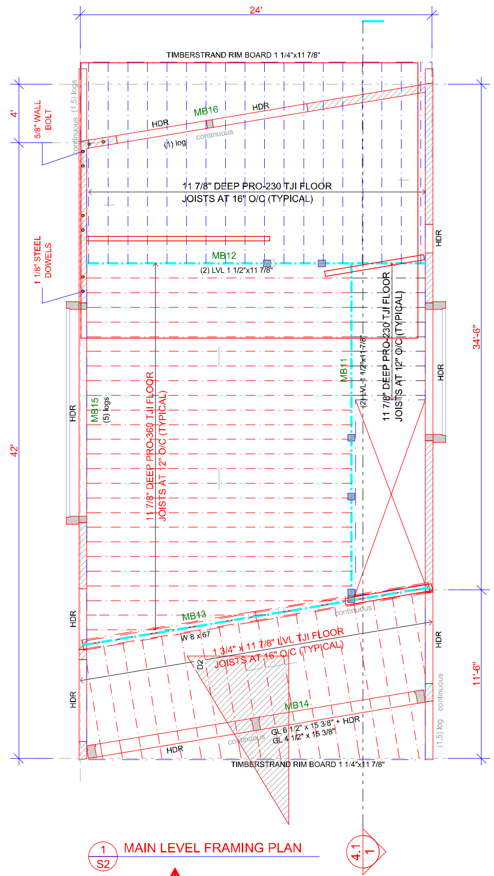
0.6 * W = 5.7 k
0.7 * EQ = 26.61 k

0.6 * W = 8.22 k
0.7 * EQ = 26.61 k

1 0.6 * W = 2010 lbs
0.7 * EQ = 7455 lbs

2 0.6 * W = 4110 lbs
0.7 * EQ = 13307 lbs

3 0.6 * W = 2100 lbs
0.7 * EQ = 5852 lbs



1 S2 MAIN LEVEL FRAMING PLAN

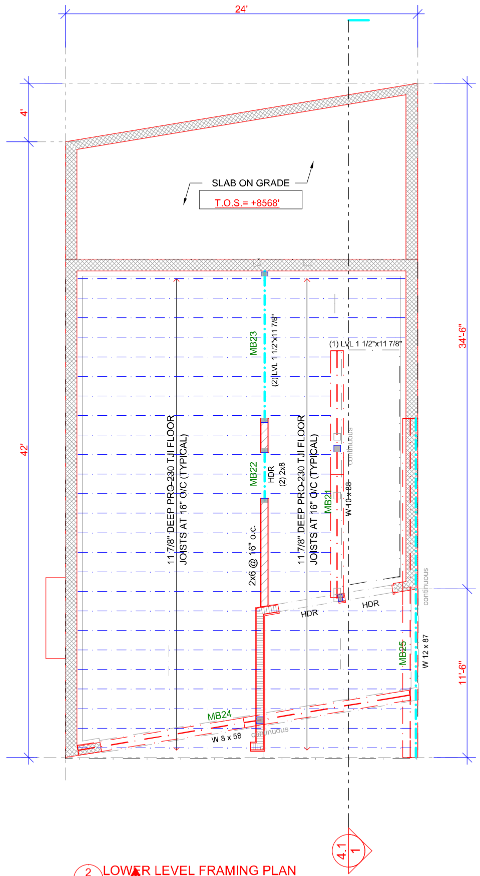
0.6 * W = 10.32 k
0.7 * EQ = 34.42 k

0.6 * W = 17.88 k
0.7 * EQ = 34.42 k

1 0.6 * W = 3459 lbs
0.7 * EQ = 8625 lbs

2 0.6 * W = 8940 lbs
0.7 * EQ = 17208 lbs

3 0.6 * W = 5481 lbs
0.7 * EQ = 8583 lbs



2 S1 LOWER LEVEL FRAMING PLAN

0.6 * W = 14.76 k
0.7 * EQ = 36.79 k

0.6 * W = 25.56 k
0.7 * EQ = 36.79 k

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Project Title: Powder Mountain
Engineer: Alex Hawkins
Project ID: 8332
Project Descr:

Printed: 7 JUN 2018, 9:57AM

File = C:\Users\ahawkins\Desktop\3SIDED~1.EC6

Torsional Analysis of Rigid Diaphragm

Lic. # : KW-06004069 **Licensee : ENSIGN ENGINEERING**

Description : Upper Level - Find center of Rigidity / where to apply load from upper level on to main level

General Information

Calculations per IBC 2015, CBC 2016, ASCE 7-1

Applied Lateral Force	26.610 k	
.....Additional Orthogonal Force	k	
Maximum Load Used for Analysis :	26.610 k	

Note: This load is the vector resolved from the above two entries and will be applied to the system of elements at angular increments.

Load Orientation Angular Increment	15.0 deg
Load Location Angular Increment	15.0 deg

Center of Rigidity Location (calculated) . . .

"X" dist. from Datum	11.521 ft
"Y" dist. from Datum	31.949 ft

Center of Shear Application :

Distance from "X" datum point	12.0 ft
Distance from "Y" datum point	18.0 ft

Accidental Torsion values per ASCE 7-05 12.8.4.2

Ecc. as % of Maximum Dimension	5.00 %

Maximum Dimensions :

Along "X" Axis	24.0 ft
Along "Y" Axis	46.0 ft

Accidental Eccentricity +/- from "X" Coord. of Load Application :	1.20 ft
Accidental Eccentricity +/- from "Y" Coord. of Load Application :	2.30 ft

Wall Information

Label	Along Wall "y" Dir (in, for 1.0 kip load)	Along Wall "x" Dir (in, for 1.0 kip load)	X Wall C.G. Location	Y Wall C.G. Location	Wall Angle CCW	Wall Fixity	Length	Height	Thickness	E - Bending	E - Shear
1 Mid	2.4533E-004 in	5.3493E+003 in	6 ft	36 ft	0 deg	Fix-Fix	12.5 ft	10 ft	6 in	1 Mpsi	1 Mpsi
2 Mid	8.3822E-004 in	9.9061E+003 in	20 ft	34 ft	9.5 deg	Fix-Fix	6.75 ft	10 ft	6 in	1 Mpsi	1 Mpsi
3 Left	2.9483E-004 in	5.9436E+003 in	0 ft	25 ft	90 deg	Fix-Fix	11.25 ft	10 ft	6 in	1 Mpsi	1 Mpsi
4 Right	5.4705E-003 in	2.0574E+004 in	24 ft	32.375 ft	90 deg	Fix-Fix	3.25 ft	10 ft	6 in	1 Mpsi	1 Mpsi
5 Right	3.6667E-004 in	6.6866E+003 in	24 ft	17 ft	90 deg	Fix-Fix	10 ft	10 ft	6 in	1 Mpsi	1 Mpsi
6 Front	1.1049E-003 in	1.1144E+004 in	21 ft	11 ft	9.5 deg	Fix-Fix	6 ft	10 ft	6 in	1 Mpsi	1 Mpsi
7 Back	1.1467E-002 in	2.6746E+004 in	1.25 ft	42 ft	9.5 deg	Fix-Fix	2.5 ft	10 ft	6 in	1 Mpsi	1 Mpsi
8 Back	6.8395E-003 in	2.2289E+004 in	22.5 ft	45 ft	9.5 deg	Fix-Fix	3 ft	10 ft	6 in	1 Mpsi	1 Mpsi

Wall Deflections (Stiffness) for 1.0 kip load given per wall above (Along Wall "y" Dir, Along Wall "x" Dir).

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Project ID: 8332
Project Descr:

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Torsional Analysis of Rigid Diaphragm

Description : Upper Level - Find center of Rigidity / where to apply load from upper level on to main level

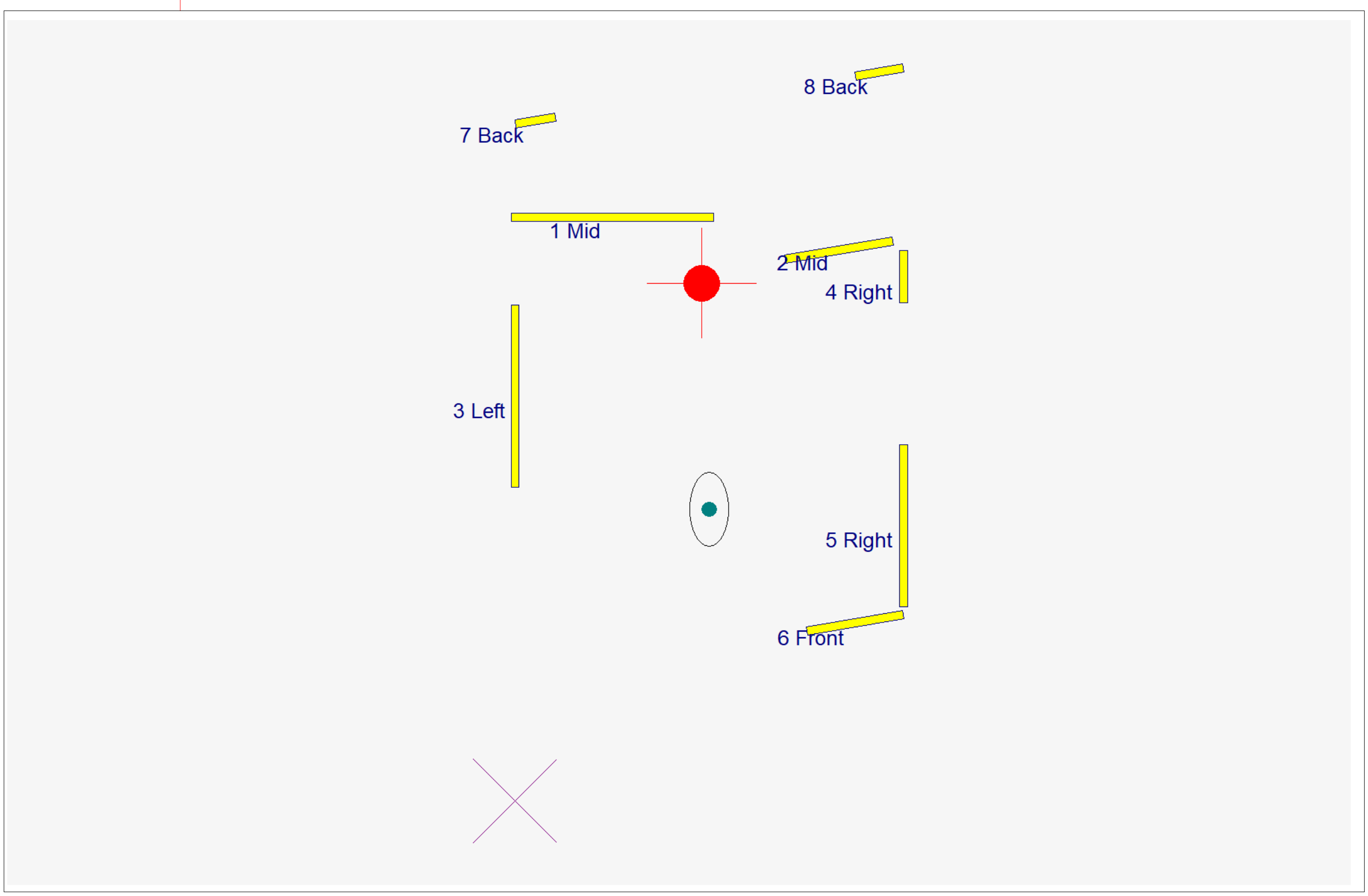
ANALYSIS SUMMARY

Maximum shear forces applied to resisting elements. Eccentricity with respect to Center of Rigidity

	Max Shear along Member Local "y-y" Axis				Max Shear along Member Local "x-x" Axis			
Resisting Element	Load Angle	X-Ecc (ft)	Y-Ecc (ft)	Shear Force (k)	Load Angle	X-Ecc (ft)	Y-Ecc (ft)	Shear Force (k)
1 Mid	0	0.72	-13.95	17.158	0	-0.48	-16.25	0.000
2 Mid	0	-0.17	-16.17	4.937	0	-0.48	-16.25	0.036
3 Left	45	-1.08	-15.94	19.310	345	-0.17	-16.17	0.000
4 Right	45	-0.17	-16.17	1.059	345	-0.79	-11.73	0.000
5 Right	45	-0.17	-16.17	15.803	345	-0.17	-16.17	0.000
6 Front	0	-0.17	-16.17	3.860	0	-0.48	-16.25	0.978
7 Back	0	-0.48	-16.25	0.364	0	-0.48	-16.25	0.049
8 Back	0	-0.17	-16.17	0.618	0	-0.48	-16.25	0.079

Layout of Resisting Elements

Legend : Defined Wall — Center of Rigidity — Center of Mass — Accidental eccentricity application boundary — X Datum



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Torsional Analysis of Rigid Diaphragm

Description : Upper Level - Find center of Rigidity / where to apply load from upper level on to main level

Analysis Notes

This program is designed to distribute an applied shear load to a set of resisting elements.

Each resisting element data entry specifies a deflection along a "major" and "minor" axis due to a 1,000 lb load. Each resisting element may be entered as a wall or a column (whereby the deflection is calculated), or as a generic resisting element with specified deflection. The deflections define the stiffness of each resisting element.

Each resisting element is defined at an (X,Y) location from a datum the user has previously defined. A counter-clockwise rotation of the element can be entered with respect to a traditional "+X" axis line.

A main "shear" load and an optional orthogonal shear load are specified for distribution to the system of resisting elements. In addition the maximum orthogonal dimensions of the structure and minimum accidental eccentricity percentage are specified.

From the entered loads the program calculates resultant force vectors for each angular orientation that is requested. The force is applied to the resisting elements in angular increments to generate a series of resulting direct and torsional shear loads on each element. This application of force is then repeated at angular intervals along an elliptical path defined by the minimum accidental eccentricity.

The end result is a table of direct shear and torsional shear values for each element from the iterated angles of load application and accidental eccentricity. These values are then searched to find the maximum major and minor axis shears applied to each resisting element.

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Torsional Analysis of Rigid Diaphragm

File = P:\8332 Powder Mountain 70\Structural Calcs\3 Sided Diaphragms.ec6

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Description : Main Level - Average of center of rigidity above and

General Information

Calculations per IBC 2015, CBC 2016, ASCE 7-1

Applied Lateral Force	34.420 k
.....Additional Orthogonal Force	10.326 k
Maximum Load Used for Analysis :	35.936 k

Note: This load is the vector resolved from the above two entries and will be applied to the system of elements at angular increments.

Load Orientation Angular Increment	15.0 deg
Load Location Angular Increment	15.0 deg

Center of Rigidity Location (calculated) . . .

"X" dist. from Datum	16.088 ft
"Y" dist. from Datum	38.482 ft

Center of Shear Application :

Distance from "X" datum point	12.0 ft
Distance from "Y" datum point	29.918 ft

Accidental Torsion values per ASCE 7-05 12.8.4.2

Ecc. as % of Maximum Dimension	5.00 %

Maximum Dimensions :

Along "X" Axis	24.0 ft
Along "Y" Axis	46.0 ft

Accidental Eccentricity +/- from "X" Coord. of Load Application :	1.20 ft
Accidental Eccentricity +/- from "Y" Coord. of Load Application :	2.30 ft

Wall Information

Label	Along Wall "y" Dir (in)	Along Wall "x" Dir (in)	X Wall C.G. Location	Y Wall C.G. Location	Wall Angle CCW	Wall Fixity	Length	Height	Thickness	E - Bending	E - Shear
1 Left	1.7333E-003	1.3373E+004	0 ft	14 ft	90 deg	Fix-Fix	5 ft	10 ft	6 in	1 Mpsi	1 Mpsi
2 Left	2.8350E-004	5.8144E+003	0 ft	36 ft	90 deg	Fix-Fix	11.5 ft	10 ft	6 in	1 Mpsi	1 Mpsi
3 Back	1.1467E-002	2.6746E+004	1.25 ft	42 ft	9.5 deg	Fix-Fix	2.5 ft	10 ft	6 in	1 Mpsi	1 Mpsi
4 Back	5.0668E-004	7.8666E+003	19.75 ft	45 ft	9.5 deg	Fix-Fix	8.5 ft	10 ft	6 in	1 Mpsi	1 Mpsi
5 Right	1.8272E-004	4.4577E+003	24 ft	38.5 ft	90 deg	Fix-Fix	15 ft	10 ft	6 in	1 Mpsi	1 Mpsi
6 Right	3.2021E-004	6.2201E+003	24 ft	16.75 ft	90 deg	Fix-Fix	10.75 ft	10 ft	6 in	1 Mpsi	1 Mpsi
7 Mid	4.2057E-004	2.6896E+004	6.25 ft	33.5 ft	0 deg	Fix-Fix	12.5 ft	10 ft	3.5 in	1 Mpsi	1 Mpsi

(Wall Deflections (Stiffness) for 1.0 kip load)

ANALYSIS SUMMARY

Maximum shear forces applied to resisting elements. Eccentricity with respect to Center of Rigidity

	Max Shear along Member Local "y-y" Axis				Max Shear along Member Local "x-x" Axis			
Resisting Element	Load Angle	X-Ecc (ft)	Y-Ecc (ft)	Shear Force (k)	Load Angle	X-Ecc (ft)	Y-Ecc (ft)	Shear Force (k)
1 Left	315	4.09	-10.86	3.102	345	4.40	-10.79	0.000
2 Left	315	4.09	-10.86	18.969	345	4.40	-10.79	0.000

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Project Title: Powder Mountain
Engineer: Alex Hawkins
Project ID: 8332
Project Descr:

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Torsional Analysis of Rigid Diaphragm

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Description : Main Level - Average of center of rigidity above and

ANALYSIS SUMMARY

Maximum shear forces applied to resisting elements. Eccentricity with respect to Center of Rigidity

	Max Shear along Member Local "y-y" Axis				Max Shear along Member Local "x-x" Axis			
Resisting Element	Load Angle	X-Ecc (ft)	Y-Ecc (ft)	Shear Force (k)	Load Angle	X-Ecc (ft)	Y-Ecc (ft)	Shear Force (k)
3 Back	0	4.09	-10.86	0.704	345	4.40	-10.79	0.020
4 Back	0	4.40	-10.79	16.008	345	4.40	-10.79	0.449
5 Right	105	5.13	-9.71	21.335	345	3.78	-6.34	0.000
6 Right	105	5.13	-9.71	12.174	345	4.40	-10.79	0.000
7 Mid	0	4.09	-10.86	22.211	345	4.40	-10.79	0.000

Layout of Resisting Elements

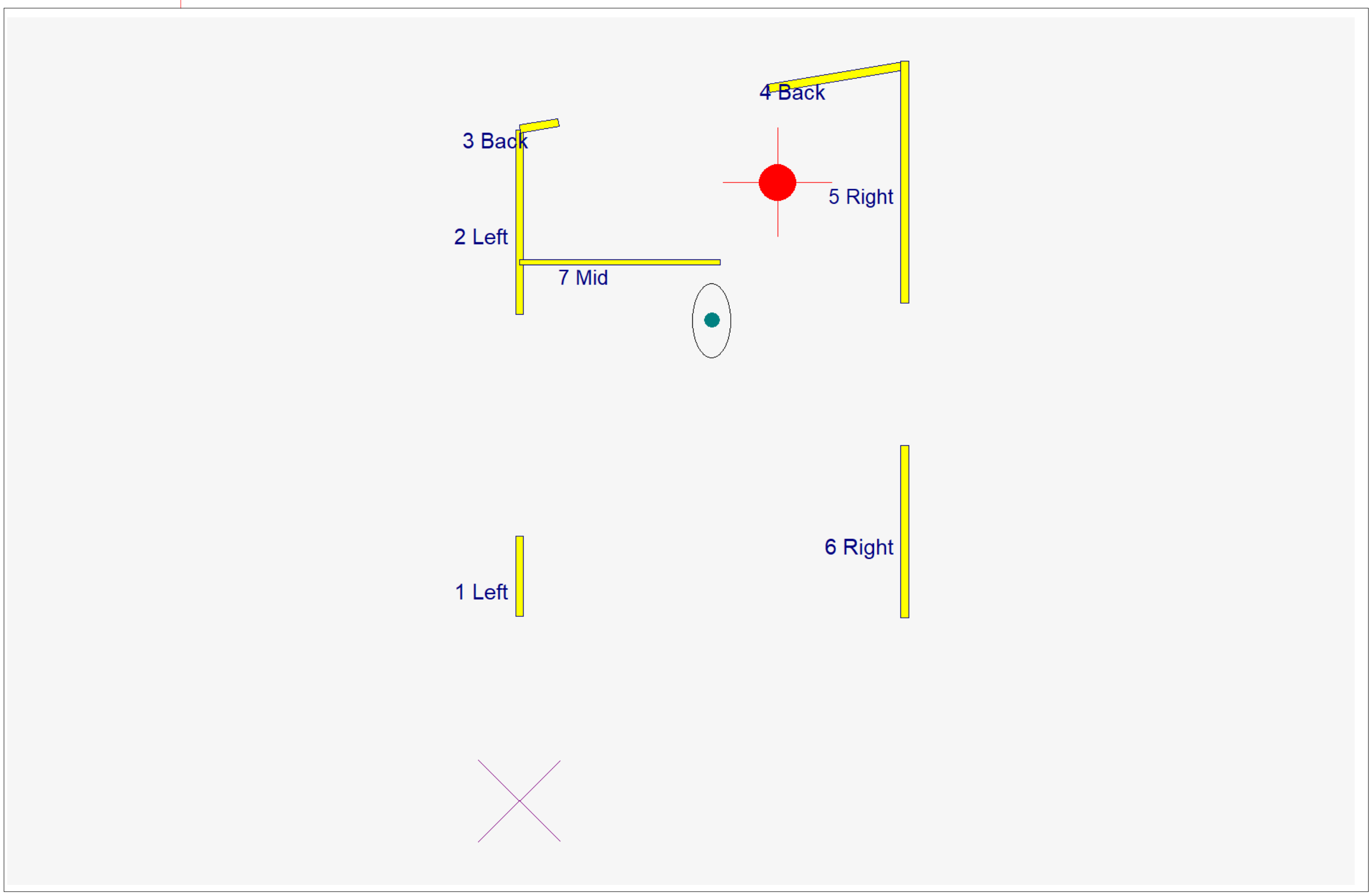
Legend : Defined Wall

X Datum

Center of Rigidity

Center of Mass

Accidental eccentricity application boundary



Torsional Analysis of Rigid Diaphragm

Description : Main Level - Average of center of rigidity above and

Analysis Notes

This program is designed to distribute an applied shear load to a set of resisting elements.

Each resisting element data entry specifies a deflection along a "major" and "minor" axis due to a 1,000 lb load. Each resisting element may be entered as a wall or a column (whereby the deflection is calculated), or as a generic resisting element with specified deflection. The deflections define the stiffness of each resisting element.

Each resisting element is defined at an (X,Y) location from a datum the user has previously defined. A counter-clockwise rotation of the element can be entered with respect to a traditional "+X" axis line.

A main "shear" load and an optional orthogonal shear load are specified for distribution to the system of resisting elements. In addition the maximum orthogonal dimensions of the structure and minimum accidental eccentricity percentage are specified.

From the entered loads the program calculates resultant force vectors for each angular orientation that is requested. The force is applied to the resisting elements in angular increments to generate a series of resulting direct and torsional shear loads on each element. This application of force is then repeated at angular intervals along an elliptical path defined by the minimum accidental eccentricity.

The end result is a table of direct shear and torsional shear values for each element from the iterated angles of load application and accidental eccentricity. These values are then searched to find the maximum major and minor axis shears applied to each resisting element.

SHEAR WALL SCHEDULE

CHAPTER 4.3, AWC SDPWS-2015 *IBC 2015 / ASCE 7-10*

Mark	Nailing Requirements		Notes	V_{allow} (8)	V_{allow} (8)	Sole Plate Nailing (10 & 13)
	Edge	Field		Seismic (plf)	Wind (plf)	(Sole Plate to 2x blocking or rim)
SW1	6"	12"	1,2,3	260	365	16d common @ 6" o.c.
SW2	4"	12"	1,2,3	350	490	16d common @ 4" o.c.
SW3	4"	12"	1,2,3,4	380	532	16d common @ 4" o.c.
SW4	3"	12"	1,2,3,4	490	685	(2) 16d common @ 6" o.c.
SW5	2"	12"	1,2,3,4	640	895	(2) 16d common @ 6" o.c.
SW6	2"	12"	1,3,4,6	770	1078	(2) 16d common @ 4" o.c.
SW7	3"	12"	1,2,3,4,5	980	1370	(2) SDS screws @ 6" o.c.
SW8	2"	12"	1,2,3,4,5,11	1280	1790	(2) SDS screws @ 4" o.c.
SW9	2"	12"	1,3,4,5,6,11	1540	2155	(2) SDS screws @ 4" o.c.
SW10	2"	12"	1,3,4,5,7,11	1740	2435	(2) SDS screws @ 3" o.c.

Notes:
1. 16" o.c. max stud spacing or panels applied with the long dimension across the studs (AF&PA SDPWS table 4.3A note 2).
2. 7/16" APA rated sheathing panel with 8d common or galvanized box nails.
3. Block all edges.
4. 3" nominal framing at abutting panel edges (AF&PA SDPWS 4.3.7.1.5.c)
5. Sheathing applied to both sides of wall
6. 15/32" APA rated sheathing with 10d common or galvanized box nails
7. 15/32" APA Structural I rated sheathing with 10d common or galvanized box nails
8. Allowable shear values per AF&PA SDPWS table 4.3A.
9. For all walls, provide hot dipped zinc-coated galvanized steel, stainless steel, silicon bronze or copper nails at preservative-treated and fire-retardant-treated wood locations.
10. SDS screws to be 4.5" minimum length and penetrate 2" into rim board or blocking
11. SDS screws must be into 2x DFL blocking or 2x DFL rim board (not LVL or LSL)
12. Where panels are applied to both faces of the wall and nail spacing is less than 6" on center on either side offset panel joints to different framing members.
13. If (2) SDS screws are required on the sole plate nailing 2x blocking must be provided adjacent to rimboard or (2) 2x blocks must be provided. SDS screws require 5/8" edge and 3" end distance.

STAPLE EQUIVALENCY CHART

Staple Type	Stapling Requirements		V_{allow} (8)	Equivalent to Nailed Shearwall	V_{allow} (8)	Equivalent to Nailed Shearwall
	Edge	Field	Seismic (plf)	designated above:	Wind (plf)	designated above:
16 Gage 1 1/2" Staples	6"	6"	155	NONE	215	NONE
	4"	6"	230	NONE	320	NONE
	3"	6"	310	SW1	435	SW1
	2"	6"	395	SW2 and SW3	555	SW2 and SW3

Notes:
1. Minimum staple penetration into main member is 1".
2. Staples shall have a minimum crown width of 7/16".
3. Install staple crown parallel to the long dimension of the framing member.
4. Where staple spacing is 2" or less, framing at adjoining panel edges shall be 3" nominal.
5. Provide 3/8" distance from panel edge to staple.
6. Table valid for shearwalls only.
7. Provide hot dipped zinc-coated galvanized steel, stainless steel, silicon bronze or copper staples at perservative-treated and fire-retardant- treated wood locations.
8. Allowable shear values per ICC-ES Evaluation Report ESR-1539 and IBC 2015 Table 2306.3(1).
9. Allowable shear values shown are based on 7/16" nominal sheathing thickness.

ENSIGN
THE STANDARD IN ENGINEERING

Project:	Blake Kingsbury and Merrit Chesson	Project No.:	8332
By:	Alex Hawkins, PE	Checked By:	DAJ
Date:	June 2018		

SINGLE-STORY WOOD SHEAR WALLS

CHAPTER 4.3, AWC SDPWS-2015 *IBC 2015 / ASCE 7-10*

NOTES: 1. Typically when seismic is found to govern wind loads will not be checked here. However, if wind loads are found to govern both wind and seismic need to be checked in order to account for the difference in shearwall capacities.
2. ASD loads are to be entered here.
3. PSW is defined as Perforated Shear Wall.
4. For PSW analysis the Length column is entered as the sum of the PSW segment lengths. The Shortest Wall Segment column is entered as the shortest segment in the PSW. The Opening Height column is the worst case opening height of all the openings in the PSW.

S_{DS}= 0.66

Grid A
Roof Balcony

Wind Force on Wall Line: 1,020 lbs — lbs / Dowel: 2,750 lbs — Total: 0 lbs
Seismic Force on Wall Line: 4,711 lbs — # of Dowels: 0 — NG

Wall ID	# of Walls	Length (ft)	Height (ft)	H:W Ratio	Aspect Ratio Reduct.	Effective Length $(2b_s/h)*L$	Total Length (ft)	Uniform DL (plf)	Seismic Uplift (lb)	Wind Uplift (lb)	PSW	Shortest Wall Seg. (ft)	Opening Height (ft)	Opening Length (ft)	Co, PSW Reduct.	Holdown Required
1	1	22.75	3.5	0.15	1.00	22.75	22.75	145	0	0					1.00	

Total Length: **22.75**
Wind Force / Wind Length = **45** **plf (SW1)** **Use: SW1**
Seismic Force / Seismic Length = **207** **plf (SW1)**
Anchor Bolt Size (inches): NA
Anchor Bolt Designation: **NA**

Grid B
Roof Balcony

Wind Force on Wall Line: 1,020 lbs — lbs / Dowel: 2,750 lbs — Total: 0 lbs
Seismic Force on Wall Line: 4,711 lbs — # of Dowels: 0 — NG

Wall ID	# of Walls	Length (ft)	Height (ft)	H:W Ratio	Aspect Ratio Reduct.	Effective Length $(2b_s/h)*L$	Total Length (ft)	Uniform DL (plf)	Seismic Uplift (lb)	Wind Uplift (lb)	PSW	Shortest Wall Seg. (ft)	Opening Height (ft)	Opening Length (ft)	Co, PSW Reduct.	Holdown Required
1	1	22.75	3.5	0.15	1.00	22.75	22.75	145	0	0					1.00	

Total Length: **22.75**
Wind Force / Wind Length = **45** **plf (SW1)** **Use: SW1**
Seismic Force / Seismic Length = **207** **plf (SW1)**
Anchor Bolt Size (inches): NA
Anchor Bolt Designation: **NA**

Grid A
Upper Floor

Wind Force on Wall Line: 2,850 lbs — lbs / Dowel: 2,750 lbs — Total: 16,500 lbs
Seismic Force on Wall Line: 13,837 lbs — # of Dowels: 6 — OK

Grid B
Upper Floor

Wind Force on Wall Line: 2,850 lbs — lbs / Dowel: 2,750 lbs — Total: 19,250 lbs
Seismic Force on Wall Line: 13,837 lbs — # of Dowels: 7 — OK

Grid A - See 3 Sided Diaphr
Main Floor

Wind Force on Wall Line: 5,160 lbs — lbs / Dowel: 2,750 lbs — Total: 22,000 lbs
Seismic Force on Wall Line: 21,987 lbs — # of Dowels: 8 — OK
Anchor Bolt Size (inches): 5/8
Anchor Bolt Designation: **AB24**

Grid B - 3 Sided Diaphragm
Main Floor

Wind Force on Wall Line: 5,160 lbs — lbs / Dowel: 2,750 lbs — Total: 35,750 lbs
Seismic Force on Wall Line: 33,380 lbs — # of Dowels: 13 — OK
Anchor Bolt Size (inches): 5/8
Anchor Bolt Designation: **AB12**

ENSIGN THE STANDARD IN ENGINEERING

Project: Blake Kingsbury and Merrit Chesson | Project No.: 8332
By: Alex Hawkins, PE | Checked By: DAJ
Date: June 2018

SINGLE-STORY WOOD SHEAR WALLS

CHAPTER 4.3, AWC SDPWS-2015 *IBC 2015 / ASCE 7-10*

NOTES: 1. Typically when seismic is found to govern wind loads will not be checked here. However, if wind loads are found to govern both wind and seismic need to be checked in order to account for the difference in shearwall capacities.
2. ASD loads are to be entered here.
3. PSW is defined as Perforated Shear Wall.
4. For PSW analysis the Length column is entered as the sum of the PSW segment lengths. The Shortest Wall Segment column is entered as the shortest segment in the PSW. The Opening Height column is the worst case opening height of all the openings in the PSW.

S_{DS}= 0.66

Grid 1 Wind Force on Wall Line: 960 lbs | lbs / Dowell: 2,750 lbs | Total: 0 lbs
Roof Balcony Seismic Force on Wall Line: 4,711 lbs | # of Dowells: 0 | NG

Wall ID	# of Walls	Length (ft)	Height (ft)	H:W Ratio	Aspect Ratio Reduct.	Effective Length $(2b_s/h)$*L	Total Length (ft)	Uniform DL (plf)	Seismic Uplift (lb)	Wind Uplift (lb)	PSW	Shortest Wall Seg. (ft)	Opening Height (ft)	Opening Length (ft)	Co, PSW Reduct.	Holdown Required
1	2	6.00	11.0	1.83	1.00	6.00	12.00	210	4234	531					1.00	MSTC52 - (44)

Total Length: **12.00** Wind Force / Wind Length = **80** **plf (SW1)** **Use: SW4** Anchor Bolt Size (inches): NA
Seismic Force / Seismic Length = **393** **plf (SW4)** Anchor Bolt Designation: **NA**

Grid 2 Wind Force on Wall Line: 960 lbs | lbs / Dowell: 2,750 lbs | Total: 0 lbs
Roof Balcony Seismic Force on Wall Line: 4,711 lbs | # of Dowells: 0 | NG

Wall ID	# of Walls	Length (ft)	Height (ft)	H:W Ratio	Aspect Ratio Reduct.	Effective Length $(2b_s/h)$*L	Total Length (ft)	Uniform DL (plf)	Seismic Uplift (lb)	Wind Uplift (lb)	PSW	Shortest Wall Seg. (ft)	Opening Height (ft)	Opening Length (ft)	Co, PSW Reduct.	Holdown Required
1	1	3.00	5.0	1.67	1.00	3.00	3.00	165	8691	1633					1.00	CMST12

Total Length: **3.00** Wind Force / Wind Length = **320** **plf (SW1)** **Use: SW1** Anchor Bolt Size (inches): NA
Seismic Force / Seismic Length = **1570** **plf (SW10)** Anchor Bolt Designation: **NA**

Grid 1 Wind Force on Wall Line: 2,010 lbs | lbs / Dowell: 2,750 lbs | Total: 8,250 lbs
Upper Floor Seismic Force on Wall Line: 7,753 lbs | # of Dowells: 3 | OK

Grid 2 Wind Force on Wall Line: 4,110 lbs | lbs / Dowell: 2,750 lbs | Total: 0 lbs
Upper Floor Seismic Force on Wall Line: 13,839 lbs | # of Dowells: 0 | NG

Wall ID	# of Walls	Length (ft)	Height (ft)	H:W Ratio	Aspect Ratio Reduct.	Effective Length $(2b_s/h)$*L	Total Length (ft)	Uniform DL (plf)	Seismic Uplift (lb)	Wind Uplift (lb)	PSW	Shortest Wall Seg. (ft)	Opening Height (ft)	Opening Length (ft)	Co, PSW Reduct.	Holdown Required
1	1	12.50	10.0	0.80	1.00	12.50	12.50	200	7473	1642					1.00	HDU11
2	1	5.00	10.0	2.00	1.00	5.00	5.00	200	8200	2195					1.00	HDU11

Total Length: **17.50** Wind Force / Wind Length = **235** **plf (SW1)** **Use: SW7** Anchor Bolt Size (inches): NA
Seismic Force / Seismic Length = **791** **plf (SW7)** Anchor Bolt Designation: **NA**

Grid 3 Wind Force on Wall Line: 2,100 lbs | lbs / Dowell: 2,750 lbs | Total: 8,250 lbs
Upper Floor Seismic Force on Wall Line: 6,086 lbs | # of Dowells: 3 | OK

ENSIGN
THE STANDARD IN ENGINEERING

Project: Blake Kingsbury and Merrit Chesson
By: Alex Hawkins, PE
Date: June 2018

Project No.: 8332
Checked By: DAJ

Grid 1 - 3 Sided Diaphragm
Main Floor

Wind Force on Wall Line: lbs
Seismic Force on Wall Line: 12,018 lbs
Anchor Bolt Size (inches): 5/8
Anchor Bolt Designation: **AB24**

lbs / Dowell: 2,750 lbs
# of Dowells: 5

Total: 13,750 lbs
OK

Grid 2 - 3 Sided Diaphragm
1st Floor

Wind Force on Wall Line: lbs
Seismic Force on Wall Line: 22,125 lbs

Wall ID	# of Walls	Length (ft)	Height (ft)	H:W Ratio	Aspect Ratio Reduct.	Effective Length $(2b_s/h)*L$	Total Length (ft)	Uniform DL (plf)	Seismic Uplift (lb)	Wind Uplift (lb)	PSW	Shortest Wall Seg. (ft)	Opening Height (ft)	Opening Length (ft)	Co, PSW Reduct.	Holdown Required
1	1	12.75	10.0	0.78	1.00	12.75	12.75	250	22128	0					1.00	(2) HDU14

Total Length: **12.75**

Wind Force / Wind Length = **0** **plf (SW1)** **Use: SW1**
Seismic Force / Seismic Length = **1735** **plf (SW10)**

Anchor Bolt Size (inches): 5/8
Anchor Bolt Designation: **AB8**

Beam Id: Lot70 / Garage column   ROINFORCED CONCR. (SIDE WALL)   26" × 10" 660 mm × 250 mm   Date 30-05-2018

Structural Engineer:

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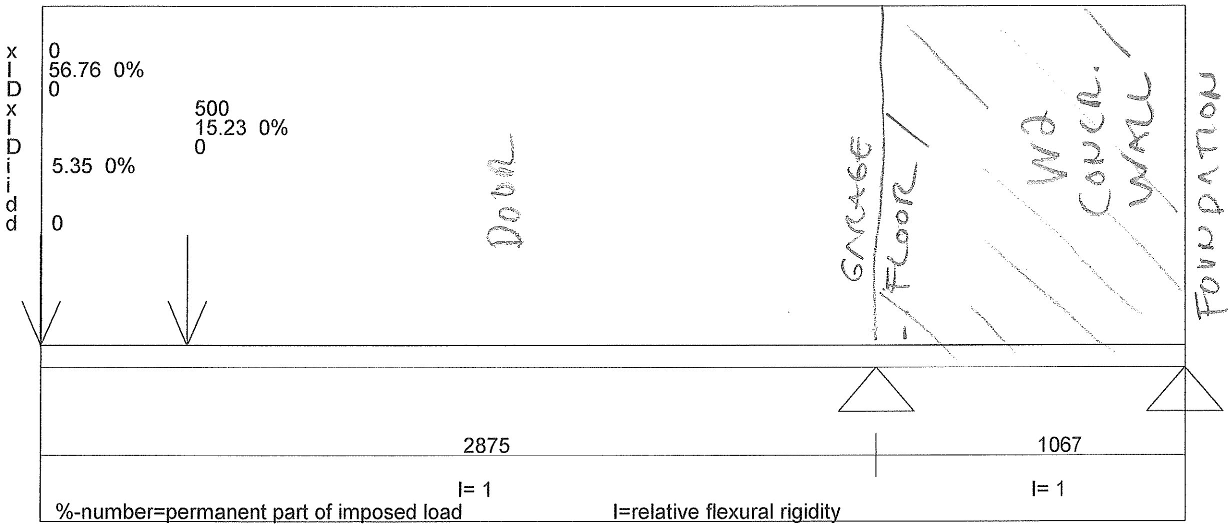

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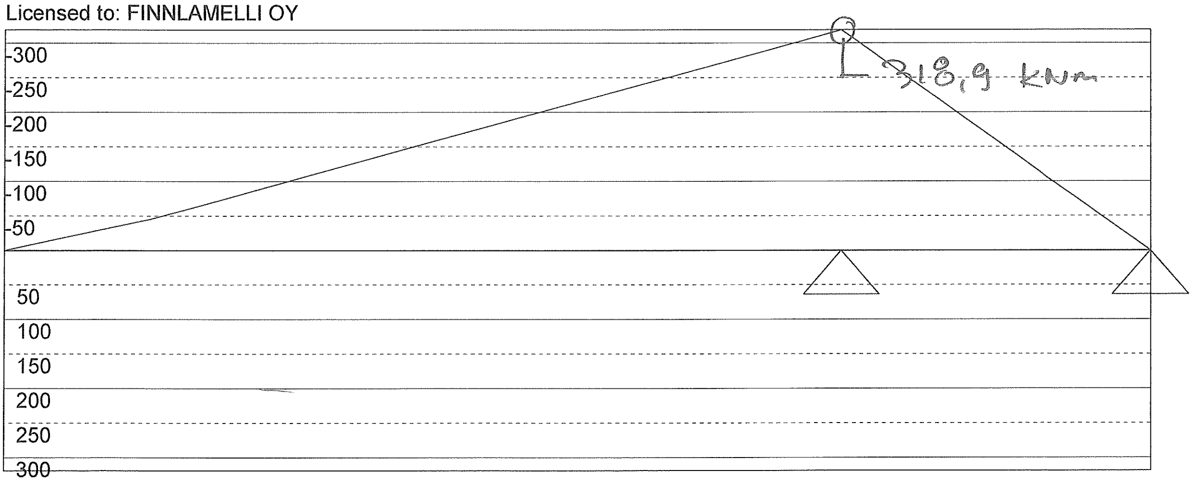

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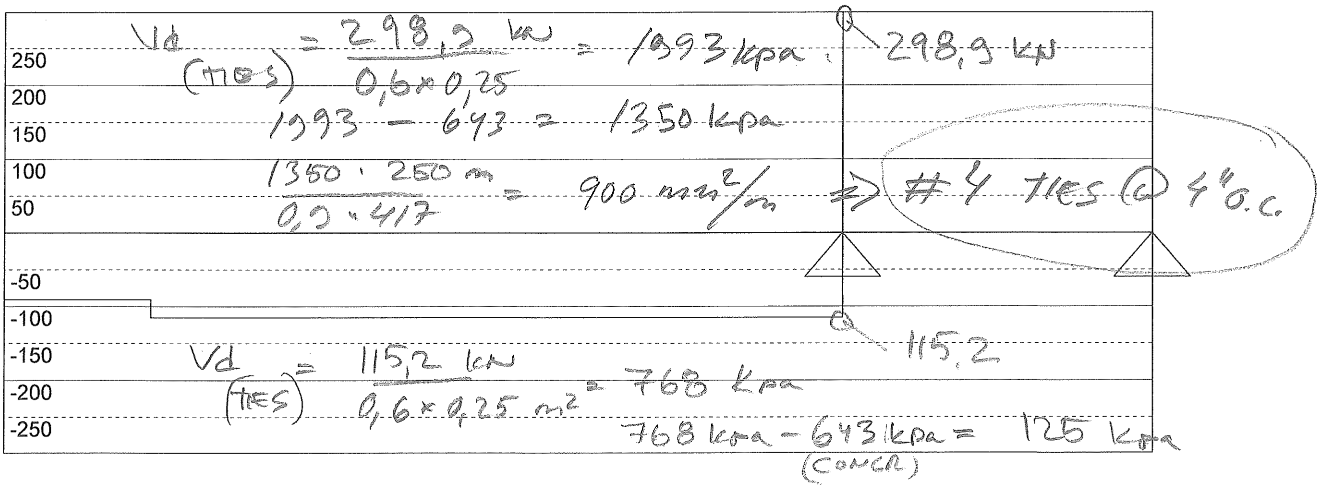


Load factor of dead load= 1.2 Load factor of imposed load= 1.6
Load width 1 (m) (by which the loads
has been multiplied during calculation)
Max/Min reactions of beam [kN]
414,125 0,000
0,000 -298,941

$\frac{125 \cdot 250}{0,9 \cdot 417} = 83\ mm^2/m$

⇒ MIN TIES ⇒ #3 @ 8" o.c.

$Md_{(MAIN\ BARS)} = \frac{Md}{Z \cdot fyd} = \frac{318,9 \cdot 10^6}{600 \cdot 417} = 1275\ mm^2$

⇒ (4) #7 BARS

ENSIGN
THE STANDARD IN ENGINEERING

Project: Blake Kingsbury and Merrit Chesson
By: Alex Hawkins, PE
Date: June 2018

Project No.: 8332
Checked By: DAJ

HOLDOWN & VERTICAL STRAP SCHEDULE

IBC 2015 / ASCE 7-10

HOLDOWN INTO CONCRETE					
Mark	Anchor	Wind or Seismic Capacity (LBS)	Rod Diameter	Min. Post Size	Minimum Embed Depth in Footing
H-1	HTT4 w/ (18) 10dx1½ nails	3610	5/8"	3" x 3 1/2"	9"
H-2	HTT5 w/ (26) 10d nails	4670	5/8"	3" x 3 1/2"	9"
H-3	HDU5 - SDS2.5 (14)	5645	5/8"	3" x 3 1/2"	9"
H-4	HDU8 - SDS2.5 (20)	7870	7/8"	4 1/2" x 3 1/2"	10 1/2"
H-5	HDU11 - SDS2.5 (30)	9535	1"	5 1/2" x 3 1/2"	14"
H-6	HDU11 - SDS2.5 (30)	11175	1"	7 1/4" x 3 1/2"	14"
H-7	HDU14 - SDS2.5 (36)	14445	1"	5 1/2" x 5 1/2"	14"
		14375	1"	7 1/4" x 3 1/2"	14"

FLOOR TO FLOOR TIES (STRAPS OR RODS)				
Mark	Anchor	Wind or Seismic Capacity (LBS)	Rod Diameter	Min. Post Size
	Strap Type			
T-1	CS16 - (20)-11"	1705	NA	2x
T-2	MSTC40 - (28)	2695	NA	(2) 2x
T-3	MSTC52 - (44)	4235	NA	(2) 2x
T-4	MSTC66 - (64)	5860	NA	(2) 2x
	Rod Type			
T-5	HDU2-SDS2.5 (6)	3075	5/8"	3" x 3 1/2"
T-6	HTT4 w/ (18) 10dx1½ nails	3610	5/8"	3" x 3 1/2"
T-7	HTT5 w/ (26) 10d nails	4670	5/8"	3" x 3 1/2"
T-8	HDU5 - SDS2.5 (14)	5645	5/8"	3" x 3 1/2"
T-9	HDU8 - SDS2.5 (20)	7870	7/8"	4 1/2" x 3 1/2"
T-10	(2) HDU4-SDS2.5 (20)	9130	5/8"	3" x 3 1/2"
T-11	HDU11 - SDS2.5 (30)	9535	1"	5 1/2" x 3 1/2"
T-12	HDU14 - SDS2.5 (36)	14445	1"	5 1/2" x 5 1/2"
		14375	1"	7 1/4" x 3 1/2"

Notes:

All anchors are Simpson Strong-Tie. Install per manufacturer's specifications.
Use 4" end distance at foundation blockouts.
CS straps are specified with: strap type - total # of of 10d nails required - end length required
MSTC straps are specified with: strap type - total # of 16d sinker nails required
All straps are designed for 18" max floor to floor clear span
Provide 3/8" X 1 1/2" X 1 1/2" plate washer for 5/8" dia. anchors, 3/8" X 2 1/4" X 2 1/4" plate washer for 7/8" dia. anchors, 3/8" X 2 1/2" X 2 1/2" plate washer for 1" dia. anchors. Provide nut top and bottom.
For stem wall applications use simspon SB 5/8" x 24" embed 18" min. in wall for HTT4, HTT5, HDU5 holdowns.
Ensure that the Min. Edge distances are met for all anchors in concrete.
Min. anchor bolt strength is ASTM F-1554 GRADE 36 U.N.O.

HORIZONTAL STRAP SCHEDULE

Mark	Strap Req.	Notes
**	CS16	Strap horizontally above and below window. Header above and sill below window must be continuous. Provide 2x blocking in wall as required and (2) 2x sill.

ANCHOR BOLTS

2015 NDS Table 12E

1/2" Diameter Anchor Bolts		
Mark	Bolt Spacing	Capacifty (plf)
AB32	32"	384
AB24	24"	512
AB16	16"	768
AB12	12"	1024
AB8	8"	1536

5/8" Diameter Anchor Bolts		
Mark	Bolt Spacing	Capacifty (plf)
AB32	32"	552
AB24	24"	736
AB16	16"	1104
AB12	12"	1472
AB8	8"	2208

Notes:
7" minimum embedment depth on all anchor bolts.
3" x 3" x 0.229" plate washers on all anchor bolts. 1/2" away from sheathing.
(2) anchor bolts min. per shear wall.
Anchors are located a minimum of 1 3/4" away from the edge of concrete
Anchor bolts are to be located 15 anchor diameters away from a concrete edge that is perpendicular to the sill plate.
Sill plate is 2x or 3x minimum. (Capacities shown here are based on a 2x sill plate)

ROOF FRAMING

NDS 2015 EDITION *IBC 2015 / ASCE 7-10*

Roof Trusses:

Use pre-engineered trusses @ 24" o.c.
Provide truss blocking as shown on plans and per manufacturer's specifications.
All truss connection hardware to be designed by the truss manufacturer.
Provide full depth blocking at all bearing locations with (1) A35 clip to top plate per block U.N.O.
Nail through sheathing with 8d common @ 4" o.c. into blocking U.N.O.
Provide "H1" clips at both ends of every truss U.N.O.

Roof Stick Frame:

Use roof joists per span chart.
Provide full depth blocking at all bearing locations with (1) A35 clip to top plate per block UNO.
Nail through sheathing with 8d common @ 4" o.c. into blocking U.N.O.
Provide "H1" clips at both ends of every joist UNO.

Roof Overbuild:

Frame roof overbuild areas with 2x6 DF#2 @ 24" o.c.
Brace joists at 6' 0" o.c.
Use 2x8 DF#2 ridge board braced at 4' 0" o.c.
Use 2x8 DF#2 valley members laid flat and nailed to trusses with (2) 16d per truss.
Brace ridge and joists such that load is distributed uniformly to trusses below.
Sheath under all overbuild areas.
Provide access and ventilation to overbuild areas as necessary.

Roof Beams:

See attached beam calculations.

Roof Sheathing:

Provide 5/8" or thicker 24/16 APA rated panel.
Nail with 8d common at 6" o.c. at panel edge and 12" o.c. in the field.
Provide 'H' clips at all unsupported edges.
Provide 1/8" gap between panels at time of installation.

ENSIGN THE STANDARD IN ENGINEERING	Project: Blake Kingsbury and Merrit Chesson	Project No.:	8332
	By: Alex Hawkins, PE	Checked By:	DAJ
	Date: June 2018		

FLOOR FRAMING

NDS 2015 EDITION — *IBC 2015 / ASCE 7-10*

Floor Joists:

TJI Engineered Floor Joist Span Tables: 20DL + 40LL + L/480					
Depth	Series	12" o.c.	16" o.c.	19.2" o.c.	24" o.c.
9 1/2"	*110*	16'-11"	15'-6"	14'-7"	13'-7"
9 1/2"	*210*	17-9"	16'-3"	15'-4"	14'-3"
11-7/8"	*110*	20'-2"	18'-5"	17'-4"	15'-9"**
11-7/8"	*210*	21'-1"	19'-3"	18'-2"	16'-11"
11-7/8"	*360*	22'-11"	20'-11"	19'-8"	18'-4"

LPI Engineered Floor Joist Span Tables: 15DL + 40LL + L/480					
Depth	Series	12" o.c.	16" o.c.	19.2" o.c.	24" o.c.
9 1/2"	*LPI 20Plus*	17'-9"	16'-2"	15'-3"	14'-3"
9 1/2"	*LPI32Plus*	18'-9"	17'-0"	16'-0"	14'-9"
11-7/8"	*LPI 20Plus*	21'-2"	19'-4"	18'-3"	17'-0"
11-7/8"	*LPI32Plus*	22'-3"	20'-2"	19'-0"	17'-7"

Roseberg Engineered Floor Joist Span Tables: 20DL + 40LL + L/480					
Depth	Series	12" o.c.	16" o.c.	19.2" o.c.	24" o.c.
9 1/2"	*RFPI 20*	16'-7"	15'-2"	14'-4"	12'-10"
9 1/2"	*RFPI 400*	18'-0"	16'-5"	15'-6"	14'-6"
11-7/8"	*RFPI 20*	19'-10"	17'-11"	16'-4"	13'-8"
11-7/8"	*RFPI 400*	21'-5"	19'-7"	18'-6"	16'-10"

1-1/4" Rimboard around perimeter of all floors.
Install per manufacturers specifications.
Equivalent engineered floor joists may be substituted based on published information.
**Web stiffener is required at intermediate support when bearing length is less than 5¼"

Floor Beams:

See attached beam calculations.

Floor Sheathing:

Provide 3/4" T&G APA rated Sturd-I-Floor sheathing.
Glue and nail with 10d common at 6" o.c. at panel edges and 12" o.c. in the field.

ENSIGN
THE STANDARD IN ENGINEERING

Project:	Blake Kingsbury and Merrit Chesson	Project No.:	8332
By:	Alex Hawkins, PE	Checked By:	DAJ
Date:	June 2018		

STUD COLUMN DESIGN

NDS 2015 EDITION *IBC 2015 / ASCE 7-10*

Species =	DFLN Stud	
Height =	11.0	ft
Fc =	900	psi
E =	1400	ksi
Kce =	0.3	psi
c =	0.8	psi

Size =	2x4	2x6	
d =	3.50	5.25	in
Fce =	295.28	664.39	psi
Cp =	0.30	0.58	
F'c =	271.77	521.08	psi

Height	(2) 2x4	(3) 2x4	(4) 2x4	(5) 2x4	(6) 2x4	(7) 2x4	
11 ft	2.9	4.3	5.7	7.1	8.6	10.0	kips
9 ft	4.0	6.0	8.1	10.1	12.1	14.1	kips
10 ft	3.4	5.1	6.8	8.4	10.1	11.8	kips
12 ft	2.4	3.7	4.9	6.1	7.3	8.5	kips
18 ft	1.1	1.7	2.3	2.8	3.4	3.9	kips

Height	(2) 2x6	(3) 2x6	(4) 2x6	(5) 2x6	(6) 2x6	(7) 2x6	
11 ft	8.2	12.3	16.4	20.5	24.6	28.7	kips
9 ft	10.3	15.4	20.5	25.6	30.8	35.9	kips
10 ft	9.2	13.8	18.4	23.1	27.7	32.3	kips
12 ft	7.3	10.9	14.5	18.2	21.8	25.4	kips
18 ft	3.7	5.5	7.3	9.1	11.0	12.8	kips

SOLID POSTS

Species =	DFLN #1	
Height =	14.0	ft
Fc =	925	psi
E =	1600	ksi
Kce =	0.3	psi
c =	0.8	psi

Size =	4x4	4X6	6x6	
d =	3.5	3.5	5.5	in
Fce =	208.33	208.33	514.46	psi
Cp =	0.214	0.214	0.472	
F'c =	197.60	197.60	436.47	psi

Height	4x4	4X6	6x6	
14 ft	2.4	3.8	13.2	kips
9 ft	5.6	8.8	21.9	kips
10 ft	4.7	7.3	20.1	kips
12 ft	3.3	5.2	16.5	kips
18 ft	1.5	2.4	8.6	kips

STANDARD FOUNDATION WALLS

ACI 318-14 *IBC 2015 / ASCE 7-10*

Foundation Schedule			Horizontal Reinforcement		Vertical Reinforcement	
Mark	Wall Height	Thickness	Size	Spacing	Size	Spacing
Typ.	4'	8"	#4	18"	#4	24"
Typ.	8'	8"	#4	18"	#4	24"
Typ.	9'	8"	#4	18"	#4	16"
Typ.	10'	8"	#4	18"	#5	12"

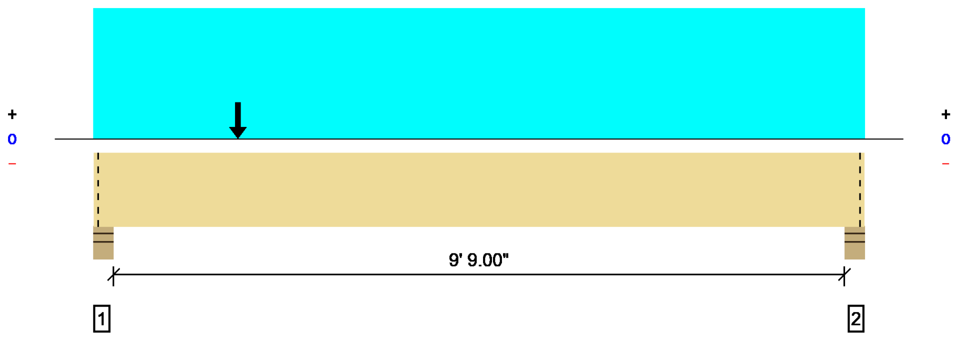
Notes: Wall height refers to final grade difference through the wall. Total height of wall may be higher due to footing drop for frost protection or native soil bearing as long as wall is backfilled such that the grade difference does not exceed the wall height at any time during construction.
ALL REBAR TO BE GRADE 60.
Place vertical bars in the center of wall.
Extend vertical bars from the footing to within 3" of the top of wall.
Provide #4 dowel with standard hook in the footing to match the vertical rebar.
Extend vertical leg of dowel 24" min. into wall.
Place (1) #4 horizontally within 4" of top and bottom of wall.
Provide corner reinforcing so as to lap 24" min.
Provide (2) #4 above, (1) #4 each side, and (1) #4 below all openings.
Place steel within 2" of openings & extend 24" min. beyond edge of opening.
Vertical bars around openings may terminate 3" from top of wall.

FORTE® **MEMBER REPORT** *Level, Below SW on Upper Floor Near MB12*

3 piece(s) 1 3/4" x 11 7/8" 2.0E Microllam® LVL

This product failed due to an excessive uplift of -16402 lbs at support located at 4.00".
This product failed due to an excessive uplift of -3166 lbs at support located at 10' 4.00".

Overall Length: 10' 8.00"



All locations are measured from the outside face of left support (or left cantilever end).All dimensions are horizontal.

Design Results	Actual @ Location	Allowed	Result	LDF	Load: Combination (Pattern)
Member Reaction (lbs)	16785 @ 4.00"	18047 (5.50")	Passed (93%)	--	1.0 D + 0.7 E (All Spans)
Shear (lbs)	16720 @ 1' 5.38"	18953	Passed (88%)	1.60	1.0 D + 0.7 E (All Spans)
Moment (Ft-lbs)	27888 @ 2' 0.00"	42836	Passed (65%)	1.60	1.0 D + 0.7 E (All Spans)
Live Load Defl. (in)	-0.274 @ 4' 8.38"	0.333	Passed (L/437)	--	0.6 D - 0.7 E (All Spans)
Total Load Defl. (in)	0.282 @ 4' 8.59"	0.500	Passed (L/425)	--	1.0 D + 0.7 E (All Spans)

System : Floor
Member Type : Flush Beam
Building Use : Residential
Building Code : IBC 2015
Design Methodology : ASD

- Deflection criteria: LL (L/360) and TL (L/240).
- Top Edge Bracing (Lu): Top compression edge must be braced at 9' 6.00" o/c unless detailed otherwise.
- Bottom Edge Bracing (Lu): Bottom compression edge must be braced at 9' 9.00" o/c unless detailed otherwise.

Supports	Bearing Length			Loads to Supports (lbs)				Accessories
	Total	Available	Required	Dead	Floor Live	Seismic	Total	
1 - Stud wall - DF	5.50"	5.50"	5.12"	239	284	23637/-23637	24160/-23637	Blocking
2 - Stud wall - DF	5.50"	5.50"	1.50"	239	284	4727/-4727	5250/-4727	Blocking

- Blocking Panels are assumed to carry no loads applied directly above them and the full load is applied to the member being designed.

Loads	Location (Side)	Tributary Width	Dead (0.90)	Floor Live (1.00)	Seismic (1.60)	Comments
0 - Self Weight (PLF)	0 to 10' 8.00"	N/A	18.2			
1 - Uniform (PSF)	0 to 10' 8.00" (Front)	1' 4.00"	20.0	40.0	-	Residential - Living Areas
2 - Point (lb)	2' 0.00" (Front)	N/A	-	-	28364	

Weyerhaeuser Notes

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The product application, input design loads, dimensions and support information have been provided by Forte Software Operator

Forte Software Operator	Job Notes
Ensign Engineering Ensign Engineering (801) 255-0529 ensign@ensignutah.com	

FAILED

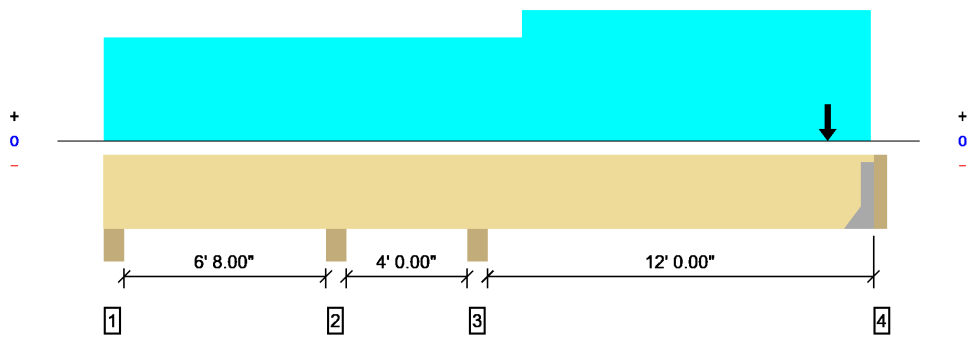
FORTE®

MEMBER REPORT *Level, MB11*

3 piece(s) 1 3/4" x 11 7/8" 2.0E Microllam® LVL

This product failed due to an excessive uplift of -3247 lbs at support located at 7' 4.25".
This product failed due to an excessive uplift of -5018 lbs at support located at 11' 9.75".
This product failed due to an excessive uplift of -16627 lbs at support located at 24' 0.50".

Overall Length: 24' 4.00"



All locations are measured from the outside face of left support (or left cantilever end).All dimensions are horizontal.

Design Results	Actual @ Location	Allowed	Result	LDF	Load: Combination (Pattern)
Member Reaction (lbs)	18596 @ 24' 0.50"	18596 (4.72")	Passed (100%)	--	1.0 D + 0.7 E (All Spans)
Shear (lbs)	18390 @ 23' 0.62"	18953	Passed (97%)	1.60	1.0 D + 0.7 E (All Spans)
Moment (Ft-lbs)	28434 @ 22' 6.00"	42836	Passed (66%)	1.60	1.0 D + 0.7 E (All Spans)
Live Load Defl. (in)	0.287 @ 19' 3.13"	0.306	Passed (L/512)	--	1.0 D + 0.525 E + 0.75 L + 0.75 S (Alt Spans)
Total Load Defl. (in)	0.343 @ 19' 1.48"	0.611	Passed (L/428)	--	1.0 D + 0.525 E + 0.75 L + 0.75 S (Alt Spans)

System : Floor
Member Type : Flush Beam
Building Use : Residential
Building Code : IBC 2015
Design Methodology : ASD

- Deflection criteria: LL (L/480) and TL (L/240).
- Top Edge Bracing (Lu): Top compression edge must be braced at 9' 2.00" o/c unless detailed otherwise.
- Bottom Edge Bracing (Lu): Bottom compression edge must be braced at 16' 7.00" o/c unless detailed otherwise.

Supports	Bearing Length			Loads to Supports (lbs)				Accessories
	Total	Available	Required	Dead	Floor Live	Seismic	Total	
1 - Column - DF	5.50"	5.50"	1.50"	760	1437/-46	473/-473	2670/-519	None
2 - Column - DF	5.50"	5.50"	1.50"	489	2837/-1060	5057/-5057	8383/-6117	None
3 - Column - DF	5.50"	5.50"	3.21"	3028	6001	9764/-9764	18793/-9764	None
4 - Hanger on 11 7/8" DF beam	3.50"	Hanger[1]	4.72"	1231	2298/-17	24807/-24807	28336/-24824	See note [1]

- At hanger supports, the Total Bearing dimension is equal to the width of the material that is supporting the hanger
- [1] See Connector grid below for additional information and/or requirements.

Connector: Simpson Strong-Tie Connectors

Support	Model	Seat Length	Top Nails	Face Nails	Member Nails	Accessories
4 - Face Mount Hanger	Connector not found	N/A	N/A	N/A	N/A	

Loads	Location (Side)	Tributary Width	Dead (0.90)	Floor Live (1.00)	Seismic (1.60)	Comments
0 - Self Weight (PLF)	0 to 24' 0.50"	N/A	18.2			
1 - Uniform (PSF)	13' 0.00" to 23' 10.00" (Front)	12' 0.00"	20.0	40.0	-	Residential - Living Areas
2 - Point (lb)	22' 6.00" (Front)	N/A	-	-	29987	
3 - Uniform (PSF)	0 to 13' 0.00" (Front)	9' 6.00"	20.0	40.0	-	Residential - Living Areas

Weyerhaeuser Notes

The product application, input design loads, dimensions and support information have been provided by Forte Software Operator

SUSTAINABLE FORESTRY INITIATIVE

Forte Software Operator	Job Notes
Ensign Engineering Ensign Engineering (801) 255-0529 ensign@ensignutah.com	

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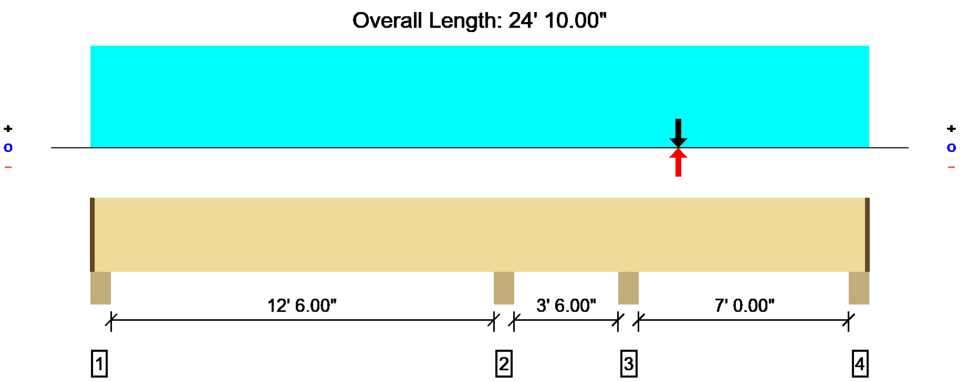
Forte v5.4, Design Engine: V7.1.1.3
Forte.4te

FORTE® **MEMBER REPORT** *Level, MB12*

3 piece(s) 1 3/4" x 11 7/8" 2.0E Microllam® LVL

This product failed due to an excessive uplift of -2356 lbs at support located at 13' 1.00".
This product failed due to an excessive uplift of -17861 lbs at support located at 17' 0.50".
This product failed due to an excessive uplift of -1511 lbs at support located at 24' 4.75".



All locations are measured from the outside face of left support (or left cantilever end).All dimensions are horizontal.

Design Results	Actual @ Location	Allowed	Result	LDF	Load: Combination (Pattern) [Group]
Member Reaction (lbs)	20807 @ 17' 1.75"	21656 (5.50")	Passed (96%)	--	1.0 D - 0.7 E (All Spans) [5]
Shear (lbs)	16964 @ 18' 4.38"	18953	Passed (90%)	1.60	1.0 D - 0.7 E (All Spans) [5]
Moment (Ft-lbs)	-9004 @ 13' 2.25"	26772	Passed (34%)	1.00	1.0 D + 1.0 L (Adj Spans) [5]
Live Load Defl. (in)	0.082 @ 20' 6.87"	0.184	Passed (L/999+)	--	1.0 D - 0.525 E + 0.75 L + 0.75 S (Alt Spans) [5]
Total Load Defl. (in)	0.097 @ 20' 7.33"	0.368	Passed (L/909)	--	1.0 D - 0.525 E + 0.75 L + 0.75 S (Alt Spans) [5]

System : Floor
Member Type : Flush Beam
Building Use : Residential
Building Code : IBC 2015
Design Methodology : ASD

- Deflection criteria: LL (L/480) and TL (L/240).
- Top Edge Bracing (Lu): Top compression edge must be braced at 22' 10.00" o/c unless detailed otherwise.
- Bottom Edge Bracing (Lu): Bottom compression edge must be braced at 20' 11.00" o/c unless detailed otherwise.

Supports	Bearing Length			Loads to Supports (lbs)				Accessories
	Total	Available	Required	Dead	Floor Live	Seismic	Total	
1 - Column - DF	5.50"	4.25"	1.50"	1100	2013	169/-169	3282/-169	1 1/4" Rim Board
2 - Column - DF	5.50"	5.50"	2.21"	2332	5191/-265	5364/-5364	12887/-5629	None
3 - Column - DF	5.50"	5.50"	5.28"	1841	5273/-1097	27093/-27093	34207/-28190	None
4 - Column - DF	5.50"	4.25"	1.50"	876	1632	2909/-2909	5417/-2909	1 1/4" Rim Board

- Rim Board is assumed to carry all loads applied directly above it, bypassing the member being designed.

Loads	Location (Side)	Tributary Width	Dead (0.90)	Floor Live (1.00)	Seismic (1.60)	Comments
0 - Self Weight (PLF)	1.25" to 24' 8.75"	N/A	18.2			
1 - Uniform (PSF)	0 to 24' 10.00" (Front)	9' 0.00"	20.0	40.0	-	Residential - Living Areas
2 - Point (lb)	18' 9.00" (Front)	N/A	1231	2298/-17	24807/-24807	Linked from: MB11, Support 4

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SUSTAINABLE FORESTRY INITIATIVE

Forte Software Operator	Job Notes
Ensign Engineering Ensign Engineering (801) 255-0529 ensign@ensignutah.com	

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Forte v5.4, Design Engine: V7.1.1.3
Forte.4te

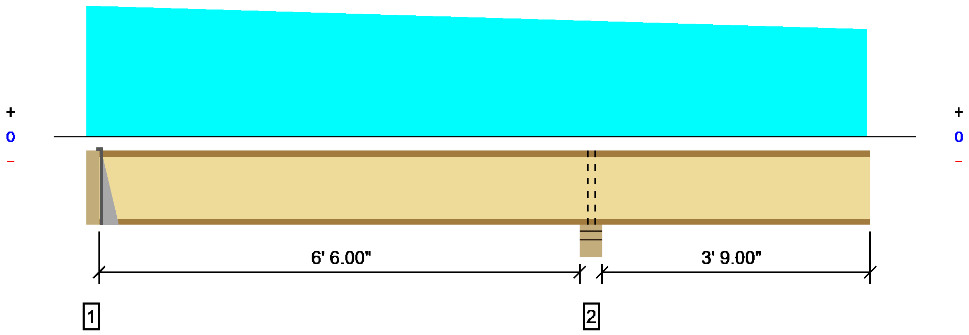
FAILED

FORTE® MEMBER REPORT *Level, Cantilevered Joists Snow*

1 piece(s) 11 7/8" TJI® 230 @ 16" OC

Right cantilever length exceeds 1/3 member length or 1/2 back span length.

Overall Length: 11' 0.50"



All locations are measured from the outside face of left support (or left cantilever end).All dimensions are horizontal.;Drawing is Conceptual

Design Results	Actual @ Location	Allowed	Result	LDF	Load: Combination (Pattern)
Member Reaction (lbs)	3263 @ 7' 0.50"	3209 (5.25")	Passed (102%)	1.15	1.0 D + 1.0 S (All Spans)
Shear (lbs)	1375 @ 7' 3.50"	1903	Passed (72%)	1.15	1.0 D + 1.0 S (All Spans)
Moment (Ft-lbs)	-2896 @ 7' 0.50"	4847	Passed (60%)	1.15	1.0 D + 1.0 S (All Spans)
Live Load Defl. (in)	0.151 @ 11' 0.50"	0.200	Passed (2L/638)	--	1.0 D + 1.0 S (Alt Spans)
Total Load Defl. (in)	0.160 @ 11' 0.50"	0.400	Passed (2L/600)	--	1.0 D + 1.0 S (Alt Spans)
TJ-Pro™ Rating	68	40	Passed	--	--

System : Floor
Member Type : Joist
Building Use : Residential
Building Code : IBC 2015
Design Methodology : ASD

- Deflection criteria: LL (L/480) and TL (L/240).
- Overhang deflection criteria: LL (0.2") and TL (2L/240).
- Top Edge Bracing (Lu): Top compression edge must be braced at 6' 9.00" o/c unless detailed otherwise.
- Bottom Edge Bracing (Lu): Bottom compression edge must be braced at 5' 0.00" o/c unless detailed otherwise.
- A structural analysis of the deck has not been performed.
- Deflection analysis is based on composite action with a single layer of 23/32" Weyerhaeuser Edge™ Panel (24" Span Rating) that is glued and nailed down.
- Additional considerations for the TJ-Pro™ Rating include: None

Supports	Bearing			Loads to Supports (lbs)				Accessories
	Total	Available	Required	Dead	Floor Live	Snow	Total	
1 - Hanger on 11 7/8" DF ledger	3.50"	Hanger[1]	1.75" / - [2]	66	293/-71	1261	1620/-71	See note [1]
2 - Stud wall - DF	6.00"	6.00"	5.25"	228	685	3035	3948	Blocking

- Blocking Panels are assumed to carry no loads applied directly above them and the full load is applied to the member being designed.
- At hanger supports, the Total Bearing dimension is equal to the width of the material that is supporting the hanger
- [1] See Connector grid below for additional information and/or requirements.
- [2] Required Bearing Length / Required Bearing Length with Web Stiffeners

Connector: Simpson Strong-Tie Connectors

Support	Model	Seat Length	Top Nails	Face Nails	Member Nails	Accessories
1 - Top Mount Hanger	ITS2.37/11.88	2.00"	4-10d x 1-1/2	2-10d x 1-1/2	N/A	

Loads	Location (Side)	Spacing	Dead (0.90)	Floor Live (1.00)	Snow (1.15)	Comments
1 - Uniform (PSF)	0 to 11' 0.50"	16"	20.0	60.0	-	Residential - Living Areas
2 - Tapered (PLF)	0 to 11' 0.00"	N/A	-	-	418.0 to 324.0	

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SUSTAINABLE FORESTRY INITIATIVE

Forte Software Operator	Job Notes
Ensign Engineering Ensign Engineering (801) 255-0529 ensign@ensignutah.com	

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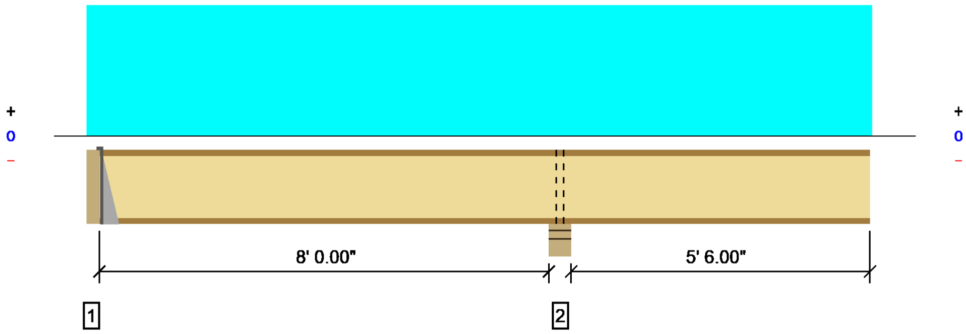
Forte v5.3, Design Engine: V7.0.0.5
Forte.4te

1 piece(s) 11 7/8" TJI® 230 @ 16" OC

Right cantilever length exceeds 1/3 member length or 1/2 back span length.
Right overhang exceeds the maximum length of 5' 0.00" for this product.

Overall Length: 14' 3.50"



All locations are measured from the outside face of left support (or left cantilever end).All dimensions are horizontal.;Drawing is Conceptual

Design Results	Actual @ Location	Allowed	Result	LDF	Load: Combination (Pattern)
Member Reaction (lbs)	950 @ 8' 6.50"	2790 (5.25")	Passed (34%)	1.00	1.0 D + 1.0 L (All Spans)
Shear (lbs)	440 @ 8' 9.50"	1655	Passed (27%)	1.00	1.0 D + 1.0 L (All Spans)
Moment (Ft-lbs)	-1323 @ 8' 6.50"	3161	Passed (42%)	1.00	1.0 D + 1.0 L (All Spans)
Live Load Defl. (in)	0.103 @ 14' 3.50"	0.287	Passed (2L/999+)	--	1.0 D + 1.0 L (Alt Spans)
Total Load Defl. (in)	0.140 @ 14' 3.50"	0.575	Passed (2L/988)	--	1.0 D + 1.0 L (Alt Spans)
TJ-Pro™ Rating	66	40	Passed	--	--

System : Floor
Member Type : Joist
Building Use : Residential
Building Code : IBC 2015
Design Methodology : ASD

- Deflection criteria: LL (L/480) and TL (L/240).
- Overhang deflection criteria: LL (2L/480) and TL (2L/240).
- Moment capacity over cantilever support 2 has been reduced by 25% to lessen the effects of buckling.
- Top Edge Bracing (Lu): Top compression edge must be braced at 9' 7.00" o/c unless detailed otherwise.
- Bottom Edge Bracing (Lu): Bottom compression edge must be braced at 7' 6.00" o/c unless detailed otherwise.
- A structural analysis of the deck has not been performed.
- Deflection analysis is based on composite action with a single layer of 23/32" Weyerhaeuser Edge™ Panel (24" Span Rating) that is glued and nailed down.
- Additional considerations for the TJ-Pro™ Rating include: None

Supports	Bearing			Loads to Supports (lbs)			Accessories
	Total	Available	Required	Dead	Floor Live	Total	
1 - Hanger on 11 7/8" DF ledger	3.50"	Hanger[1]	1.75" / - [2]	64	236/-91	300/-91	See note [1]
2 - Stud wall - DF	6.00"	6.00"	3.50"	317	634	951	Blocking

- Blocking Panels are assumed to carry no loads applied directly above them and the full load is applied to the member being designed.
- At hanger supports, the Total Bearing dimension is equal to the width of the material that is supporting the hanger
- [1] See Connector grid below for additional information and/or requirements.
- [2] Required Bearing Length / Required Bearing Length with Web Stiffeners

Connector: Simpson Strong-Tie Connectors						
Support	Model	Seat Length	Top Nails	Face Nails	Member Nails	Accessories
1 - Top Mount Hanger	ITS2.37/11.88	2.00"	4-10d x 1-1/2	2-10d x 1-1/2	N/A	

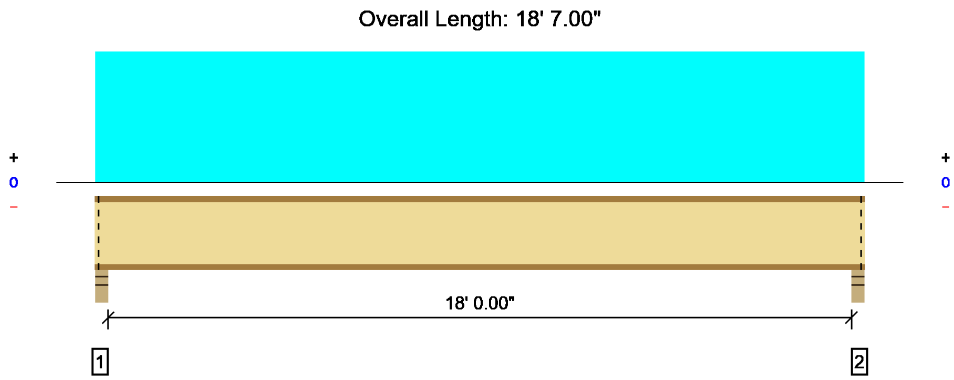
Loads	Location (Side)	Spacing	Dead (0.90)	Floor Live (1.00)	Comments
1 - Uniform (PSF)	0 to 14' 3.50"	16"	20.0	40.0	Residential - Living Areas

Weyerhaeuser Notes

Forte Software Operator	Job Notes
Ensign Engineering Ensign Engineering (801) 255-0529 ensign@ensignutah.com	

FORTE® **MEMBER REPORT** *Level, Typ. Joists*

1 piece(s) 11 7/8" TJI® 230 @ 16" OC



All locations are measured from the outside face of left support (or left cantilever end).All dimensions are horizontal.;Drawing is Conceptual

Design Results	Actual @ Location	Allowed	Result	LDF	Load: Combination (Pattern)
Member Reaction (lbs)	743 @ 2.50"	1485 (3.50")	Passed (50%)	1.00	1.0 D + 1.0 L (All Spans)
Shear (lbs)	720 @ 3.50"	1655	Passed (44%)	1.00	1.0 D + 1.0 L (All Spans)
Moment (Ft-lbs)	3300 @ 9' 3.50"	4215	Passed (78%)	1.00	1.0 D + 1.0 L (All Spans)
Live Load Defl. (in)	0.350 @ 9' 3.50"	0.454	Passed (L/623)	--	1.0 D + 1.0 L (All Spans)
Total Load Defl. (in)	0.525 @ 9' 3.50"	0.908	Passed (L/415)	--	1.0 D + 1.0 L (All Spans)
TJ-Pro™ Rating	40	40	Passed	--	--

System : Floor
Member Type : Joist
Building Use : Residential
Building Code : IBC 2015
Design Methodology : ASD

- Deflection criteria: LL (L/480) and TL (L/240).
- Top Edge Bracing (Lu): Top compression edge must be braced at 4' 7.00" o/c unless detailed otherwise.
- Bottom Edge Bracing (Lu): Bottom compression edge must be braced at 18' 7.00" o/c unless detailed otherwise.
- A structural analysis of the deck has not been performed.
- Deflection analysis is based on composite action with a single layer of 23/32" Weyerhaeuser Edge™ Panel (24" Span Rating) that is glued and nailed down.
- Additional considerations for the TJ-Pro™ Rating include: None

Supports	Bearing			Loads to Supports (lbs)			Accessories
	Total	Available	Required	Dead	Floor Live	Total	
1 - Stud wall - DF	3.50"	3.50"	1.75"	248	496	744	Blocking
2 - Stud wall - DF	3.50"	3.50"	1.75"	248	496	744	Blocking

- Blocking Panels are assumed to carry no loads applied directly above them and the full load is applied to the member being designed.

Loads	Location (Side)	Spacing	Dead (0.90)	Floor Live (1.00)	Comments
1 - Uniform (PSF)	0 to 18' 7.00"	16"	20.0	40.0	Residential - Living Areas

Weyerhaeuser Notes

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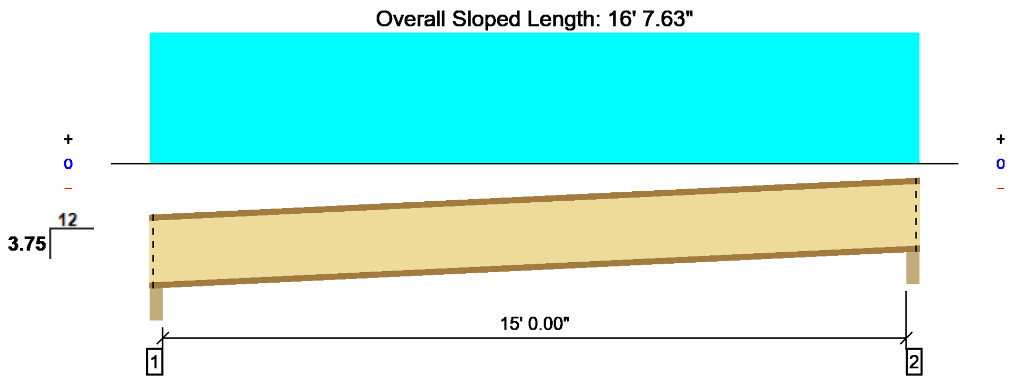
Forte Software Operator	Job Notes
Ensign Engineering Ensign Engineering (801) 255-0529 ensign@ensignutah.com	

FORTE® **MEMBER REPORT** *Level, Roof Joists* **PASSED**

1 piece(s) 11 7/8" TJI® 360 @ 12" OC



All locations are measured from the outside face of left support (or left cantilever end).All dimensions are horizontal.;Drawing is Conceptual

Design Results	Actual @ Location	Allowed	Result	LDF	Load: Combination (Pattern)
Member Reaction (lbs)	1628 @ 2.50"	1731 (3.50")	Passed (94%)	1.15	1.0 D + 1.0 S (All Spans)
Shear (lbs)	1567 @ 15' 3.50"	1961	Passed (80%)	1.15	1.0 D + 1.0 S (All Spans)
Moment (Ft-lbs)	6008 @ 7' 9.50"	7107	Passed (85%)	1.15	1.0 D + 1.0 S (All Spans)
Live Load Defl. (in)	0.683 @ 7' 9.50"	0.794	Passed (L/279)	--	1.0 D + 1.0 S (All Spans)
Total Load Defl. (in)	0.760 @ 7' 9.50"	1.059	Passed (L/251)	--	1.0 D + 1.0 S (All Spans)

System : Roof
Member Type : Joist
Building Use : Residential
Building Code : IBC 2015
Design Methodology : ASD
Member Pitch: 3.75/12

- Deflection criteria: LL (L/240) and TL (L/180).
- Top Edge Bracing (Lu): Top compression edge must be braced at 3' 7.00" o/c unless detailed otherwise.
- Bottom Edge Bracing (Lu): Bottom compression edge must be braced at 16' 4.00" o/c unless detailed otherwise.

	Bearing			Loads to Supports (lbs)			
Supports	Total	Available	Required	Dead	Snow	Total	Accessories
1 - Beveled Plate - DF	3.50"	3.50"	3.13"	163	1465	1628	Blocking
2 - Beveled Plate - DF	3.50"	3.50"	3.13"	163	1465	1628	Blocking

- Blocking Panels are assumed to carry no loads applied directly above them and the full load is applied to the member being designed.

Loads	Location (Side)	Spacing	Dead (0.90)	Snow (1.15)	Comments
1 - Uniform (PSF)	0 to 15' 7.00"	12"	20.0	188.0	Roof Snow

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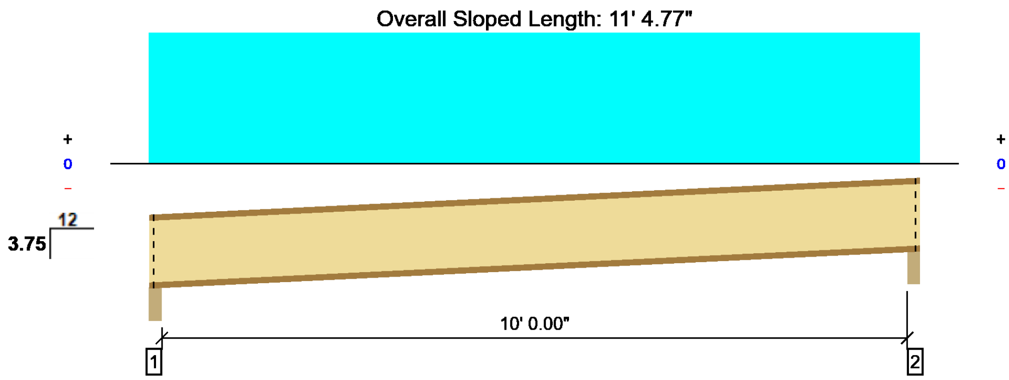
SUSTAINABLE FORESTRY INITIATIVE

Forte Software Operator	Job Notes
Ensign Engineering Ensign Engineering (801) 255-0529 ensign@ensignutah.com	

6/12/2018 11:05:59 AM
Forte v5.3, Design Engine: V7.0.0.5
Forte.4te

FORTE® **MEMBER REPORT** *Level, Roof Joists* PASSED

1 piece(s) 11 7/8" TJI® 230 @ 12" OC



All locations are measured from the outside face of left support (or left cantilever end).All dimensions are horizontal.;Drawing is Conceptual

Design Results	Actual @ Location	Allowed	Result	LDF	Load: Combination (Pattern)
Member Reaction (lbs)	1106 @ 2.50"	1708 (3.50")	Passed (65%)	1.15	1.0 D + 1.0 S (All Spans)
Shear (lbs)	1045 @ 10' 3.50"	1903	Passed (55%)	1.15	1.0 D + 1.0 S (All Spans)
Moment (Ft-lbs)	2700 @ 5' 3.50"	4847	Passed (56%)	1.15	1.0 D + 1.0 S (All Spans)
Live Load Defl. (in)	0.187 @ 5' 3.50"	0.533	Passed (L/685)	--	1.0 D + 1.0 S (All Spans)
Total Load Defl. (in)	0.207 @ 5' 3.50"	0.710	Passed (L/616)	--	1.0 D + 1.0 S (All Spans)

- Deflection criteria: LL (L/240) and TL (L/180).
- Top Edge Bracing (Lu): Top compression edge must be braced at 5' 2.00" o/c unless detailed otherwise.
- Bottom Edge Bracing (Lu): Bottom compression edge must be braced at 11' 1.00" o/c unless detailed otherwise.

System : Roof
Member Type : Joist
Building Use : Residential
Building Code : IBC 2015
Design Methodology : ASD
Member Pitch: 3.75/12

Supports	Bearing			Loads to Supports (lbs)			Accessories
	Total	Available	Required	Dead	Snow	Total	
1 - Beveled Plate - DF	3.50"	3.50"	1.75"	111	995	1106	Blocking
2 - Beveled Plate - DF	3.50"	3.50"	1.75"	111	995	1106	Blocking

- Blocking Panels are assumed to carry no loads applied directly above them and the full load is applied to the member being designed.

Loads	Location (Side)	Spacing	Dead (0.90)	Snow (1.15)	Comments
1 - Uniform (PSF)	0 to 10' 7.00"	12"	20.0	188.0	Roof Snow

Weyerhaeuser Notes

Weyerhaeuser warrants that the sizing of its products will be in accordance with Weyerhaeuser product design criteria and published design values. Weyerhaeuser expressly disclaims any other warranties related to the software. Use of this software is not intended to circumvent the need for a design professional as determined by the authority having jurisdiction. The designer of record, builder or framer is responsible to assure that this calculation is compatible with the overall project. Accessories (Rim Board, Blocking Panels and Squash Blocks) are not designed by this software. Products manufactured at Weyerhaeuser facilities are third-party certified to sustainable forestry standards. Weyerhaeuser Engineered Lumber Products have been evaluated by ICC ES under technical reports ESR-1153 and ESR-1387 and/or tested in accordance with applicable ASTM standards. For current code evaluation reports, Weyerhaeuser product literature and installation details refer to www.weyerhaeuser.com/woodproducts/document-library.

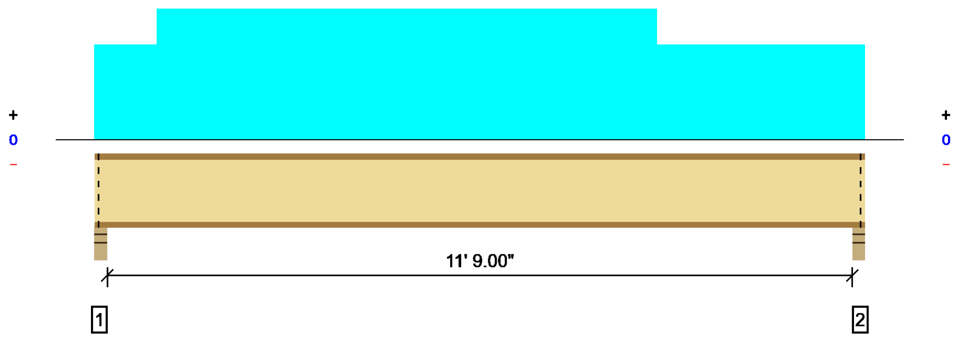
The product application, input design loads, dimensions and support information have been provided by Forte Software Operator

SUSTAINABLE FORESTRY INITIATIVE

Forte Software Operator	Job Notes
Ensign Engineering Ensign Engineering (801) 255-0529 ensign@ensignutah.com	

1 piece(s) 11 7/8" TJI® 230 @ 12" OC

Overall Length: 12' 4.00"



All locations are measured from the outside face of left support (or left cantilever end).All dimensions are horizontal.;Drawing is Conceptual

Design Results	Actual @ Location	Allowed	Result	LDF	Load: Combination (Pattern)
Member Reaction (lbs)	1606 @ 2.50"	1708 (3.50")	Passed (94%)	1.15	1.0 D + 0.75 L + 0.75 S (All Spans)
Shear (lbs)	1547 @ 3.50"	1903	Passed (81%)	1.15	1.0 D + 0.75 L + 0.75 S (All Spans)
Moment (Ft-lbs)	4728 @ 6' 0.76"	4847	Passed (98%)	1.15	1.0 D + 0.75 L + 0.75 S (All Spans)
Live Load Defl. (in)	0.398 @ 6' 1.66"	0.397	Passed (L/359)	--	1.0 D + 0.75 L + 0.75 S (All Spans)
Total Load Defl. (in)	0.431 @ 6' 1.69"	0.596	Passed (L/332)	--	1.0 D + 0.75 L + 0.75 S (All Spans)

- Deflection criteria: LL (L/360) and TL (L/240).
- Top Edge Bracing (Lu): Top compression edge must be braced at 3' 9.00" o/c unless detailed otherwise.
- Bottom Edge Bracing (Lu): Bottom compression edge must be braced at 12' 4.00" o/c unless detailed otherwise.

System : Roof
Member Type : Joist
Building Use : Residential
Building Code : IBC 2015
Design Methodology : ASD
Member Pitch: 0/12

Supports	Bearing			Loads to Supports (lbs)				Accessories
	Total	Available	Required	Dead	Floor Live	Snow	Total	
1 - Stud wall - DF	3.50"	3.50"	3.13"	123	725	1252	2100	Blocking
2 - Stud wall - DF	3.50"	3.50"	2.71"	123	568	1252	1943	Blocking

- Blocking Panels are assumed to carry no loads applied directly above them and the full load is applied to the member being designed.

Loads	Location (Side)	Spacing	Dead (0.90)	Floor Live (1.00)	Snow (1.15)	Comments
1 - Uniform (PSF)	0 to 12' 4.00"	12"	20.0	40.0	203.0	Roof Snow
2 - Uniform (PSF)	1' 0.00" to 9' 0.00"	12"	-	100.0	-	Hot tub

Weyerhaeuser Notes

Weyerhaeuser warrants that the sizing of its products will be in accordance with Weyerhaeuser product design criteria and published design values. Weyerhaeuser expressly disclaims any other warranties related to the software. Use of this software is not intended to circumvent the need for a design professional as determined by the authority having jurisdiction. The designer of record, builder or framer is responsible to assure that this calculation is compatible with the overall project. Accessories (Rim Board, Blocking Panels and Squash Blocks) are not designed by this software. Products manufactured at Weyerhaeuser facilities are third-party certified to sustainable forestry standards. Weyerhaeuser Engineered Lumber Products have been evaluated by ICC ES under technical reports ESR-1153 and ESR-1387 and/or tested in accordance with applicable ASTM standards. For current code evaluation reports, Weyerhaeuser product literature and installation details refer to www.weyerhaeuser.com/woodproducts/document-library.

The product application, input design loads, dimensions and support information have been provided by Forte Software Operator

SUSTAINABLE FORESTRY INITIATIVE

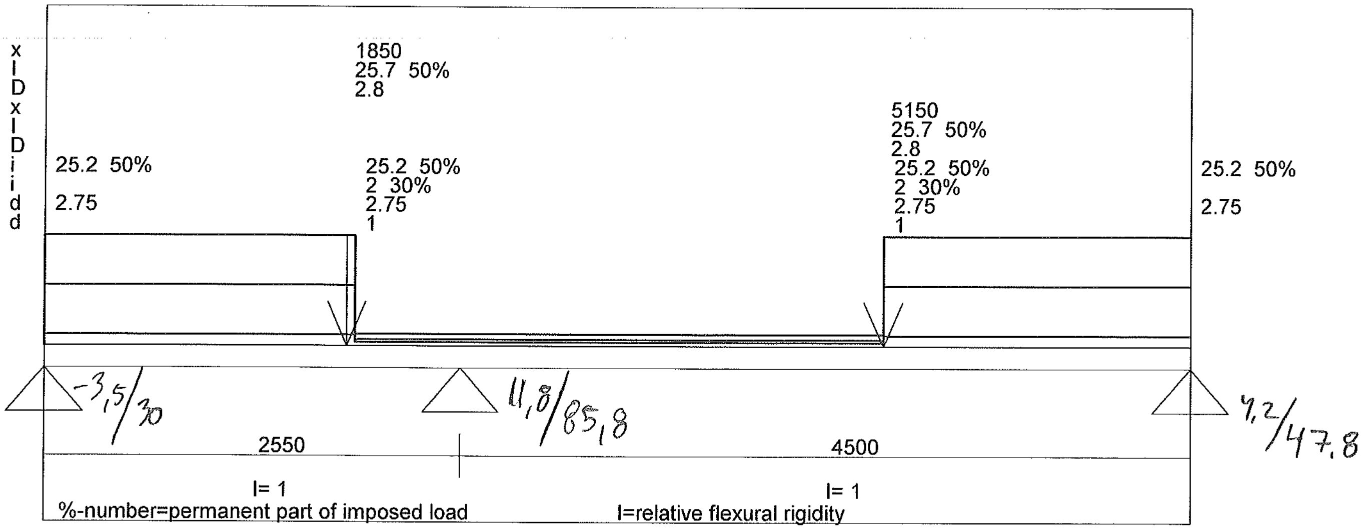
Forte Software Operator	Job Notes
Ensign Engineering Ensign Engineering (801) 255-0529 ensign@ensignutah.com	

Beam Id: Lot # 70 - NB

Date 18-05-2018

Structural Engineer:

Licensed to: FINNLAMELLI OY



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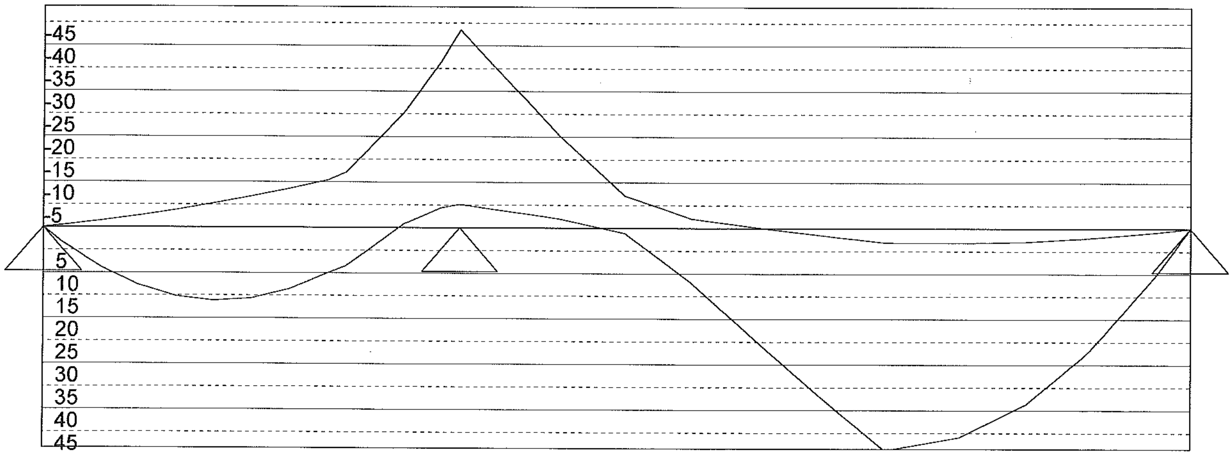
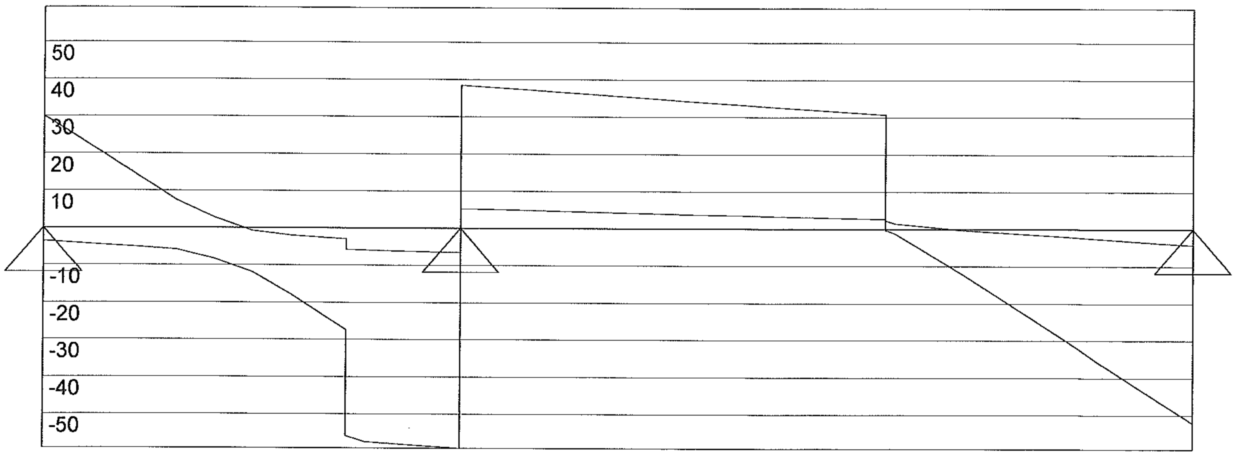


Licensed to: FINNLAMELLI OY



Load factor of dead load= 1 Load factor of imposed load= 1
Load width 1 (m) (by which the loads
has been multiplied during calculation)
Max/Min reactions of beam [kN]
30,043 97,664 52,083
-3,577 11,805 4,227

KER 133 x 406 B 2 Cf=0,97 Design method: Allowable stress design
Increasing factor of the allowable stress 1,03
Factored Moment/Moment capacity [kNm] 48,519 65,401 74 %
Factored shear force/shear capacity [kN] 59,212 62,936 94 %

(3) LVL 1 3/4 x 16" ✓

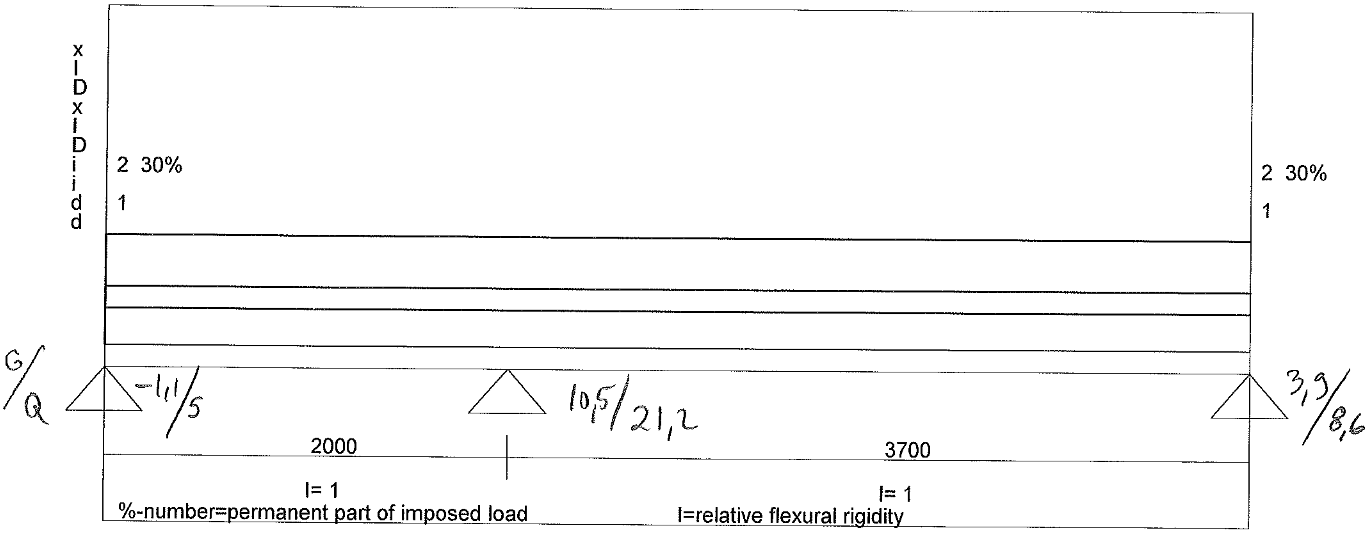
Deflection due to unfactored load (Deflection limit L/360)
1,2 mm (16 %) 9,1 mm (73 %)

Beam Id: Lot # 70 - MB 2

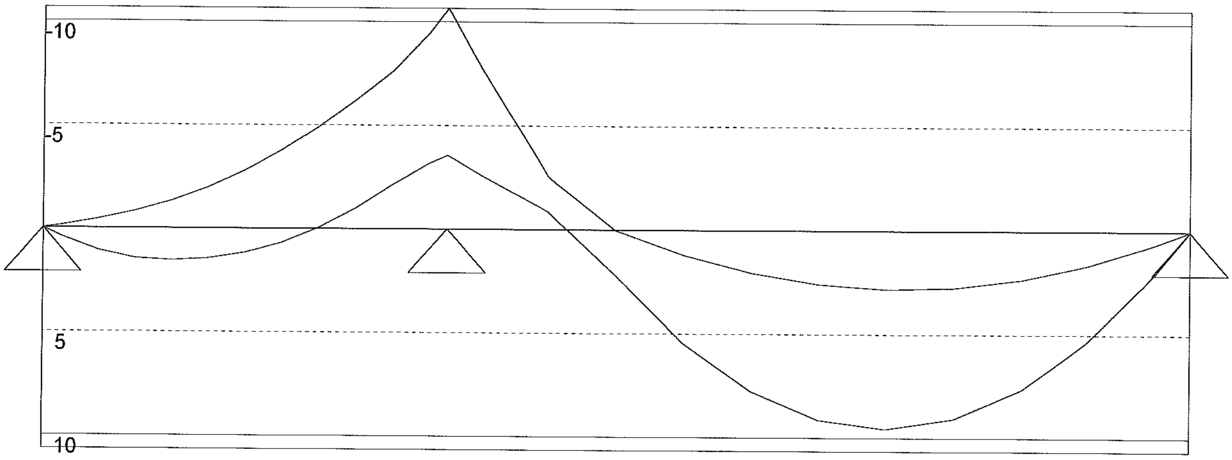
Date 18-05-2018

Structural Engineer:

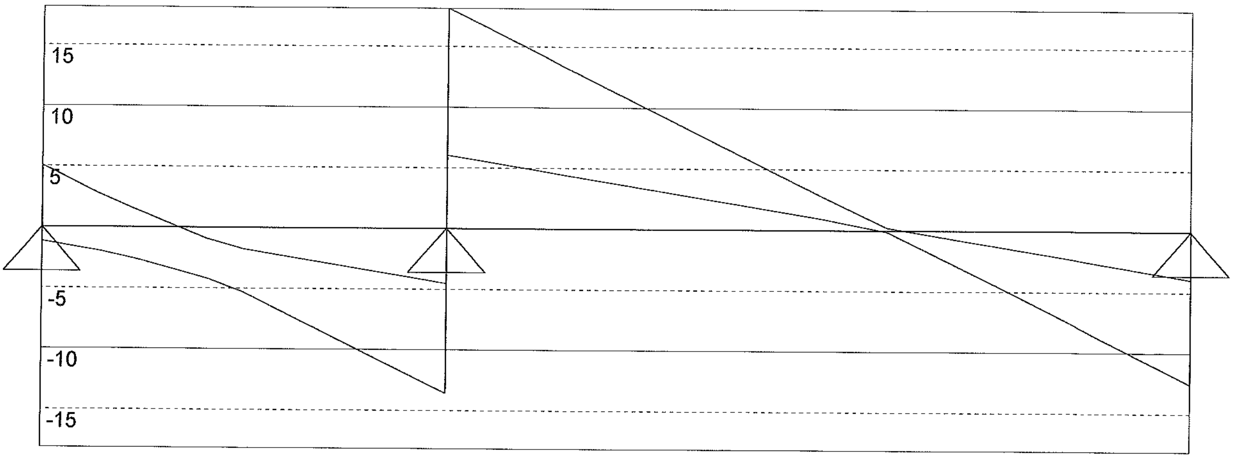
Licensed to: FINNLAMELLI OY



Licensed to: FINNLAMELLI OY



Licensed to: FINNLAMELLI OY



Load factor of dead load= 1 Load factor of imposed load= 1
Load width 2.75 (m) (by which the loads
has been multiplied during calculation)
Max/Min reactions of beam [kN]
5,069 31,713 12,567
-1,173 10,571 3,945

T24 164 x 260 B 2 Cf=1,00 Design method: Allowable stress design
Increasing factor of the allowable stress 1,09
Factored Moment/Moment capacity [kNm] 10,650 19,351 55 %
Factored shear force/shear capacity [kN] 18,138 29,770 61 %

(1) LOG ✓

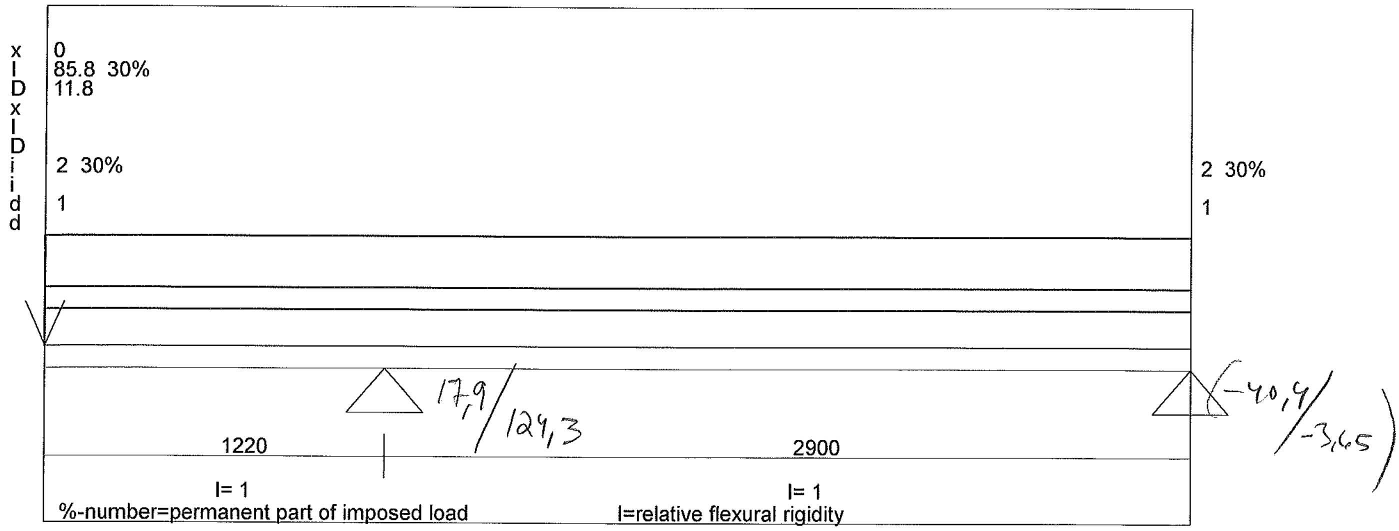
Deflection due to unfactored load (Deflection limit L/360)
0,2 mm (4 %) 7,5 mm (73 %)

Beam Id: Lot # 70 - MB 3

Date 18-05-2018

Structural Engineer:

Licensed to: FINNLAMELLI OY



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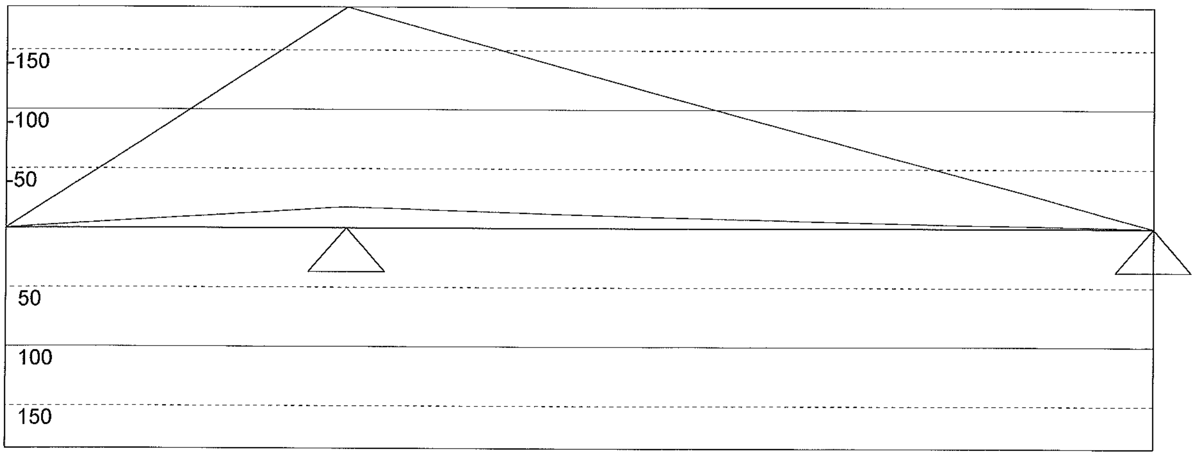

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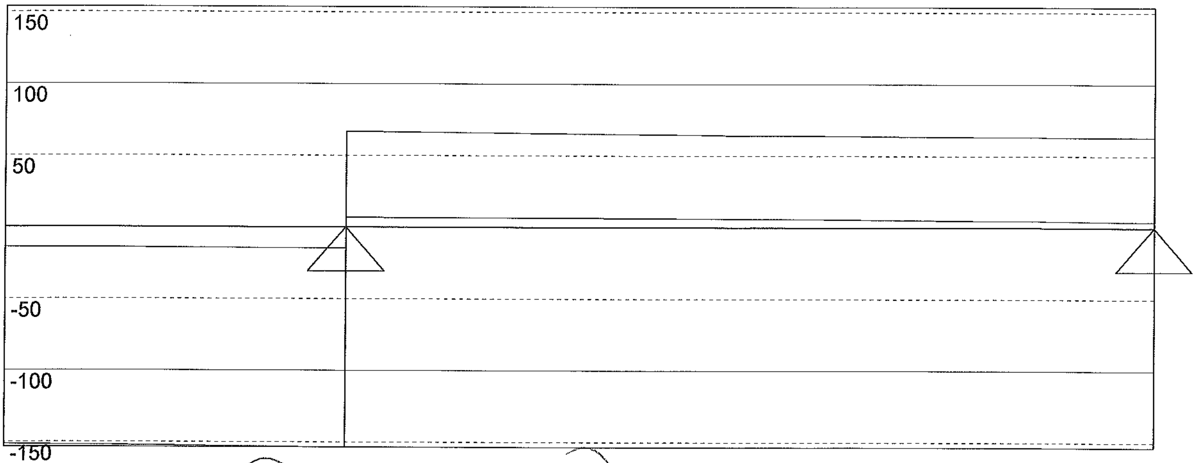

Load factor of dead load= 1.2 Load factor of imposed load= 1.6
Load width .406 (m) (by which the loads
has been multiplied during calculation)
Max/Min reactions of beam [kN]
220,373 -4,057
21,542 -62,893

HEB 260 (Class of section=1/1) G= 93 I(cm4)=14919 W(cm3)=1150 fy=235
Factored Moment/Moment capacity [kNm] 186,058 301,270 62 %
Factored shear force/shear capacity [kN] 153,619 341,925 45 %

W10 x 68 ✓

Deflection due to unfactored load (Deflection limit L/360)/L/180)!
6,4 mm (94 %) -0,2 mm (3 %)
Attention! Ultimate limit design! Remember the load factors!!

Beam Id: Lot # 70 - MB 4 | Date 22-05-2018

Structural Engineer:

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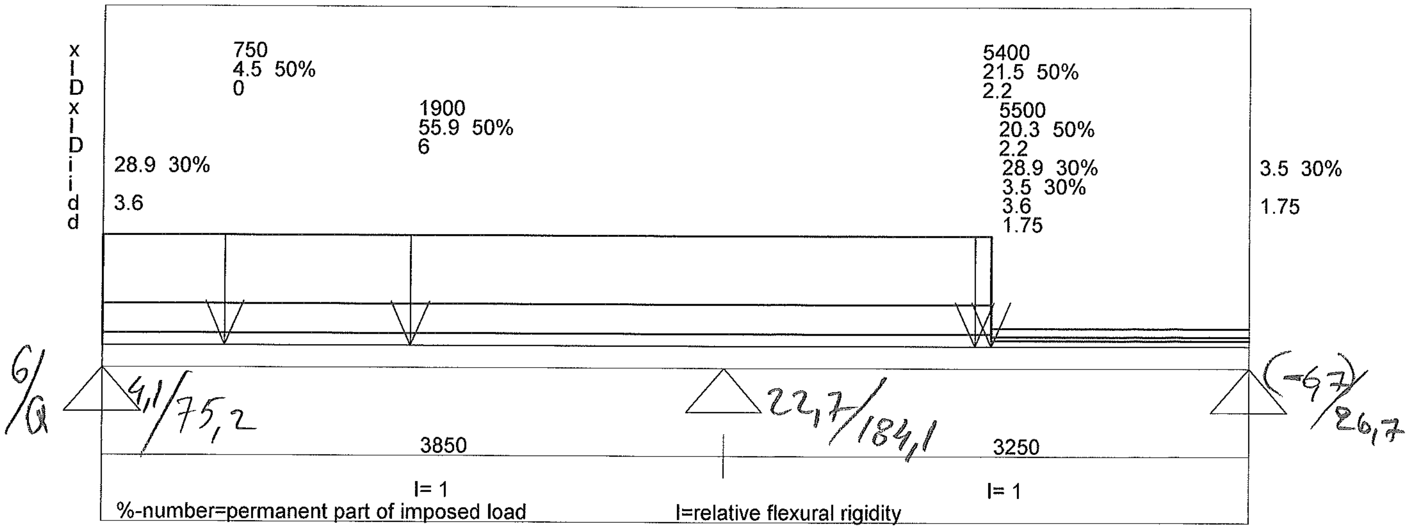

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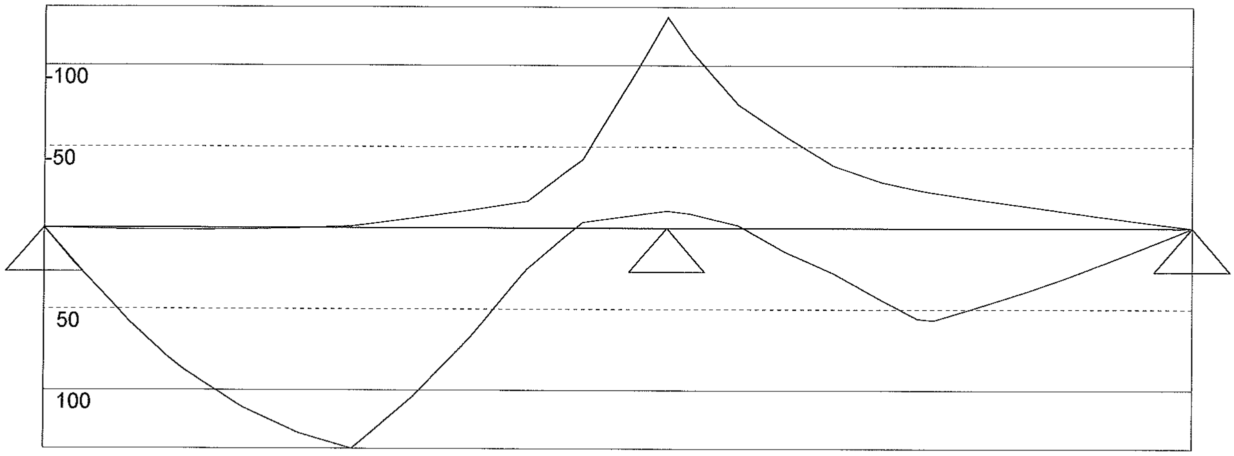

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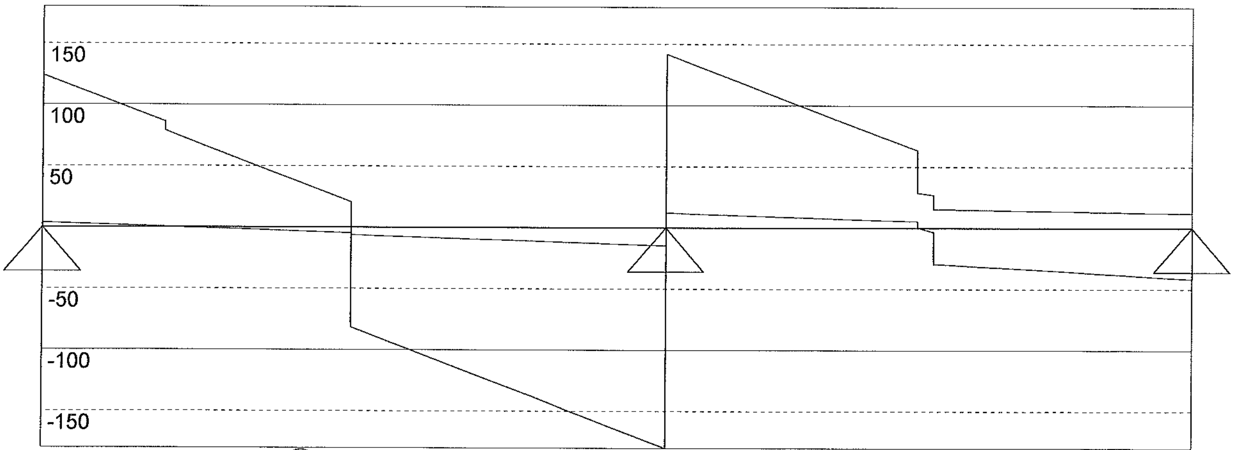

Load factor of dead load= 1.2 Load factor of imposed load= 1.6
Load width 1 (m) (by which the loads
has been multiplied during calculation)
Max/Min reactions of beam [kN]
123,932 322,000 41,617
3,559 26,755 -11,967

HEB 200 (Class of section=1/1) G= 61,3 I(cm4)=5696 W(cm3)=570 fy=235 W 8x40 ✓
Factored Moment/Moment capacity [kNm] 135,979 150,870 90 %
Factored shear force/shear capacity [kN] 180,123 234,765 77 %

Sum infl M+S 0,89 (must be<=1) x= 1900 M=135,98 S=75,93
Deflection due to unfactored load (Deflection limit L/360)
8,8 mm (82 %) 2,2 mm (25 %)
Attention! Ultimate limit design! Remember the load factors!!

Beam Id: Lot # 70 - MB 5

Date 22-05-2018

Structural Engineer:

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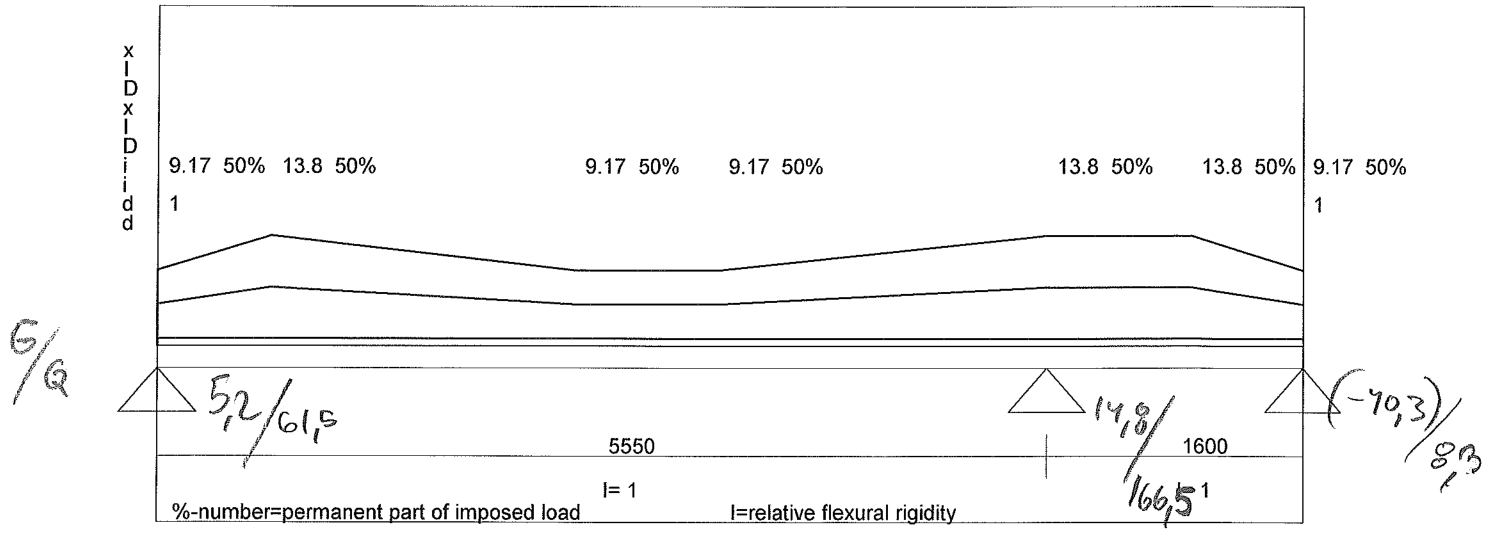

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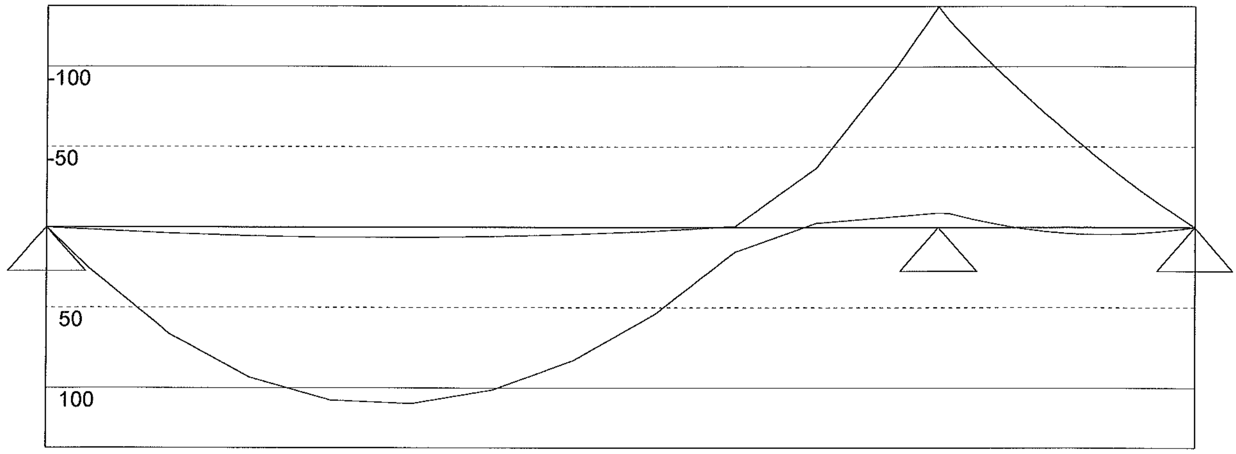

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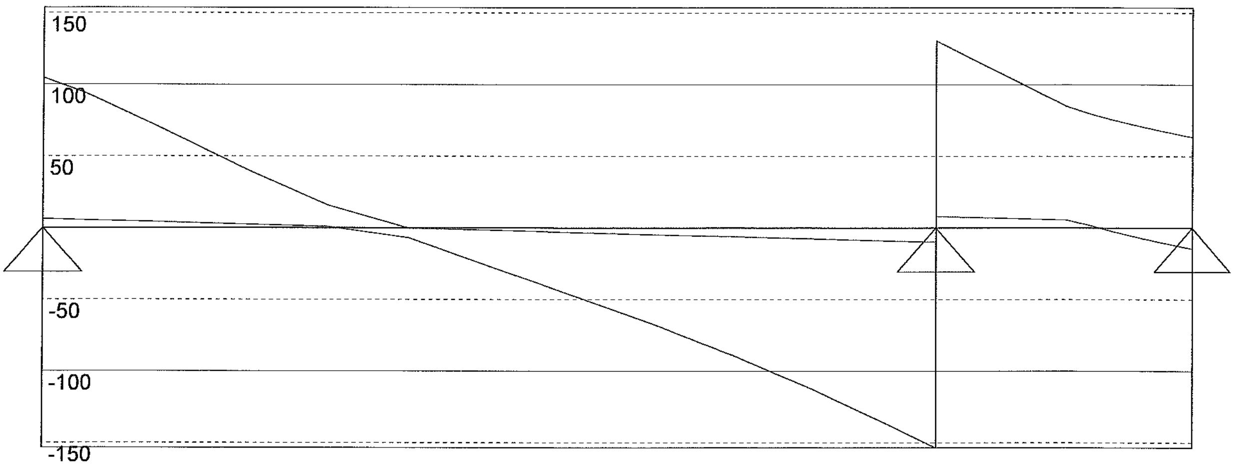

Load factor of dead load= 1.2 Load factor of imposed load= 1.6
Load width 2.45 (m) (by which the loads
has been multiplied during calculation)
Max/Min reactions of beam [kN]
104,678 284,183 14,466
6,193 17,781 -63,424

HEA 240 (Class of section=1/1) G= 60,3 I(cm4)=7763 W(cm3)=675 fy=235
Factored Moment/Moment capacity [kNm] 137,367 174,840 79 %
Factored shear force/shear capacity [kN] 153,622 230,535 67 %

W 8 x 58 ✓

Sum infl M+S 0,80 (must be<=1) x= 5549 M=137,3 S=153,62
Deflection due to unfactored load (Deflection limit L/360)
11,3 mm (74 %) 0,0 mm (0 %)
Attention! Ultimate limit design! Remember the load factors!!

Beam Id: Lot # 70 - MB 6

Date 22-05-2018

Structural Engineer:

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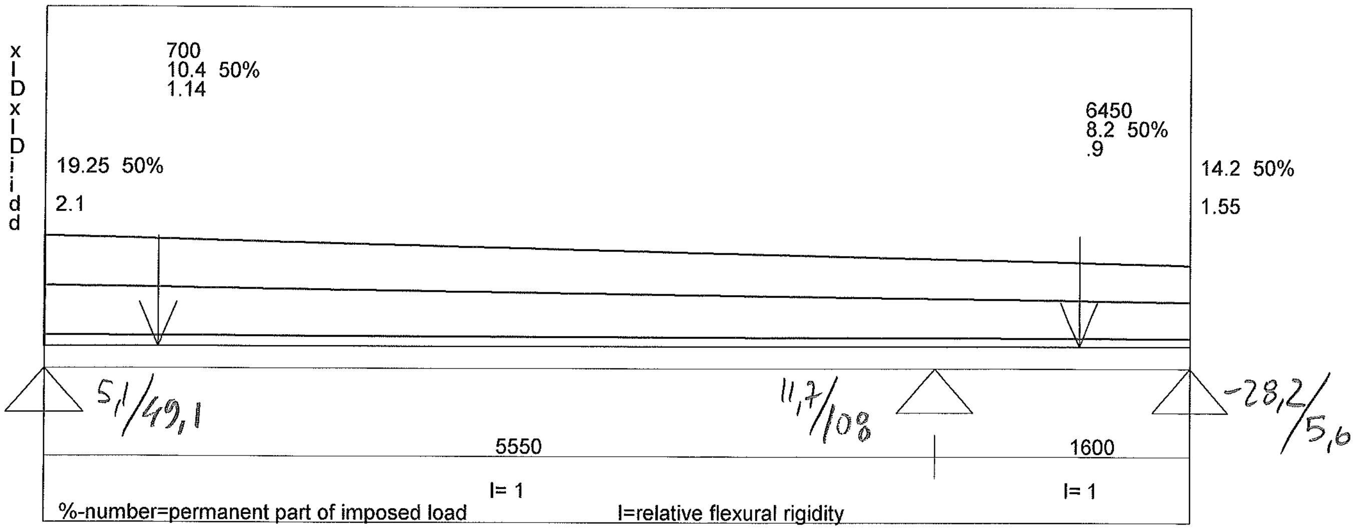

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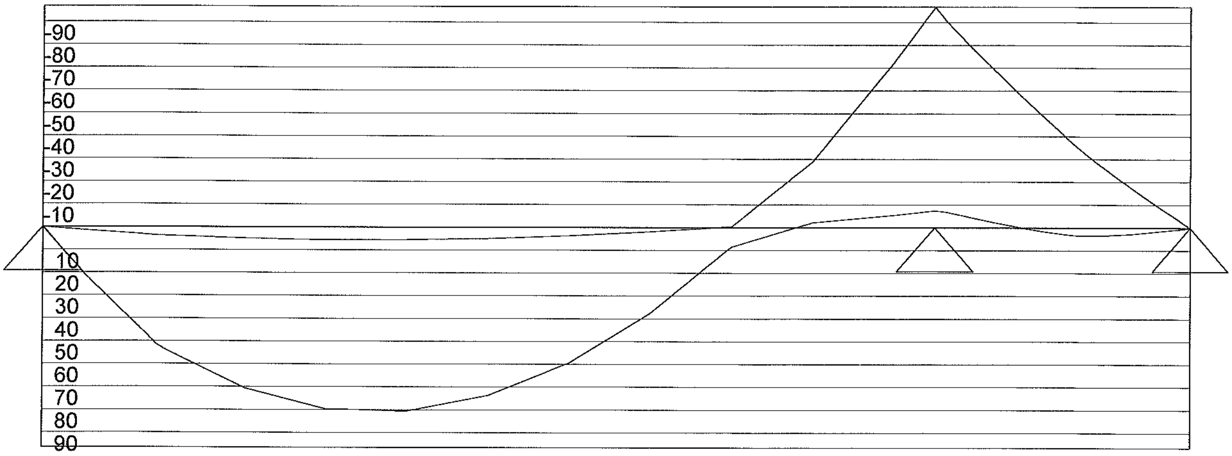

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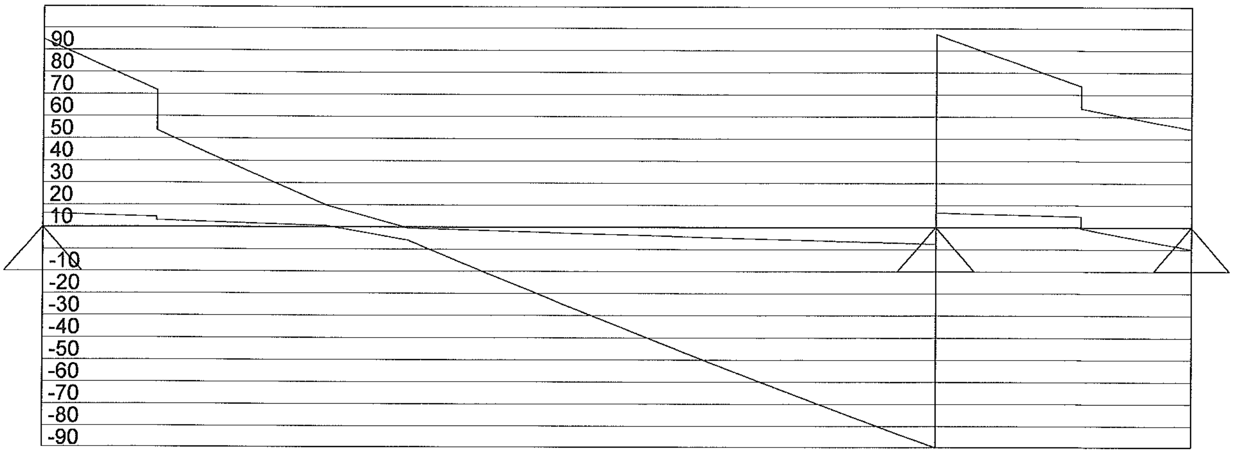


Load factor of dead load= 1.2 Load factor of imposed load= 1.6
Load width 1 (m) (by which the loads
has been multiplied during calculation)
Max/Min reactions of beam [kN]
84,707 186,903 9,791
6,166 14,144 -44,463

HEA 220 (Class of section=1/1) G= 50,5 I(cm4)=5410 W(cm3)=515 fy=235
Factored Moment/Moment capacity [kNm] 96,638 133,480 72 %
Factored shear force/shear capacity [kN] 99,582 196,413 51 %

W 8 x 35

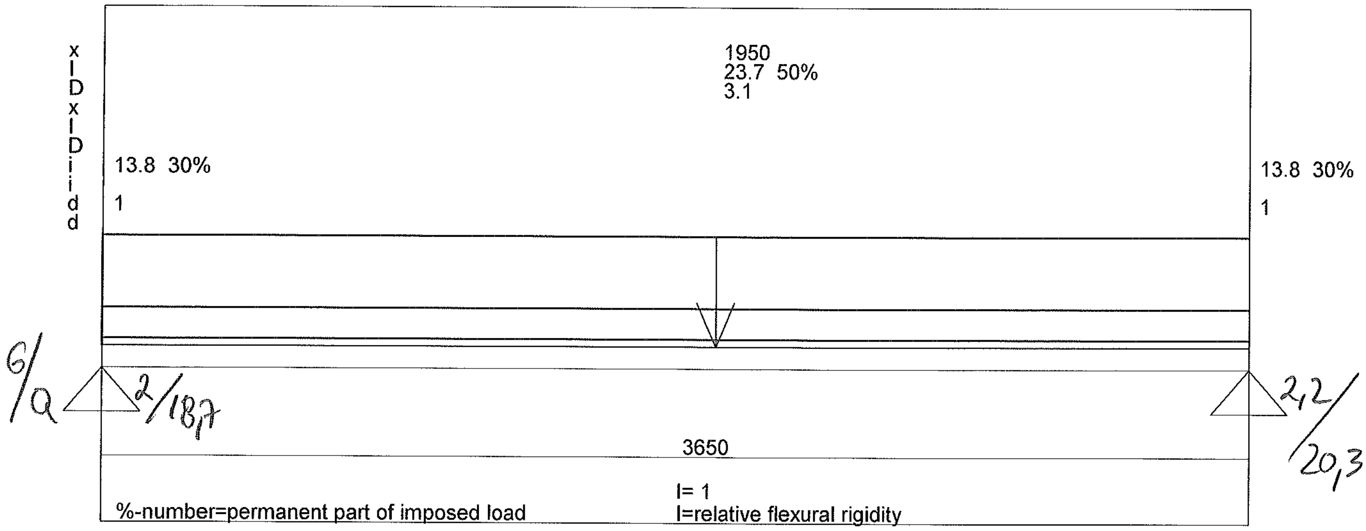
Deflection due to unfactored load (Deflection limit L/360)
11,9 mm (77 %) 0,0 mm (0 %)
Attention! Ultimate limit design! Remember the load factors!!

Beam Id: Lot # 70 - MB 7

Date 22-05-2018

Structural Engineer:

Licensed to: FINNLAMELLI OY



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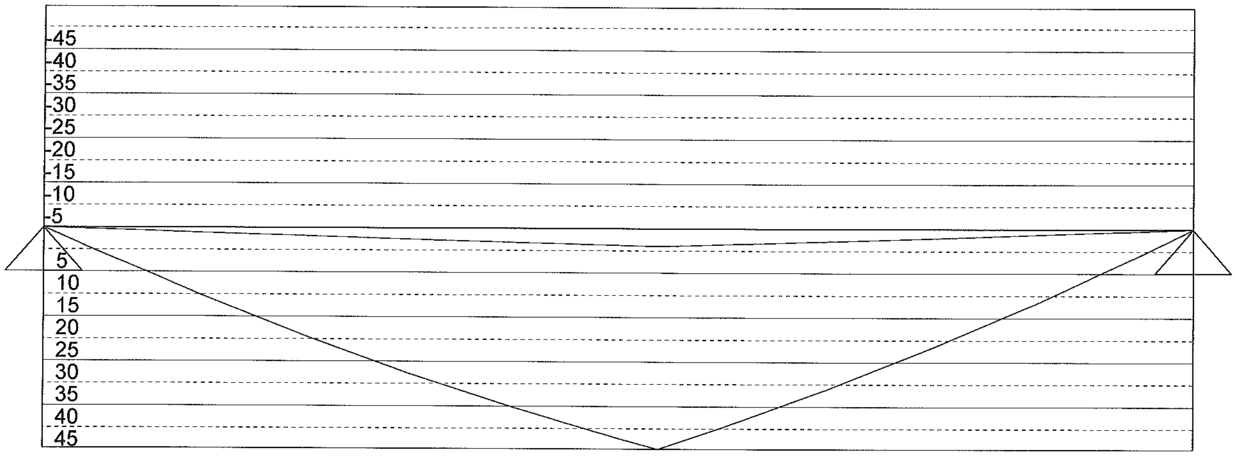

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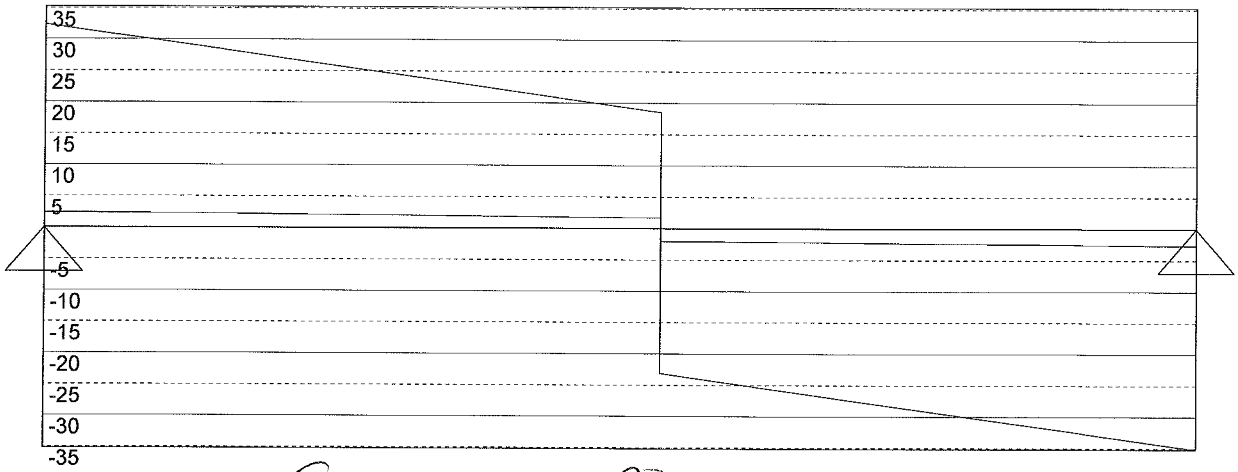

Load factor of dead load= 1.2 Load factor of imposed load= 1.6
Load width .305 (m) (by which the loads
has been multiplied during calculation)
Max/Min reactions of beam [kN]
32,349 35,204
2,400 2,655

HEA 180 (Class of section=1/1) G= 35,5 I(cm4)=2510 W(cm3)=294 fy=235
Factored Moment/Moment capacity [kNm] 49,586 76,140 65 %
Factored shear force/shear capacity [kN] 35,201 136,629 26 %

W8x21 ✓

Deflection due to unfactored load (Deflection limit L/360)
7,1 mm (70 %)
Attention! Ultimate limit design! Remember the load factors!!

Beam Id: Lot # 70 - MB 8

Structural Engineer:

Date 22-05-2018

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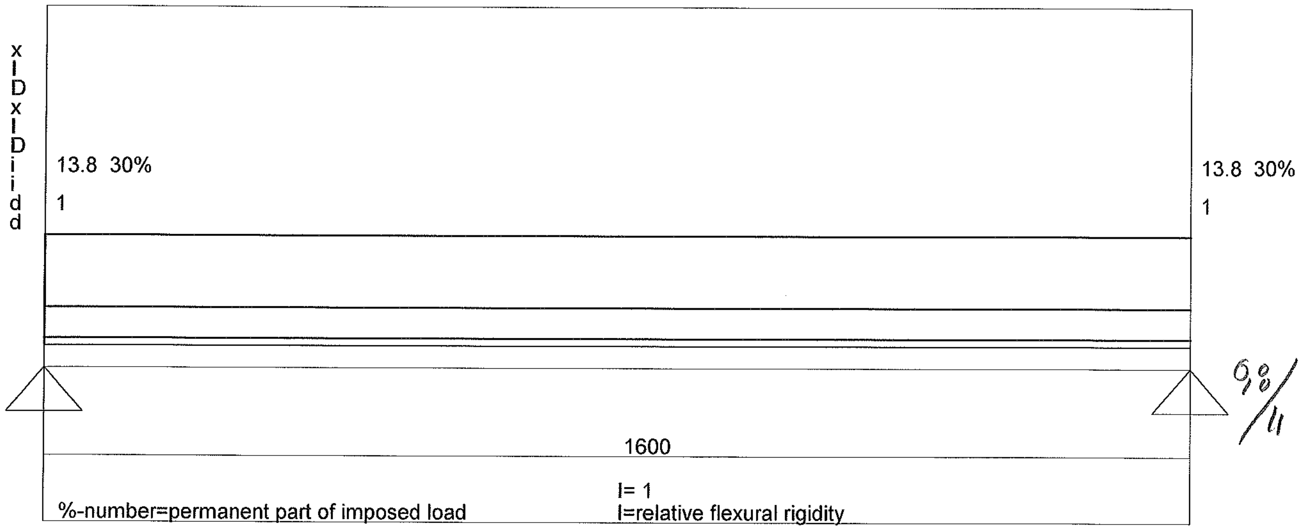

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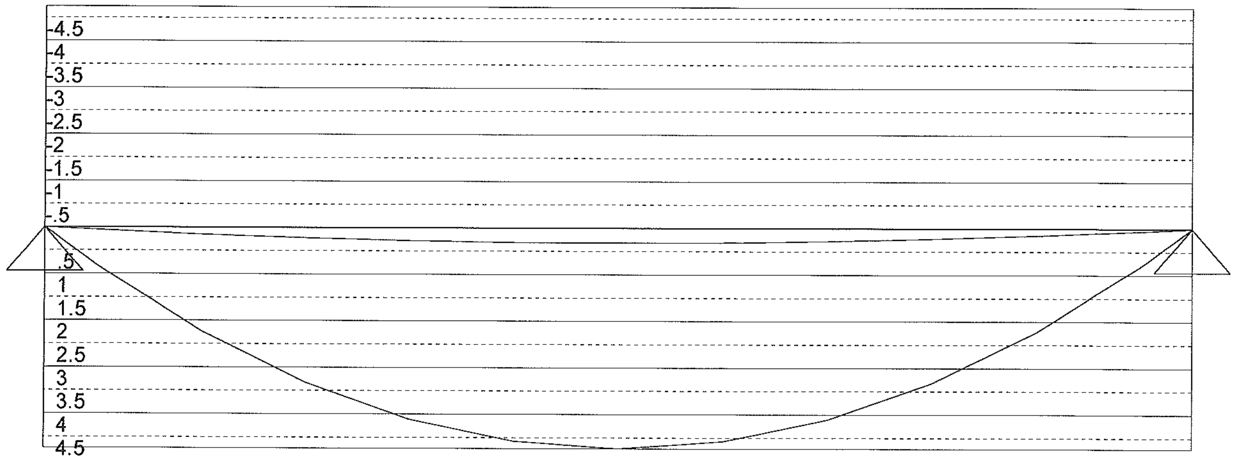

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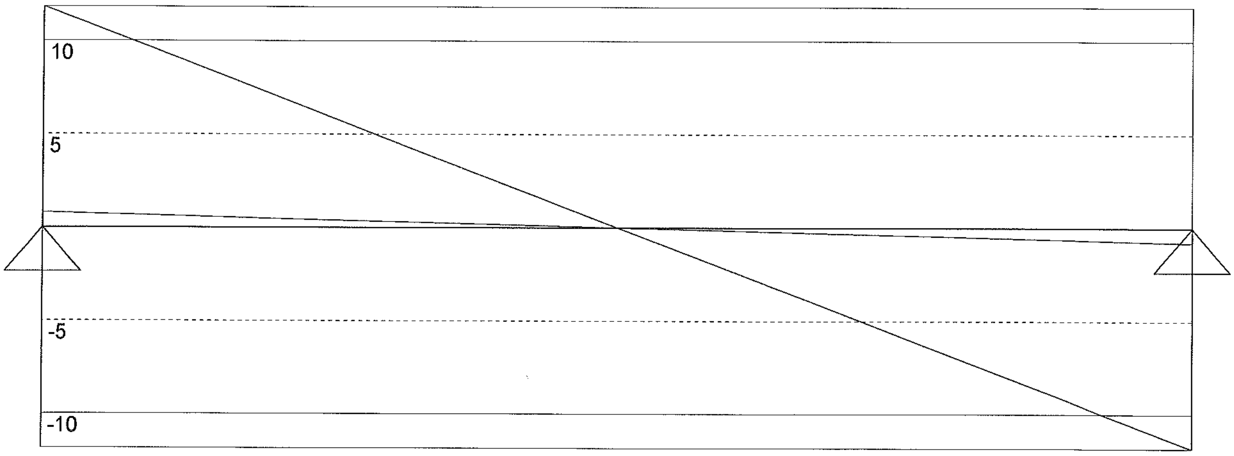

Load factor of dead load= 1 Load factor of imposed load= 1
Load width 1 (m) (by which the loads
has been multiplied during calculation)
Max/Min reactions of beam [kN]
11,838 11,840
0,800 0,800

T24 76 x 300 B 2 Cf=1,00 Design method: Allowable stress design
Increasing factor of the allowable stress 1,02
Factored Moment/Moment capacity [kNm] 4,736 11,132 43 %
Factored shear force/shear capacity [kN] 11,838 14,843 80 %

(2) LVL 1 1/2 x 11 7/8

Deflection due to unfactored load (Deflection limit L/360)
1,1 mm (26 %)

Beam Id: Lot # 70 - MB 9 (LVL)
Structural Engineer:
Date 22-05-2018

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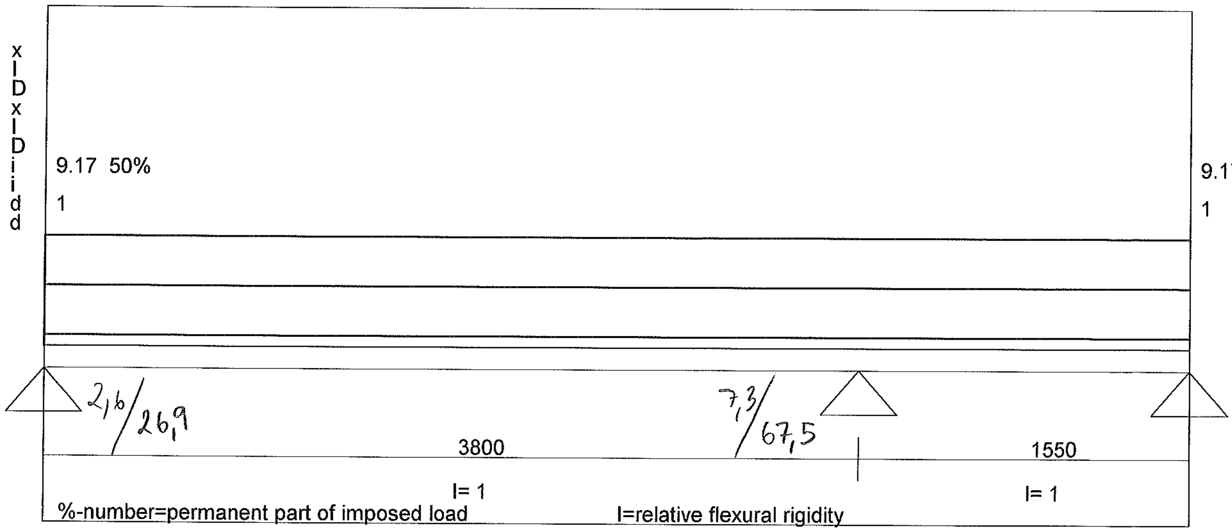

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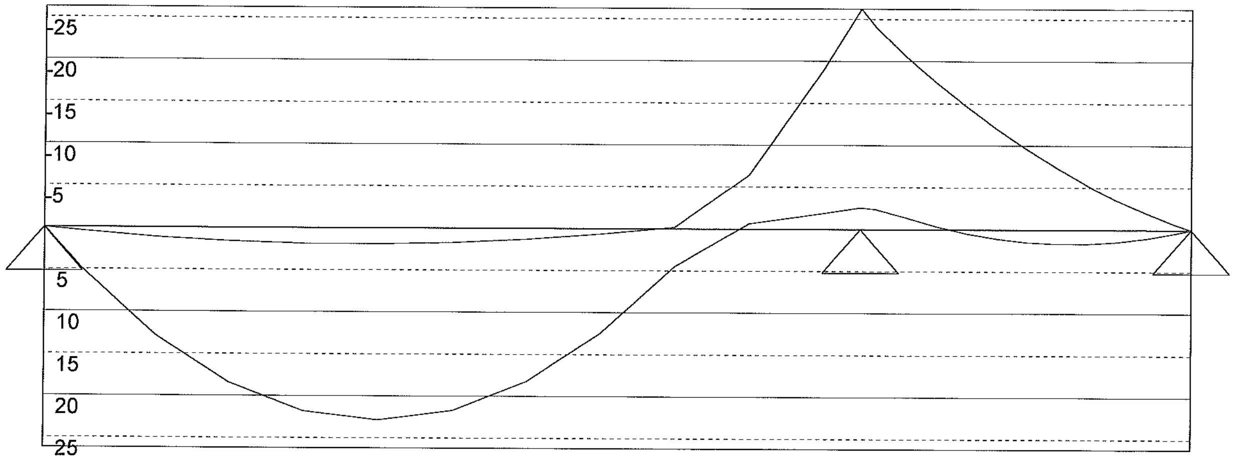

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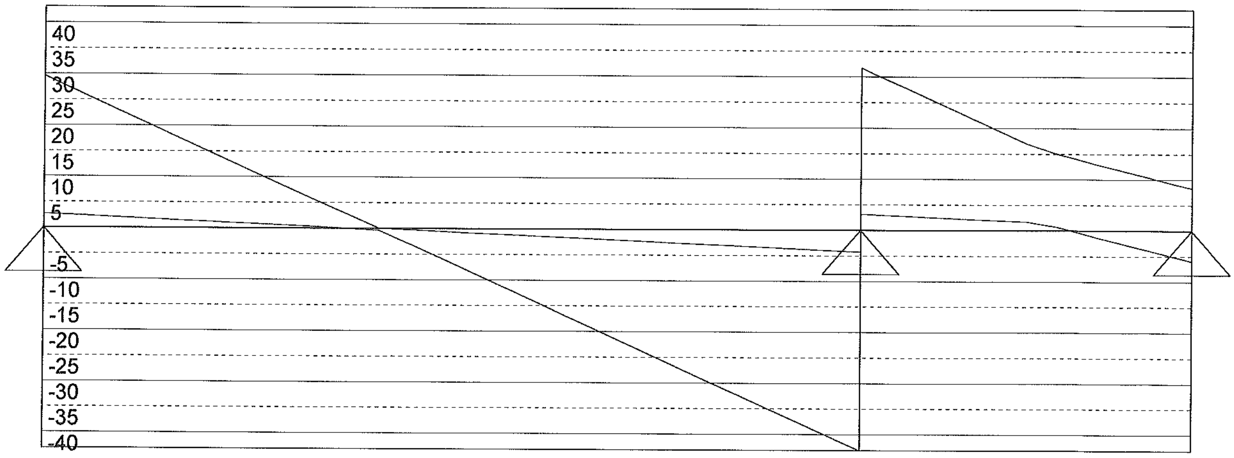


Load factor of dead load= 1 Load factor of imposed load= 1
Load width 1.875 (m) (by which the loads
has been multiplied during calculation)
Max/Min reactions of beam [kN]
29,524 74,803 5,969
2,686 7,355 -8,307

KER 44 x 300 B 2 Cf=1,00 Design method: Allowable stress design
Increasing factor of the allowable stress 1,03
Factored Moment/Moment capacity [kNm] 26,202 12,179 215 %
Factored shear force/shear capacity [kN] 43,120 15,337 281 %

LVL Log
=> 12,1 + 18,1 = 30,2 ok
=> 15,3 + 27,9 = 43,2 ok

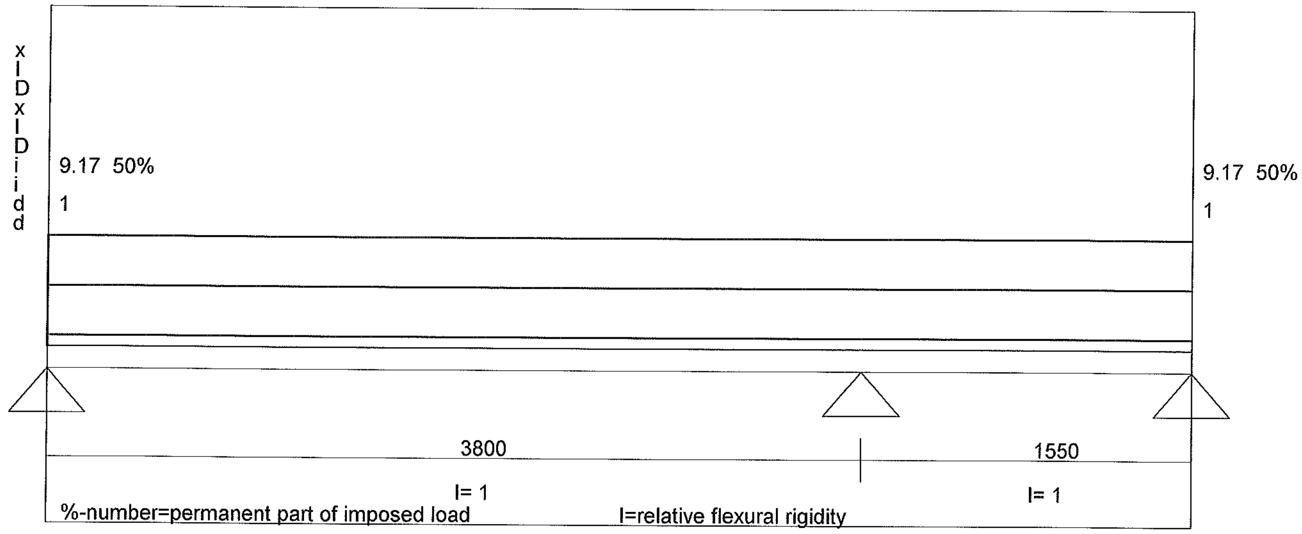
Deflection due to unfactored load (Deflection limit L/240)
28,1 mm (178 %) 0,3 mm (4 %)

Beam Id: Lot # 70 - MB 9 (L06)

Date 22-05-2018

Structural Engineer:

Licensed to: FINNLAMELLI OY



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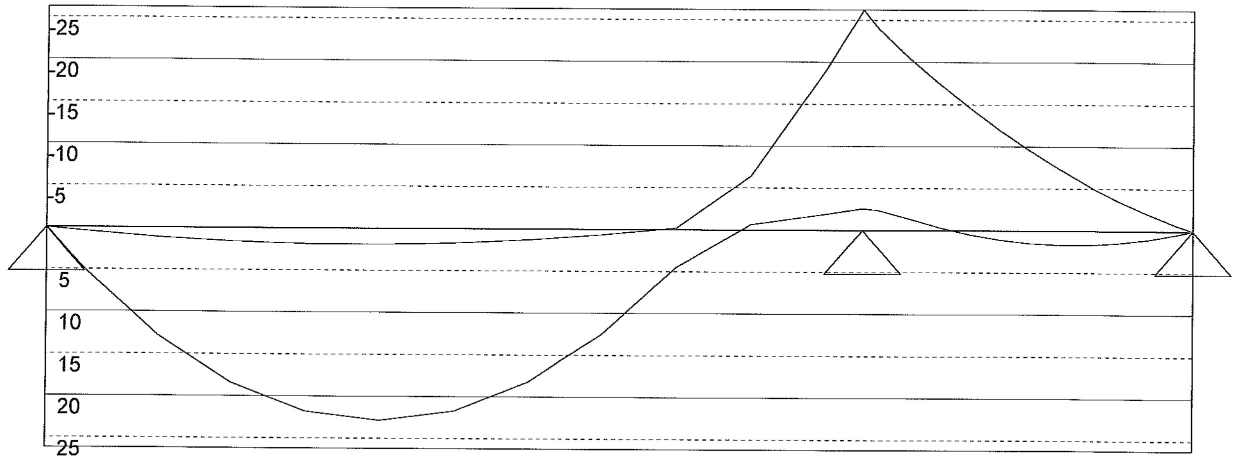
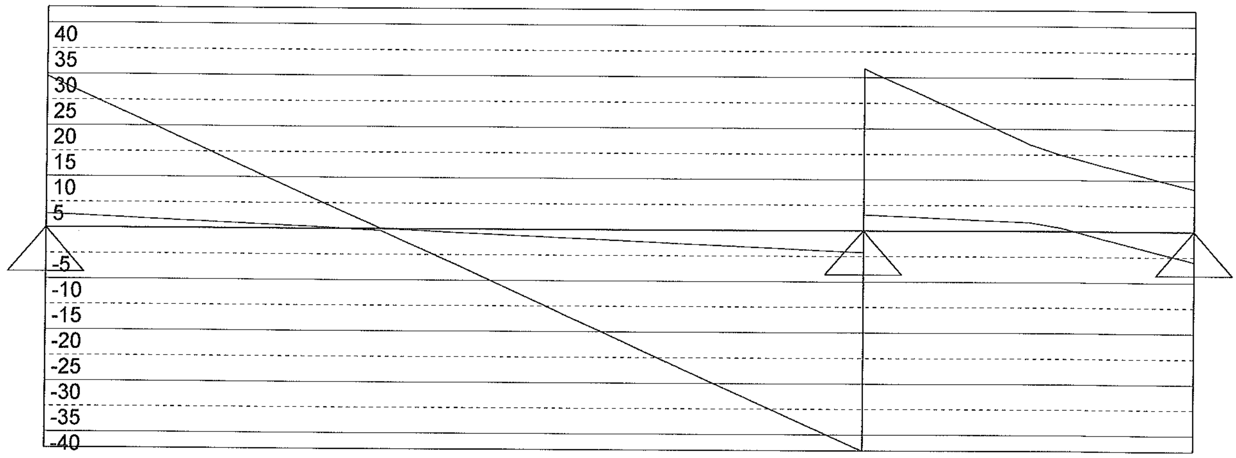


Licensed to: FINNLAMELLI OY



Load factor of dead load= 1 Load factor of imposed load= 1
Load width 1.875 (m) (by which the loads
has been multiplied during calculation)
Max/Min reactions of beam [kN]
29,524 74,803 5,969
2,686 7,355 -8,307

T24 164 x 260 B 2 Cf=1,00 Design method: Allowable stress design
Increasing factor of the allowable stress 1,03
Factored Moment/Moment capacity [kNm] 26,202 18,185 144 %
Factored shear force/shear capacity [kN] 43,120 27,977 154 %

(1) L06 +
(1) LVL $1\frac{3}{4} \times 11\frac{7}{8}$ ✓

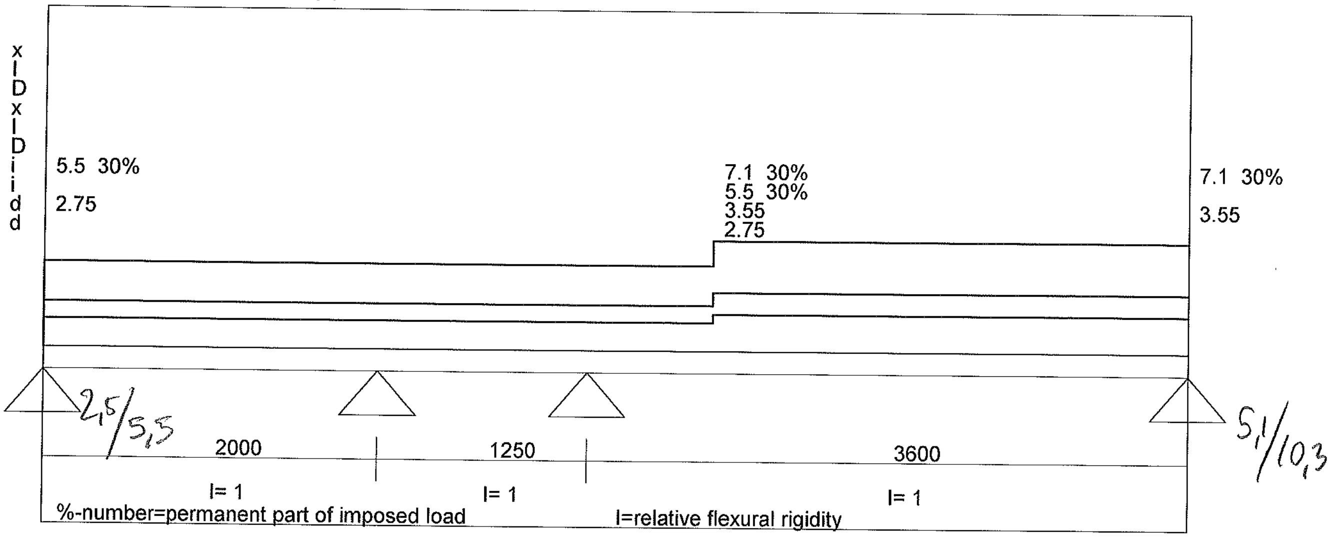
Deflection due to unfactored load (Deflection limit L/240)
18,6 mm (117 %) 0,2 mm (3 %)

Beam Id: Lot # 70 - MB 11

Date 23-05-2018

Structural Engineer:

Licensed to: FINNLAMELLI OY



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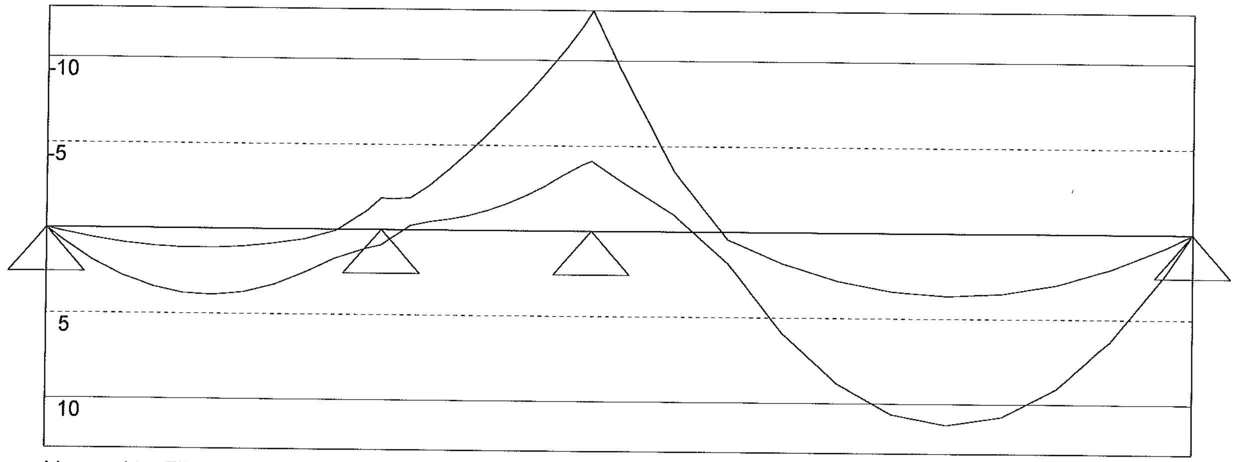

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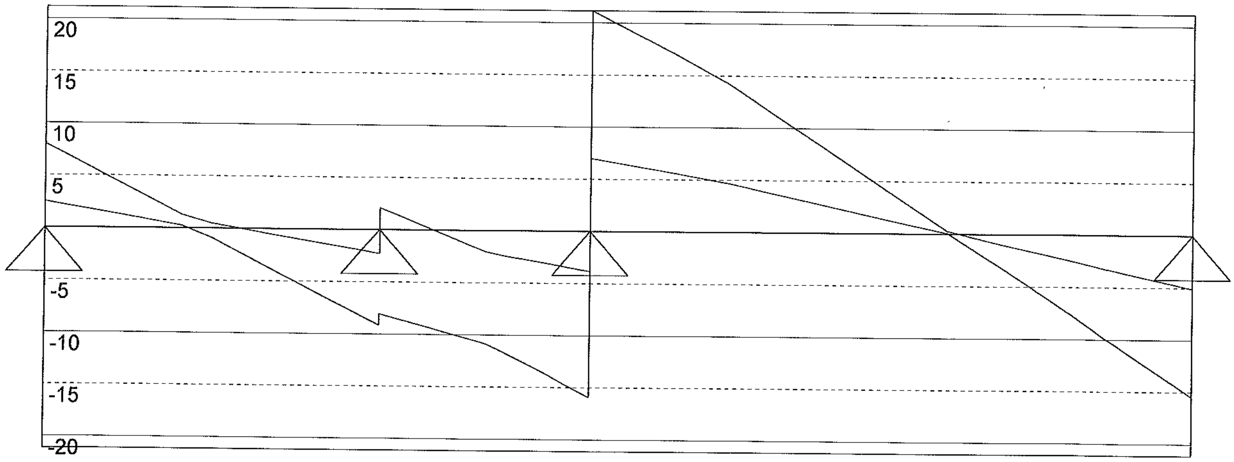

Load factor of dead load= 1 Load factor of imposed load= 1
Load width 1 (m) (by which the loads
has been multiplied during calculation)
Max/Min reactions of beam [kN]
8,022 10,366 37,053 15,479
2,501 -4,886 10,825 5,100

KER 76 x 300 B 2 Cf=1,00 Design method: Allowable stress design
Increasing factor of the allowable stress 1,09
Factored Moment/Moment capacity [kNm] 12,930 22,385 58 %
Factored shear force/shear capacity [kN] 21,147 28,189 75 %

(2) LVL $1\frac{1}{2} \times 11\frac{7}{8}$

SEE FORTE CALCULATION WITH
SEISMIC HOLD DOWN FORCE

Deflection due to unfactored load (Deflection limit L/360)
0,9 mm (16 %) -0,2 mm (4 %) 7,1 mm (71 %)

Beam Id: Lot # 70 - MB 12

Date 23-05-2018

Structural Engineer:

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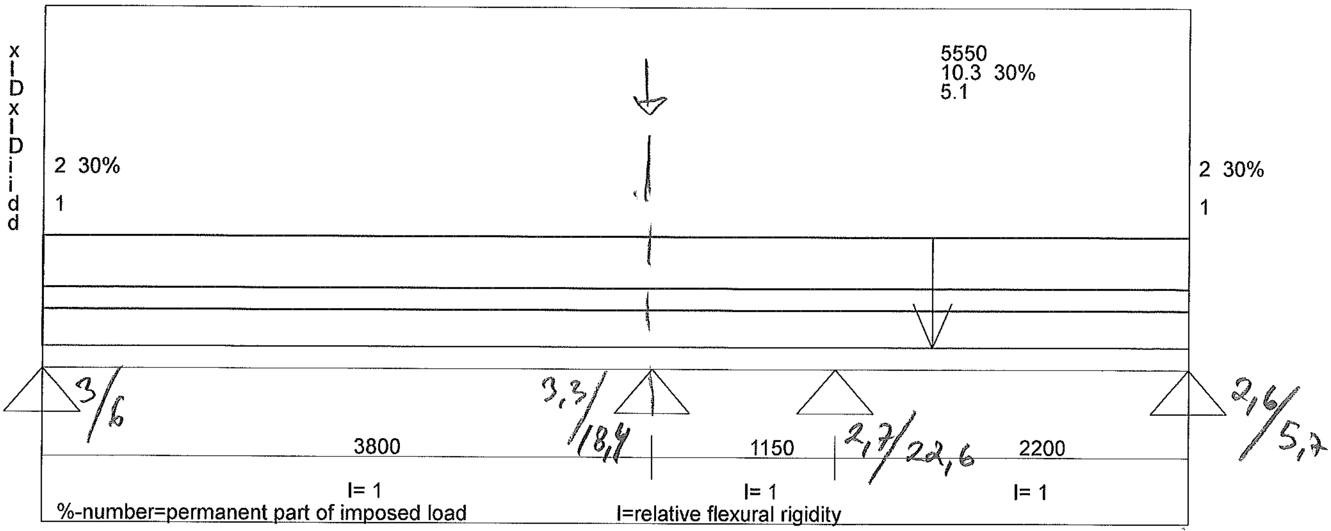

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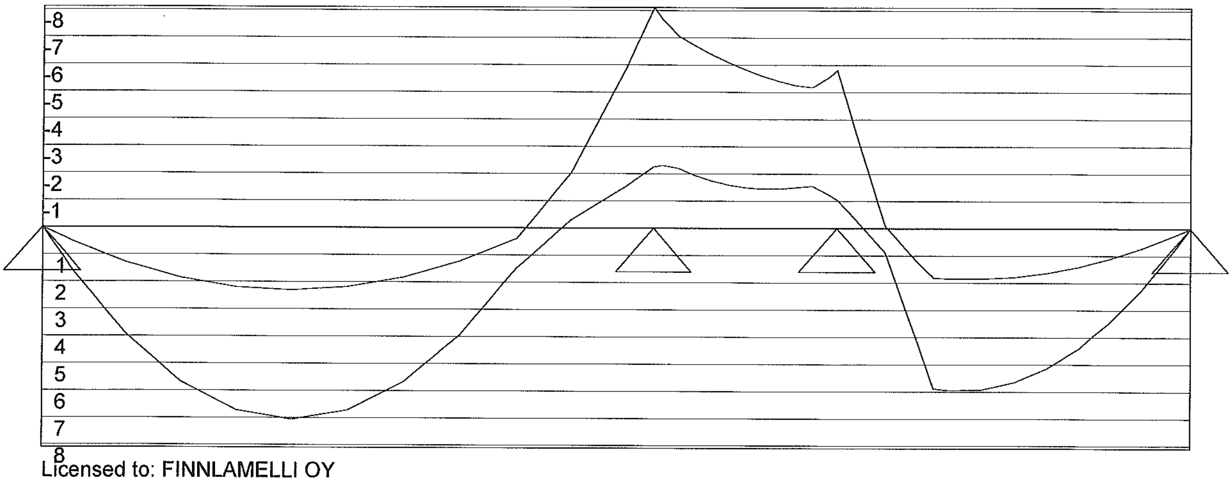

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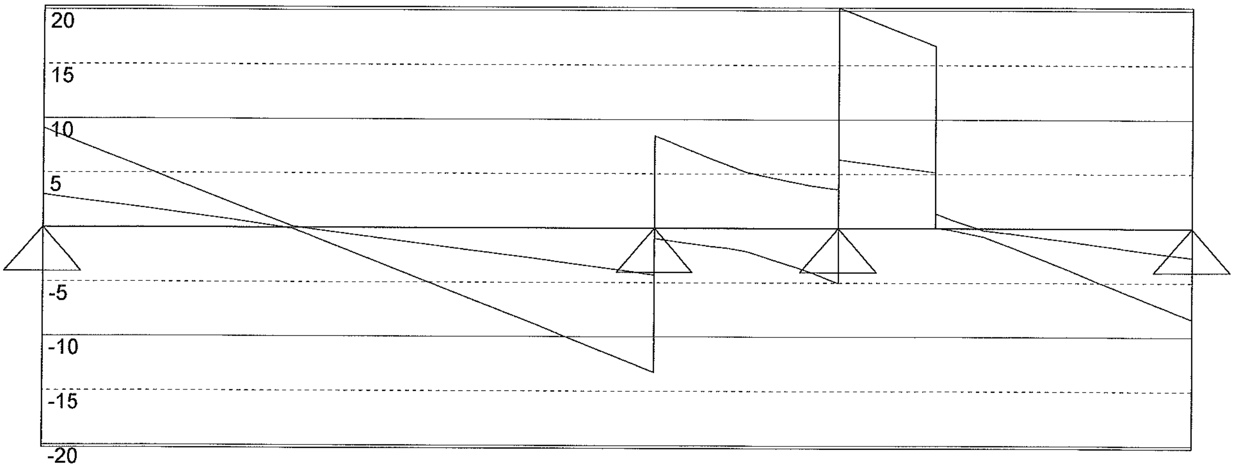

Load factor of dead load= 1 Load factor of imposed load= 1
Load width 1.95 (m) (by which the loads
has been multiplied during calculation)
Max/Min reactions of beam [kN]
9,093 21,730 25,306 8,360
3,000 3,339 2,735 2,697

KER 76 x 300 B 2 Cf=1,00 Design method: Allowable stress design
Increasing factor of the allowable stress 1,09
Factored Moment/Moment capacity [kNm] 8,102 22,377 36 %
Factored shear force/shear capacity [kN] 20,274 28,178 72 %

(2) LVL 1 1/2 x 11 7/8

Deflection due to unfactored load (Deflection limit L/360)
5,1 mm (48 %) -0,2 mm (5 %) 1,6 mm (26 %)

SEE FORTE CALCULATION WITH
SEISMIC HOLD DOWN FORCE

Beam Id: Lot # 70 - MB 13

Date 23-05-2018

Structural Engineer:

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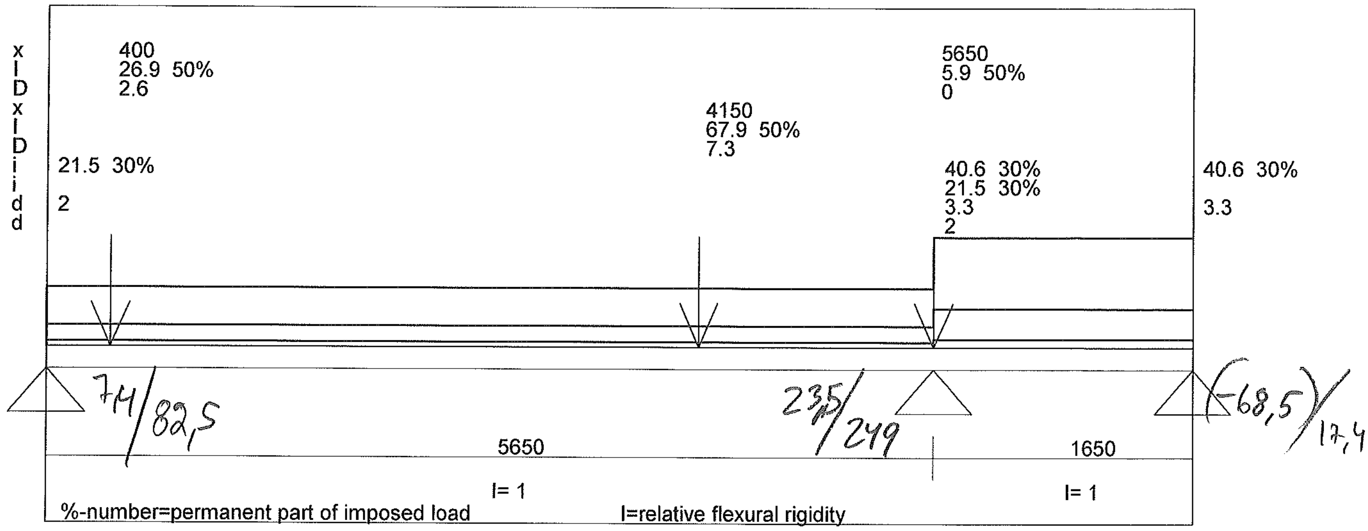

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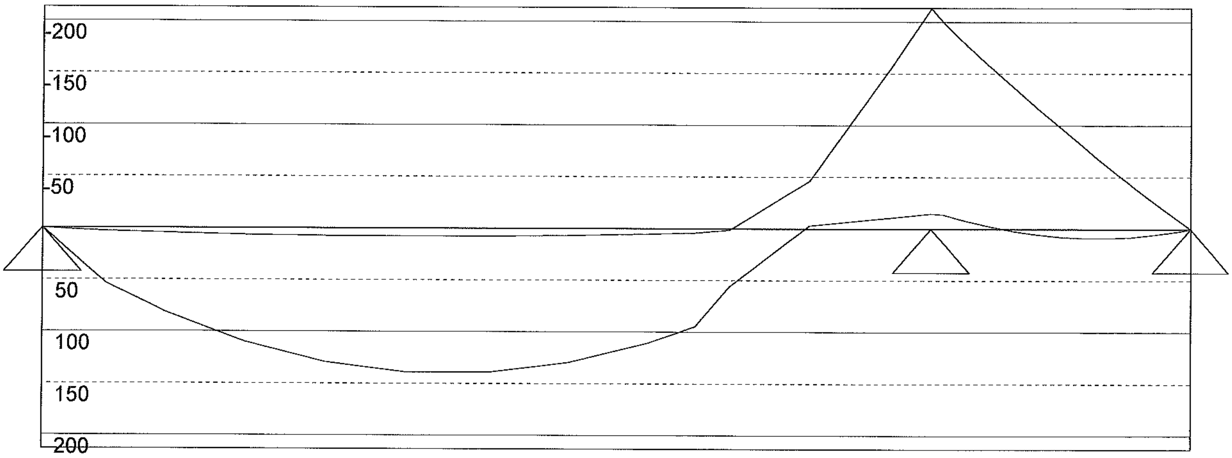

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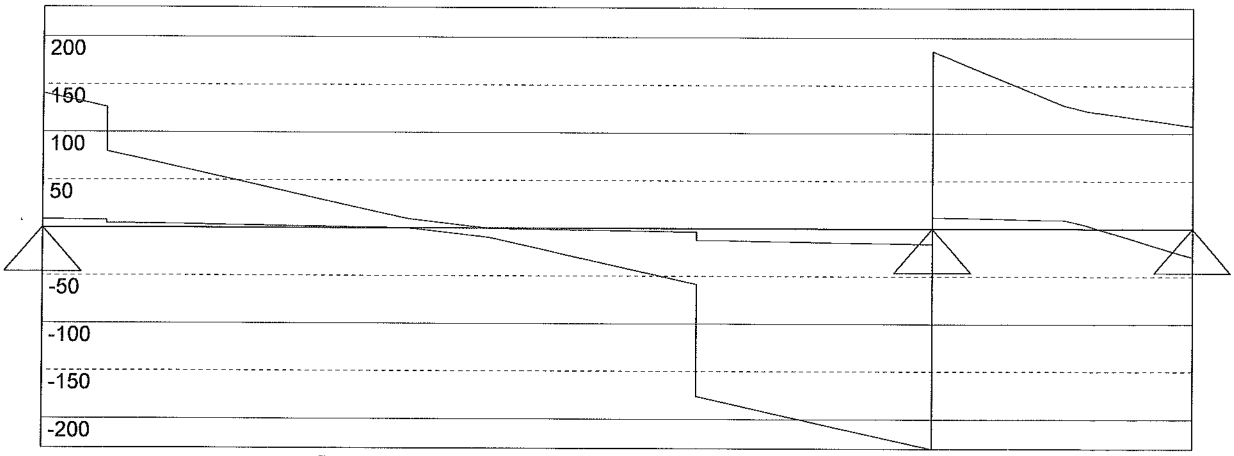

Load factor of dead load= 1.2 Load factor of imposed load= 1.6
Load width 1 (m) (by which the loads
has been multiplied during calculation)
Max/Min reactions of beam [kN]
140,863 426,732 29,743
8,779 28,212 -107,791

HEB 240 (Class of section=1/1) G= 83,2 I(cm4)=11259 W(cm3)=938 fy=235
Factored Moment/Moment capacity [kNm] 213,255 247,690 86 %
Factored shear force/shear capacity [kN] 231,180 314,430 74 %

W8x67 ✓

Sum infl M+S 0,88 (must be<=1) x= 5649 M=213,15 S=231,18
Deflection due to unfactored load (Deflection limit L/360)
11,5 mm (73 %) 0,0 mm (1 %)
Attention! Ultimate limit design! Remember the load factors!!

Beam Id: Lot # 70 - MB 14

Date 23-05-2018

Structural Engineer:

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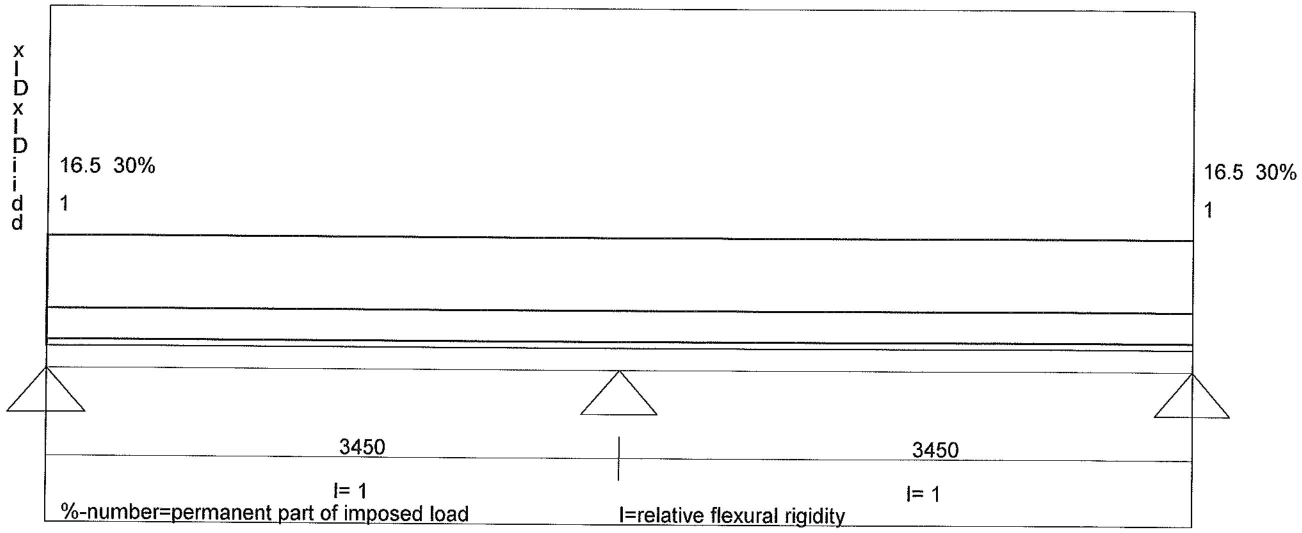

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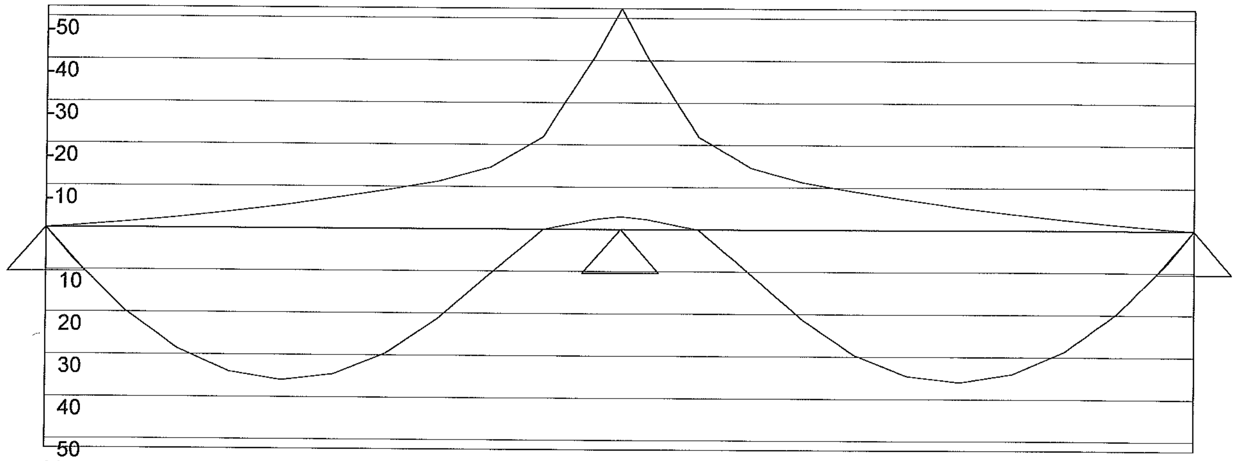

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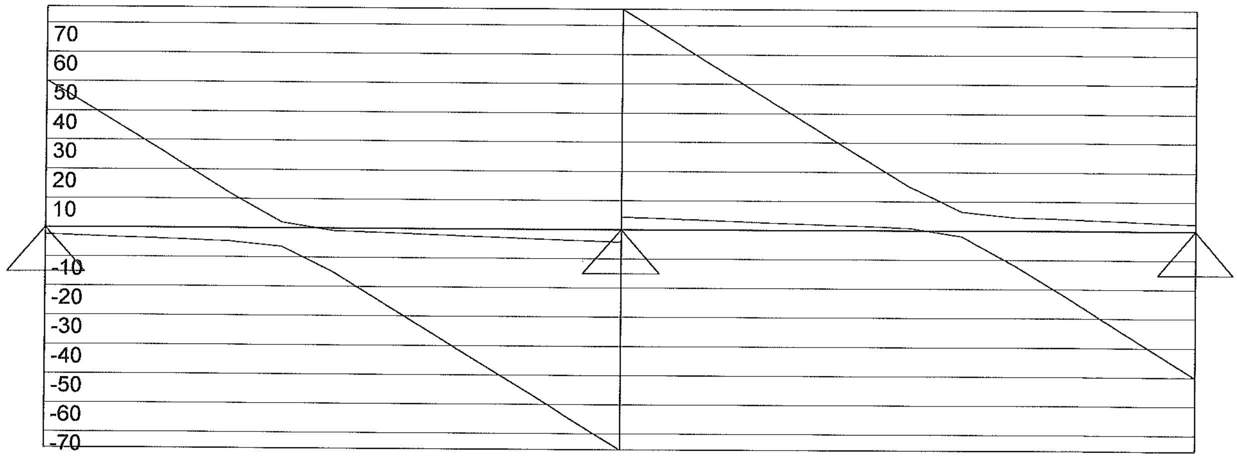

Load factor of dead load= 1 Load factor of imposed load= 1
Load width 2 (m) (by which the loads
has been multiplied during calculation)
Max/Min reactions of beam [kN]
50,211 151,030 50,223
-2,417 8,630 -2,416

L40 255 x 390 B 2 Cf=0,97 Design method: Allowable stress design
Increasing factor of the allowable stress 1,01
Factored Moment/Moment capacity [kNm] 52,248 94,906 55 %
Factored shear force/shear capacity [kN] 75,515 77,350 98 %

$GL\ (\overbrace{90 + 165}^{255}) \times 390$

$115 + 165 = 280$

$GL\ (4\tfrac{1}{2}'' + 6\tfrac{1}{2}'') \times 15\tfrac{3}{8}''$

Deflection due to unfactored load (Deflection limit L/360)
3,6 mm (38 %) 3,6 mm (38 %)

Beam Id: Lot # 70 - MB 15 Date 23-05-2018

Structural Engineer:

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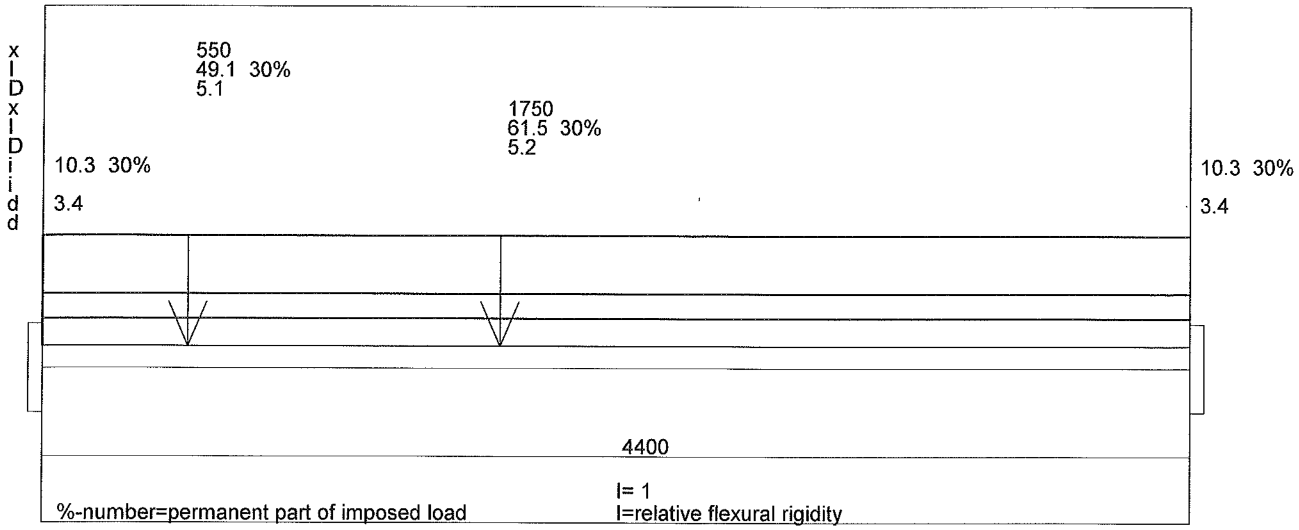

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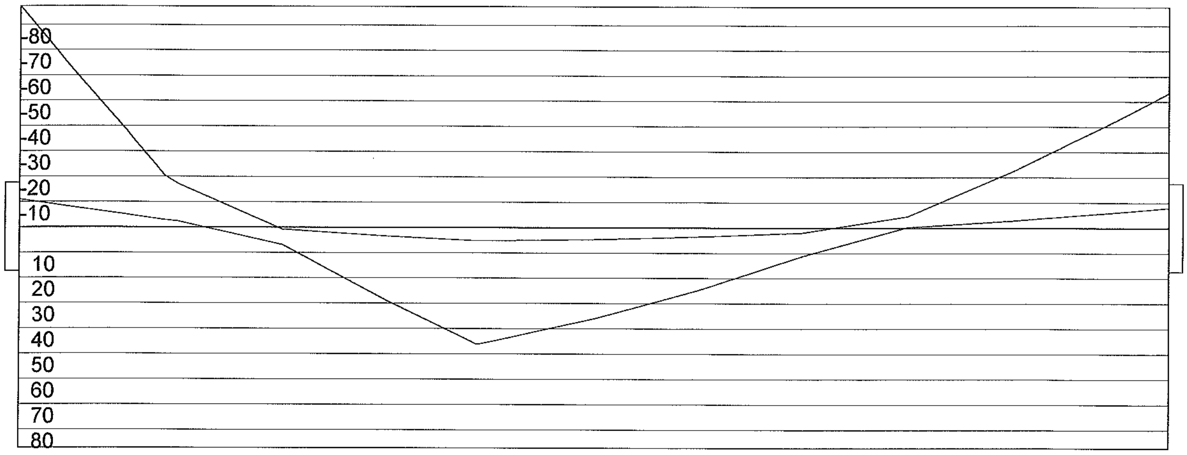

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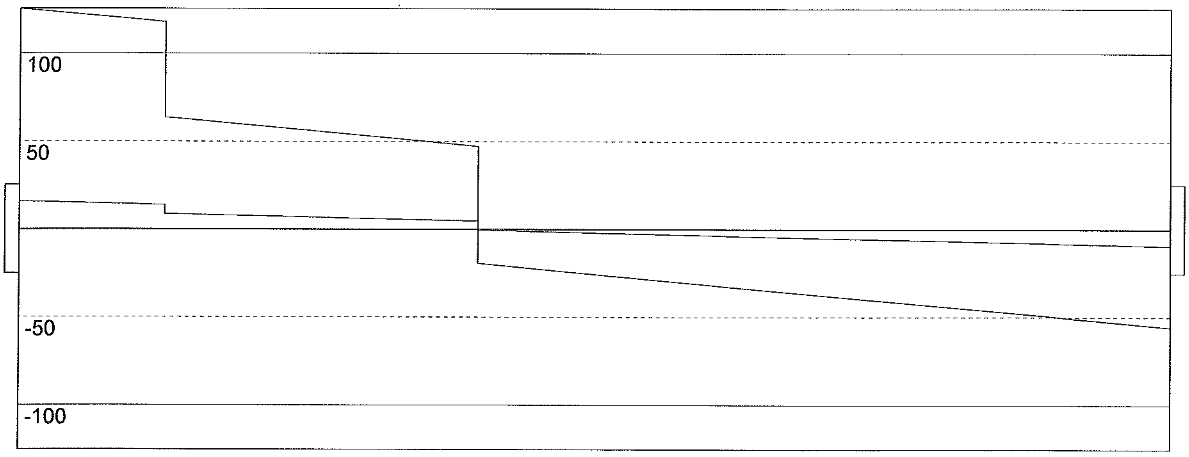

Load factor of dead load= 1 Load factor of imposed load= 1
Load width 1 (m) (by which the loads
has been multiplied during calculation)
Max/Min reactions of beam [kN]
125,433 55,731
15,743 9,513

T24 825 x 260 B 2 Cf=1,00 Design method: Allowable stress design
Increasing factor of the allowable stress 1,03
Factored Moment/Moment capacity [kNm] 87,277 92,120 95 %
Factored shear force/shear capacity [kN] 125,433 141,722 89 %

(5) LOGS ✓

Deflection due to unfactored load (Deflection limit L/360)
5,5 mm (45 %)

Beam Id: Lot # 70 - MB /6 Date 23-05-2018

Structural Engineer:

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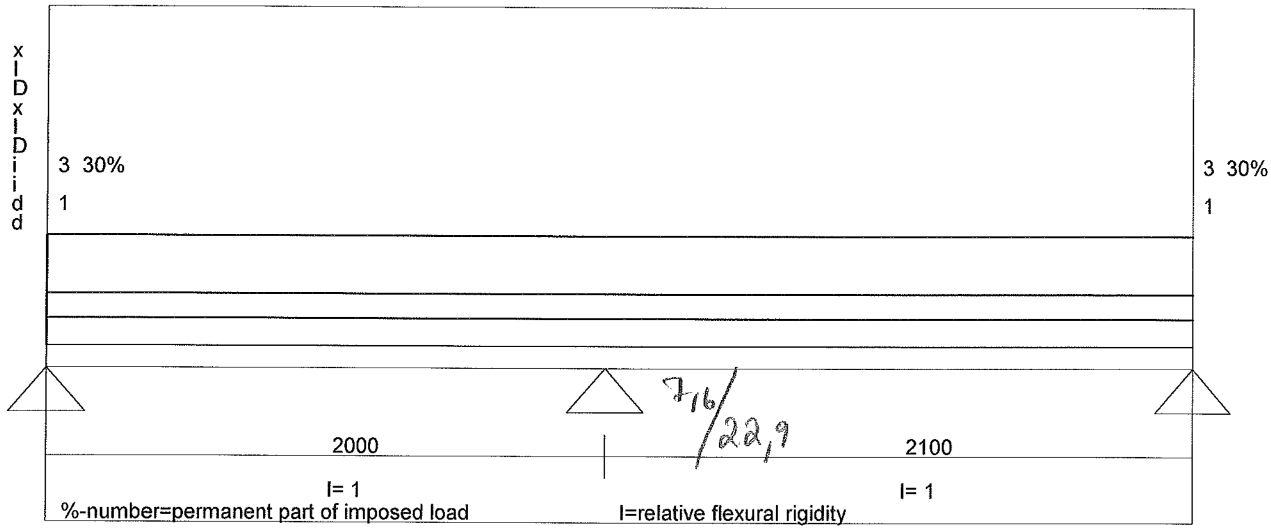

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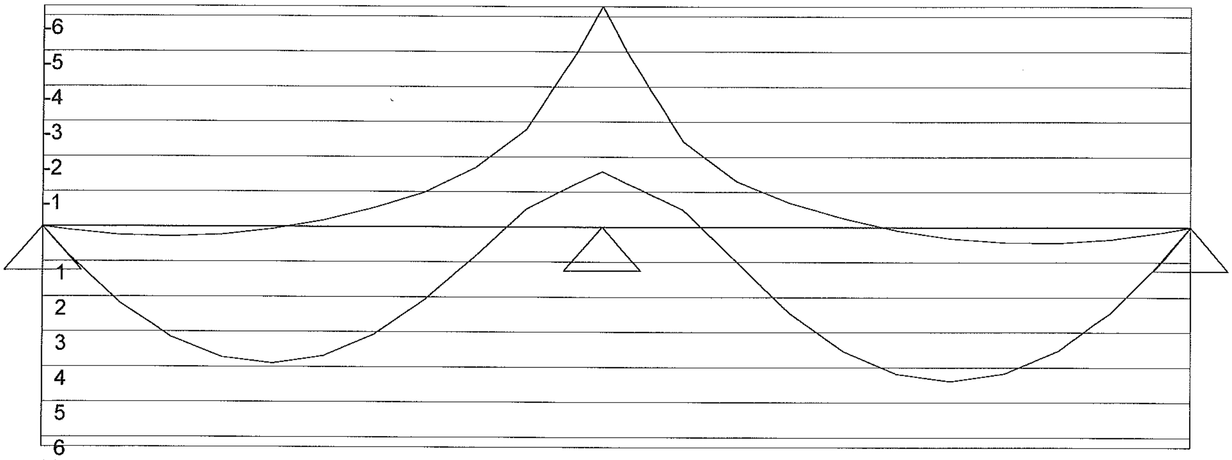

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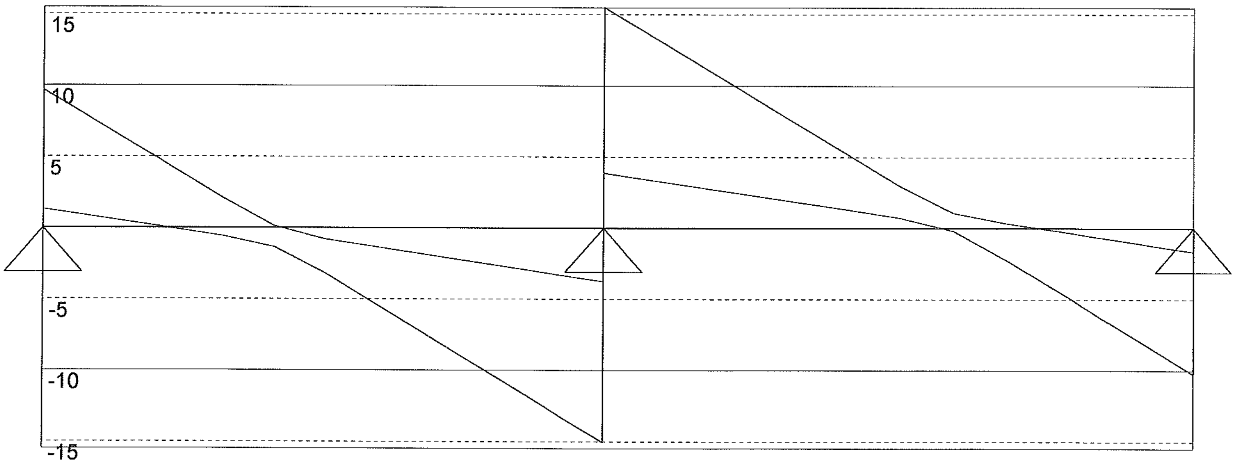

Load factor of dead load= 1 Load factor of imposed load= 1
Load width 2.975 (m) (by which the loads
has been multiplied during calculation)
Max/Min reactions of beam [kN]
9,640 30,527 10,230
1,303 7,632 1,647

T24 164 x 260 B 2 Cf=1,00 Design method: Allowable stress design 1 LOG
Increasing factor of the allowable stress 1,07
Factored Moment/Moment capacity [kNm] 6,283 18,921 33 %
Factored shear force/shear capacity [kN] 15,486 29,109 53 %

Deflection due to unfactored load (Deflection limit L/360)
0,9 mm (16 %) 1,1 mm (19 %)

Beam Id: Lot # 70 - MB 21

Date 24-05-2018

Structural Engineer:

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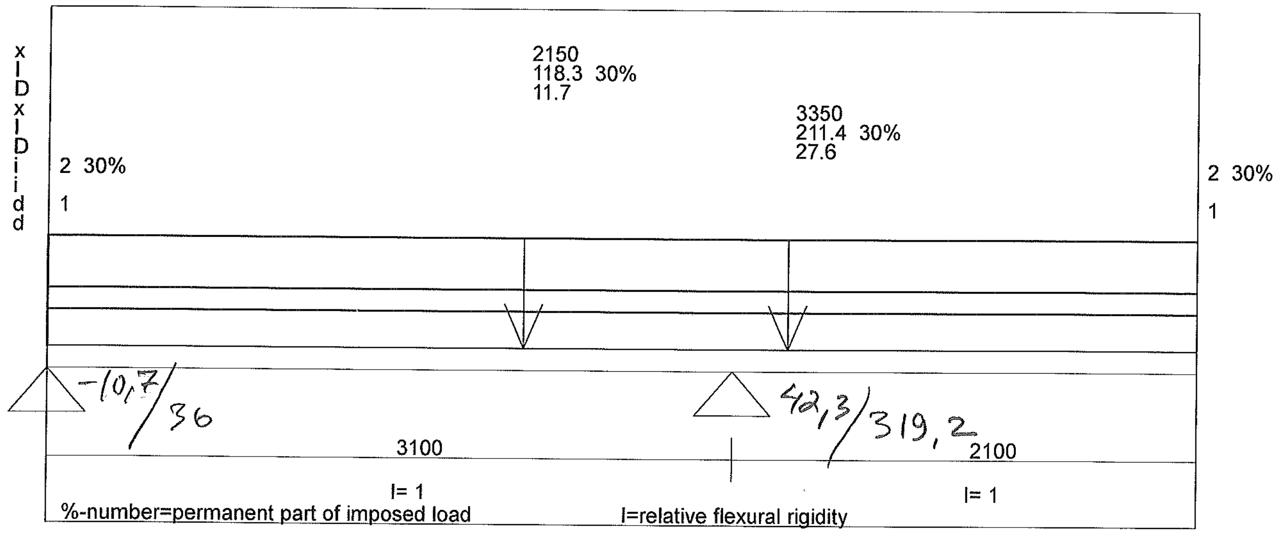

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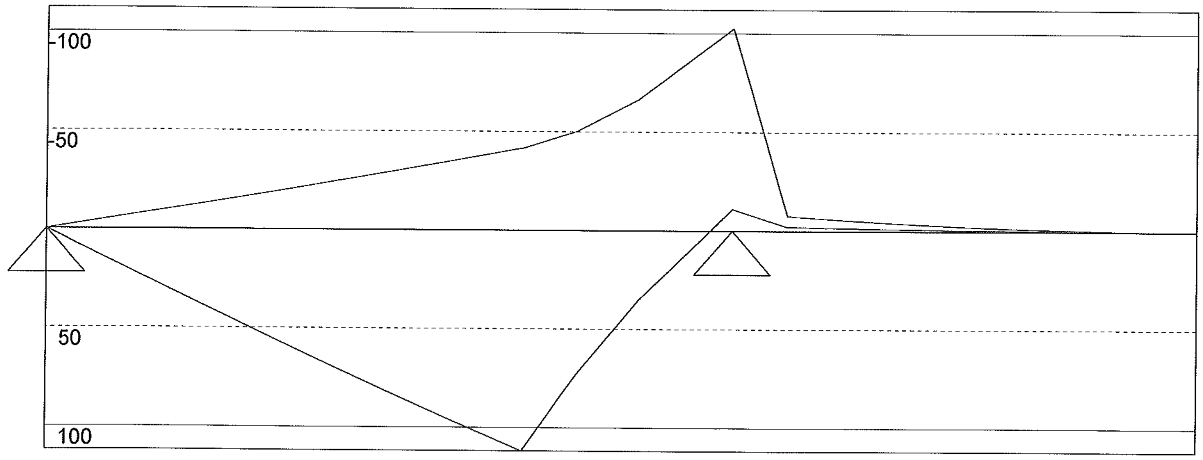

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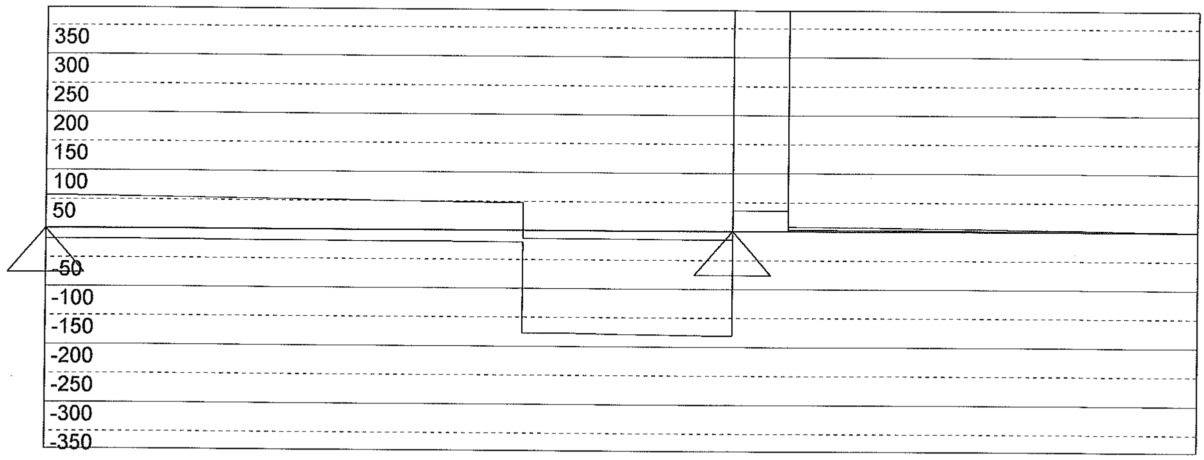

Load factor of dead load= 1.2 Load factor of imposed load= 1.6
Load width 1 (m) (by which the loads
has been multiplied during calculation)
Max/Min reactions of beam [kN]
56,736 561,507
-18,050 50,761

HEB 280 (Class of section=1/1) G= 103 I(cm4)=19270 W(cm3)=1380 fy=235
Factored Moment/Moment capacity [kNm] 111,815 360,490 31 %
Factored shear force/shear capacity [kN] 380,598 387,891 98 %

W10 x 88

Deflection due to unfactored load (Deflection limit L/360)/L/180)!
1,3 mm (15 %) 2,6 mm (23 %)
Attention! Ultimate limit design! Remember the load factors!!

Beam Id: Lot # 70 - MB 22

Structural Engineer:

Date 24-05-2018

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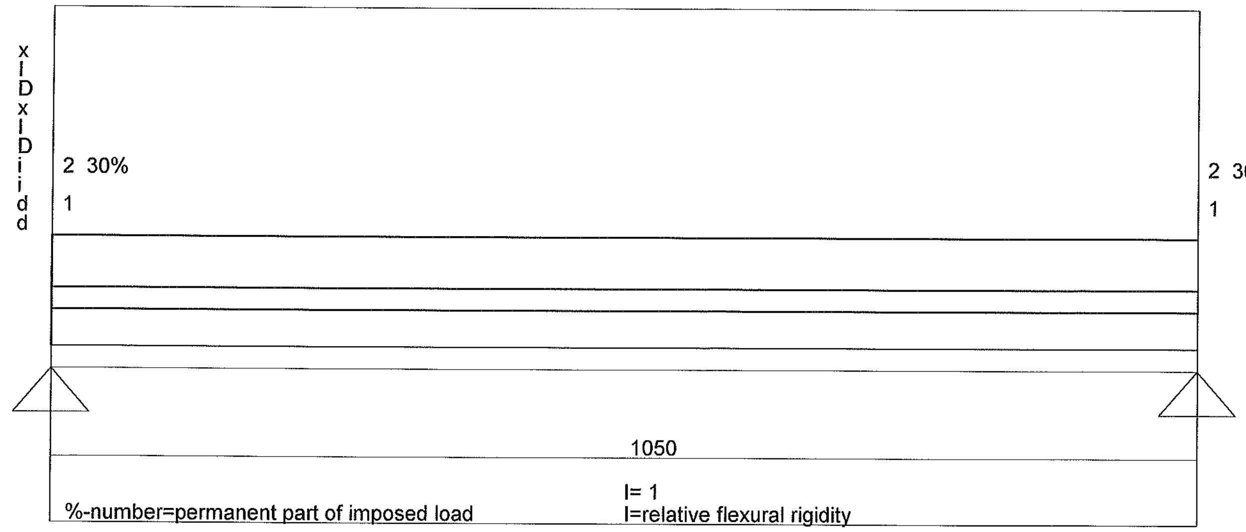

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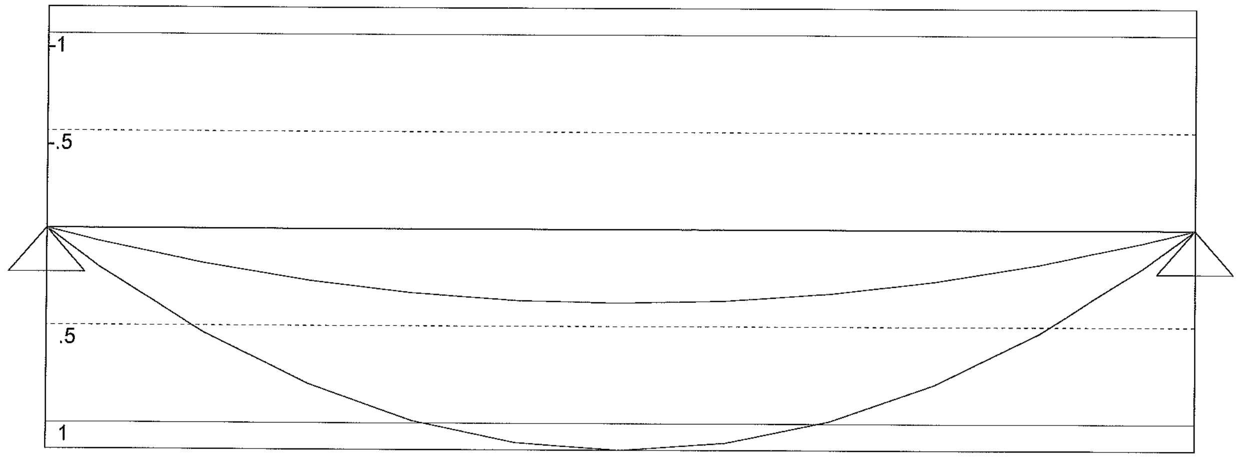

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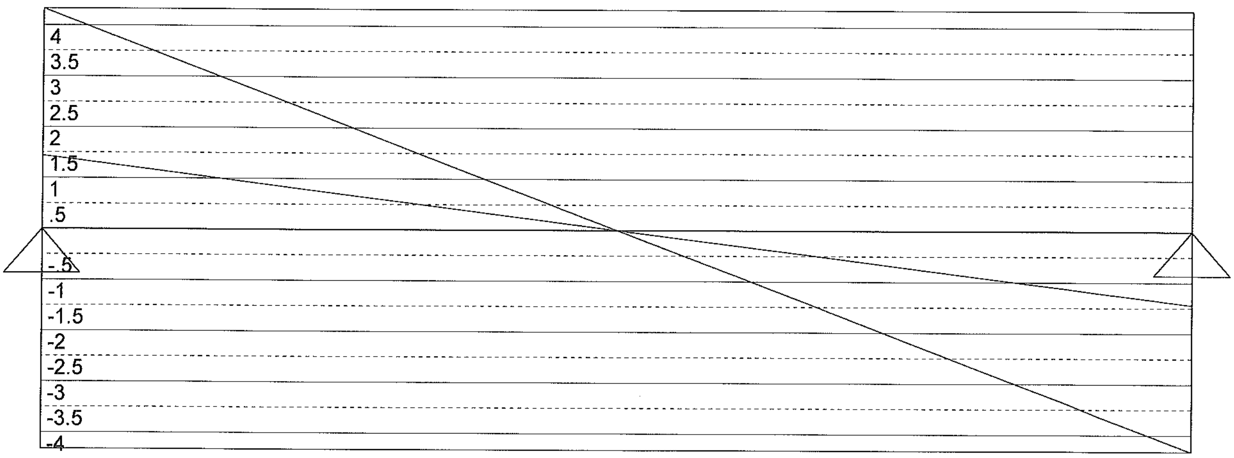

Load factor of dead load= 1 Load factor of imposed load= 1
Load width 2.75 (m) (by which the loads
has been multiplied during calculation)
Max/Min reactions of beam [kN]
4,330 4,331
1,443 1,444

T24 76 x 185 B 2 Cf=1,00 Design method: Allowable stress design
Increasing factor of the allowable stress 1,09
Factored Moment/Moment capacity [kNm] 1,137 4,540 25 %
Factored shear force/shear capacity [kN] 4,330 9,816 44 %

(2) 2×8

Deflection due to unfactored load (Deflection limit L/360)
0,5 mm (17 %)

Beam Id: Lot # 70 - MB 23

Date 24-05-2018

Structural Engineer:

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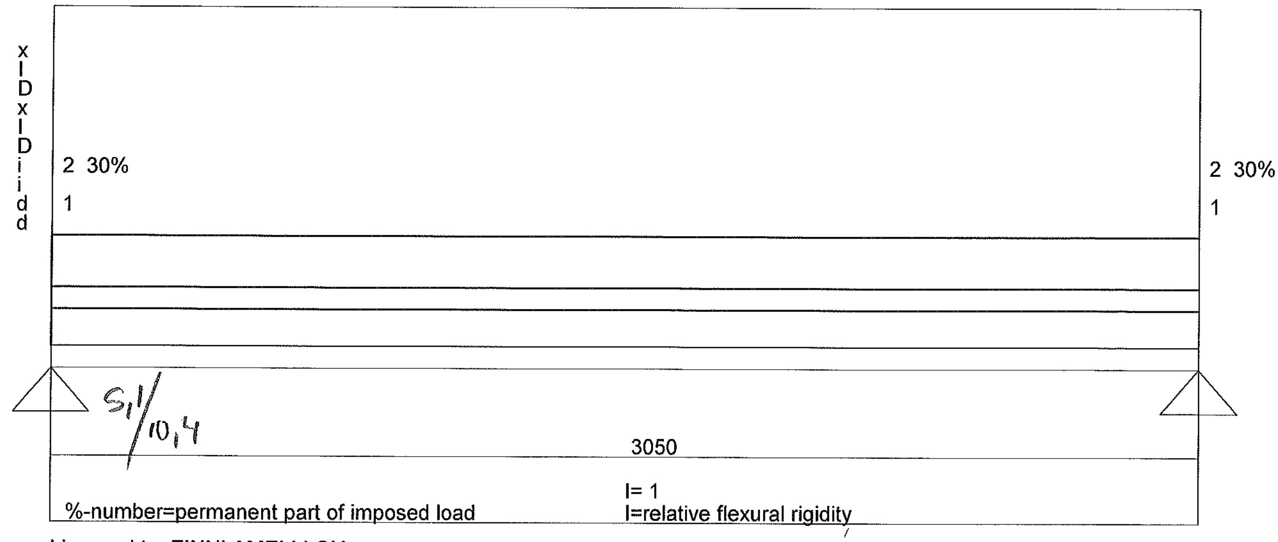

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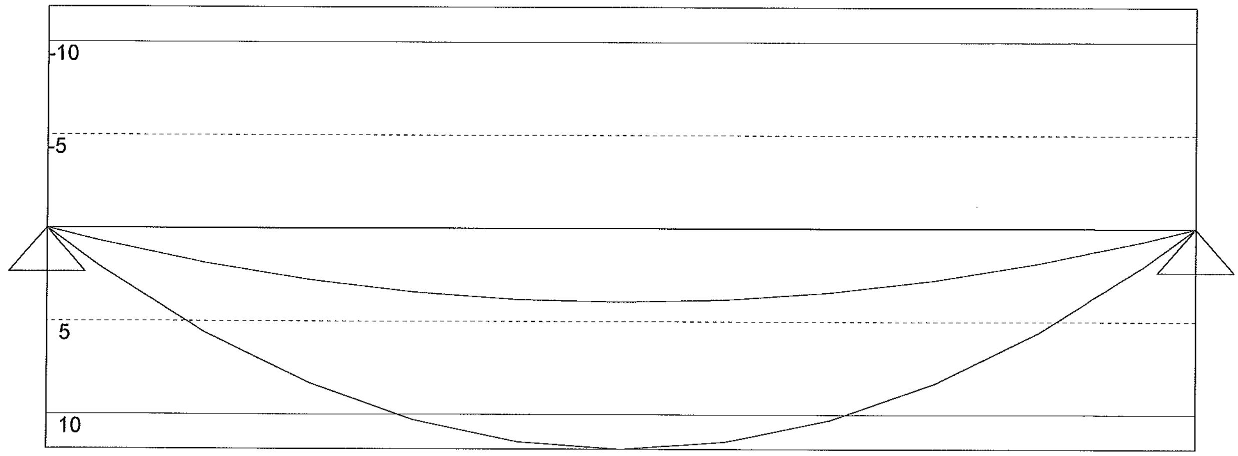

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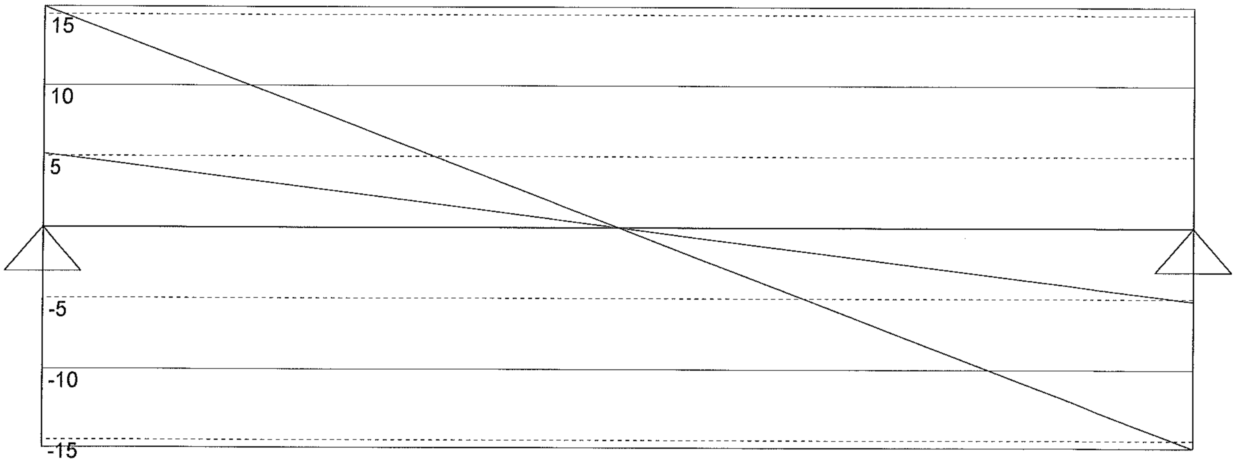

Load factor of dead load= 1 Load factor of imposed load= 1
Load width 3.4 (m) (by which the loads
has been multiplied during calculation)
Max/Min reactions of beam [kN]
15,552 15,555
5,184 5,185

KER 76 x 300 B 2 Cf=1,00 Design method: Allowable stress design
Increasing factor of the allowable stress 1,09
Factored Moment/Moment capacity [kNm] 11,861 22,385 53 %
Factored shear force/shear capacity [kN] 15,552 28,189 55 %

(2) LVL 1 1/2 x 11 7/8

Deflection due to unfactored load (Deflection limit L/360)
6,5 mm (77 %)

Beam Id: Lot £ 70 - MB 24

Structural Engineer:

Date 24-05-2018

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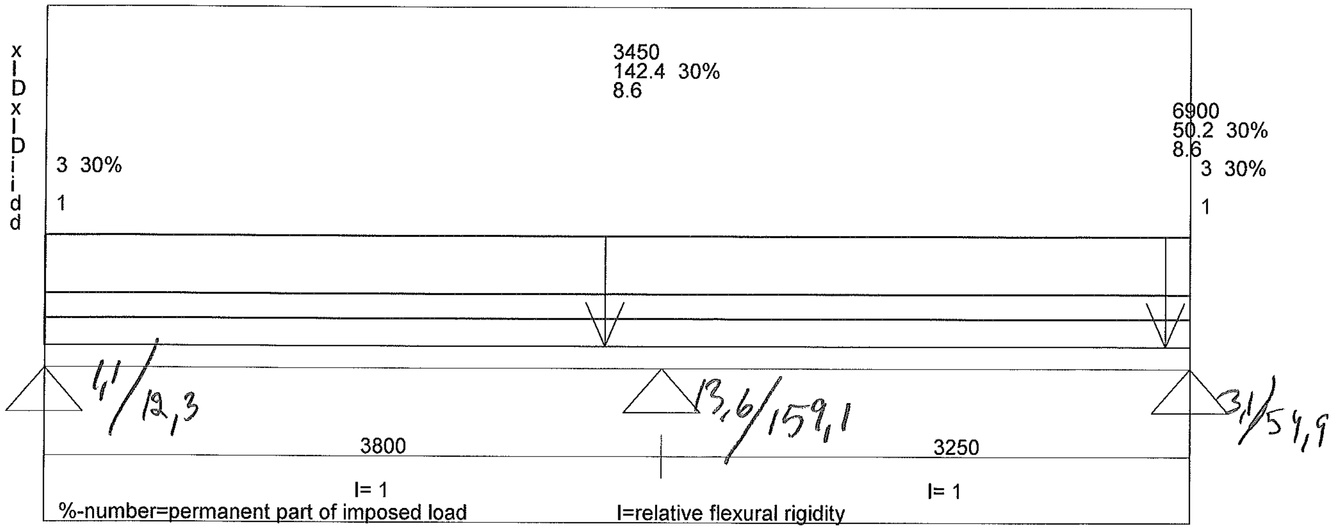

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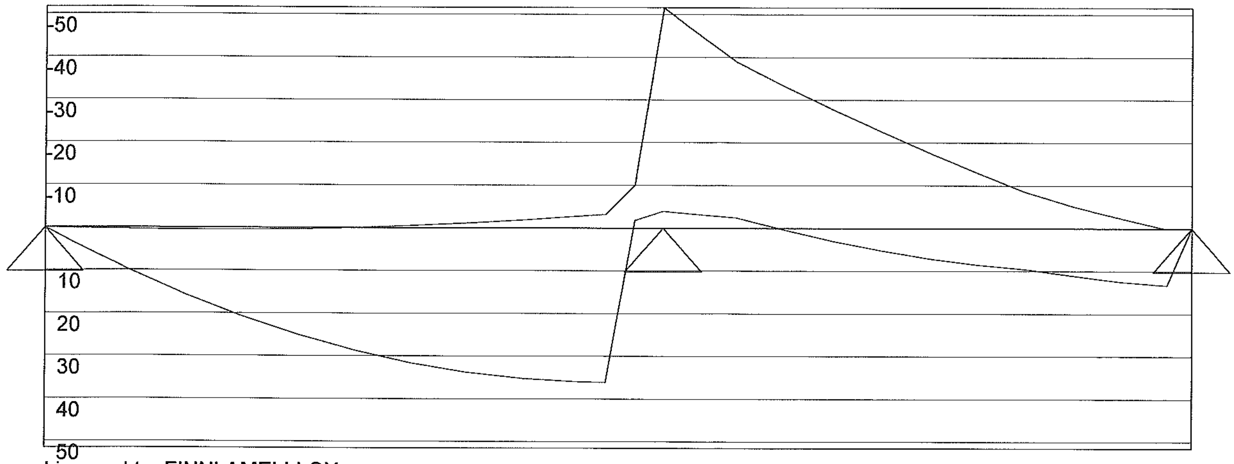

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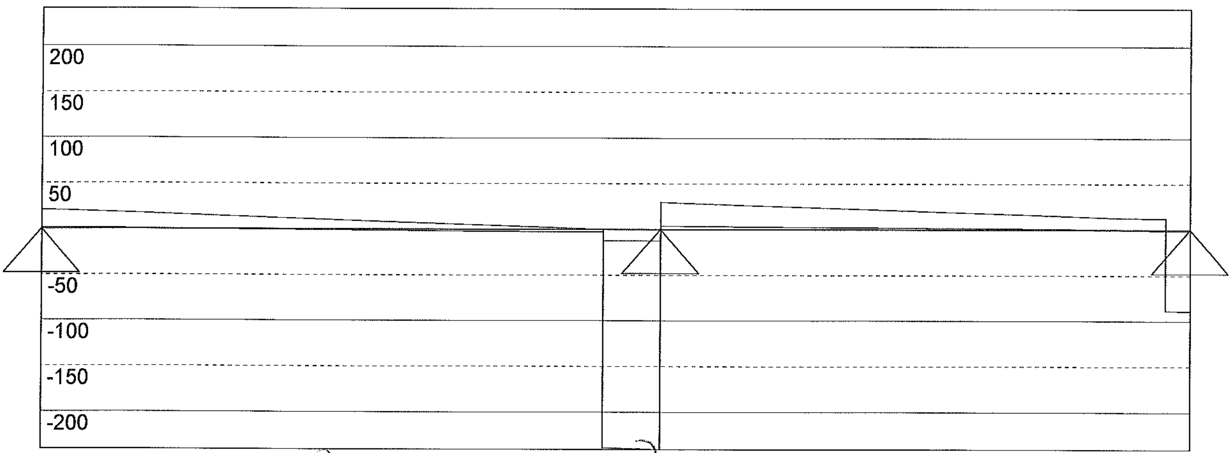

Load factor of dead load= 1.2 Load factor of imposed load= 1.6
Load width 1 (m) (by which the loads
has been multiplied during calculation)
Max/Min reactions of beam [kN]
20,815 270,987 89,367
1,141 16,328 1,552

HEB 220 (Class of section=1/1) G= 71,5 I(cm4)=8091 W(cm3)=736 fy=235
Factored Moment/Moment capacity [kNm] 51,553 194,580 26 %
Factored shear force/shear capacity [kN] 241,190 273,258 88 %

W8x 58

Deflection due to unfactored load (Deflection limit L/360)
1,8 mm (17 %) 0,3 mm (3 %)
Attention! Ultimate limit design! Remember the load factors!!

Beam Id: Lot # 70 - MB 25

Date 24-05-2018

Structural Engineer:

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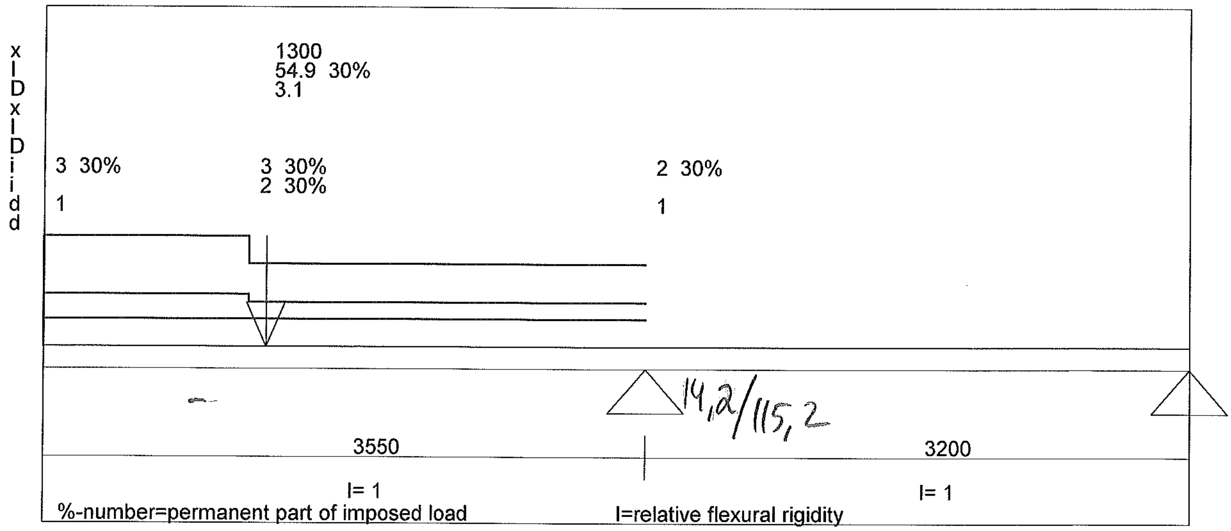
G/Q



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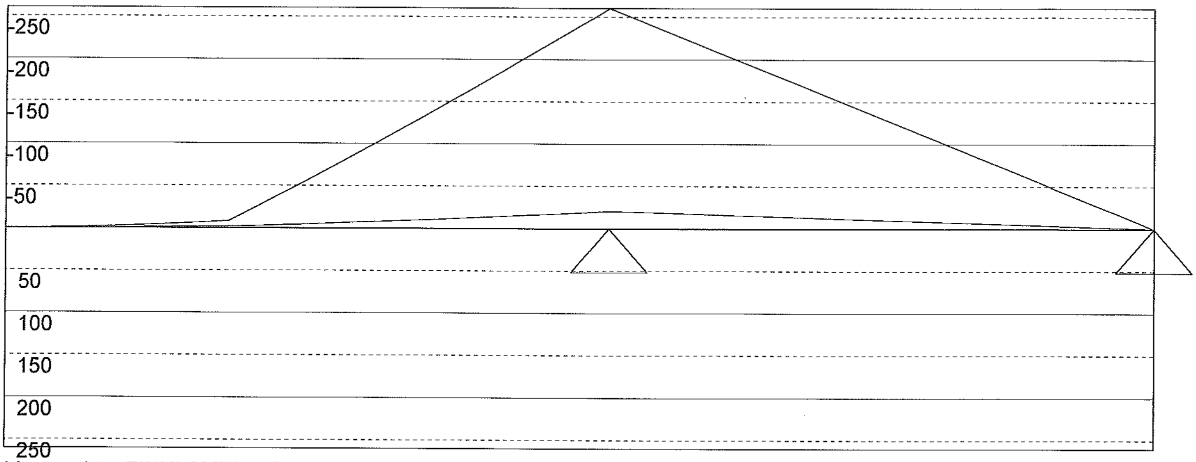

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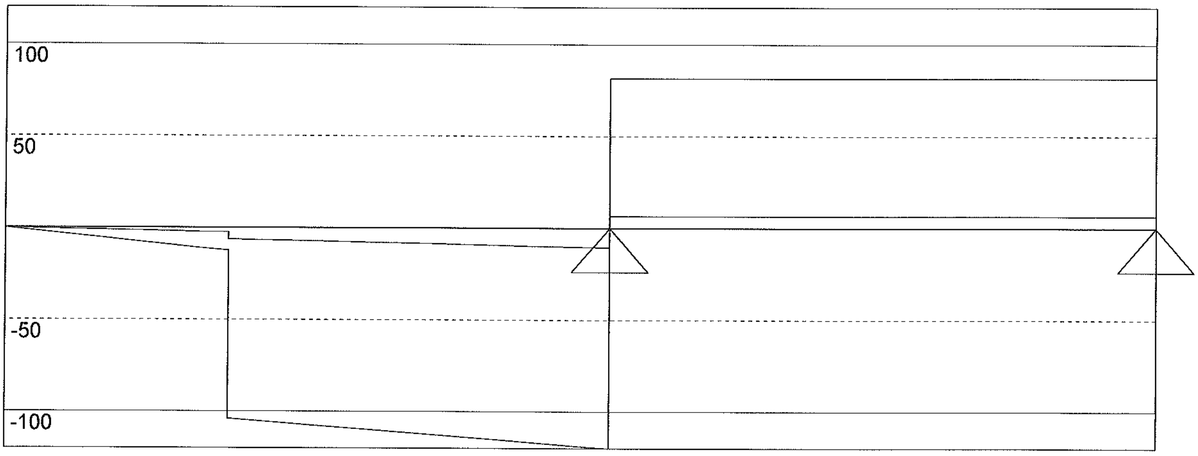

Load factor of dead load= 1.2 Load factor of imposed load= 1.6
Load width 1.625 (m) (by which the loads
has been multiplied during calculation)
Max/Min reactions of beam [kN]
201,392 -6,455
17,097 -81,333

HEB 320 (Class of section=1/1) G= 127 I(cm4)=30823 W(cm3)=1930 fy=235
Factored Moment/Moment capacity [kNm] 260,239 502,900 52 %
Factored shear force/shear capacity [kN] 120,059 485,604 25 %

W 12x87

Deflection due to unfactored load (Deflection limit L/360)/L/180)!
17,9 mm (91 %) -0,2 mm (2 %)
Attention! Ultimate limit design! Remember the load factors!!

Beam Id: LOT#70 MB 4 GLULAM

Date 19-06-2018

Structural Engineer:

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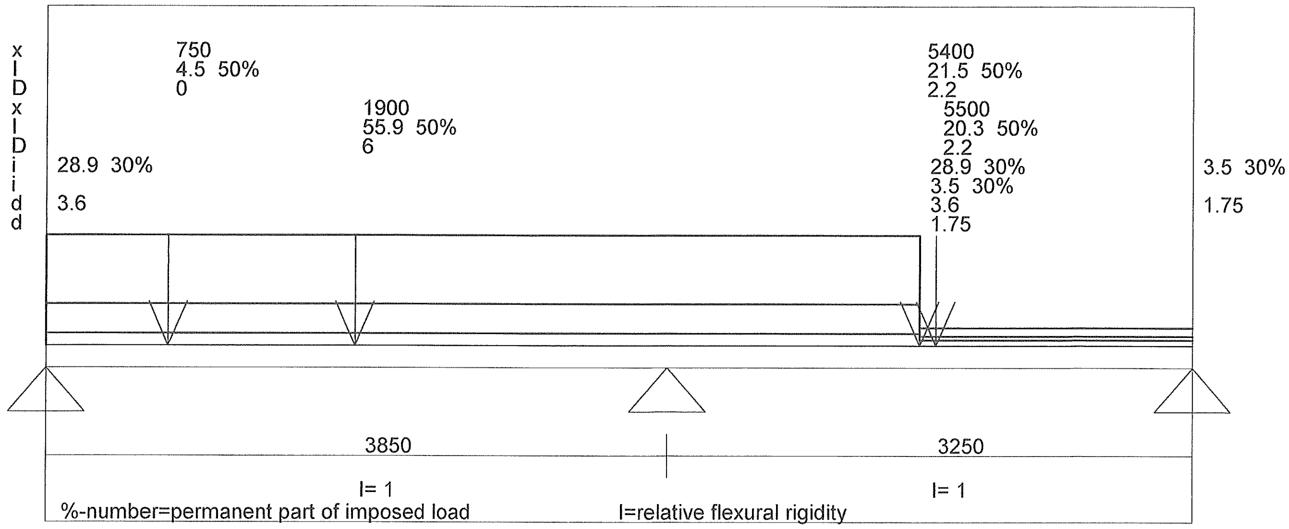

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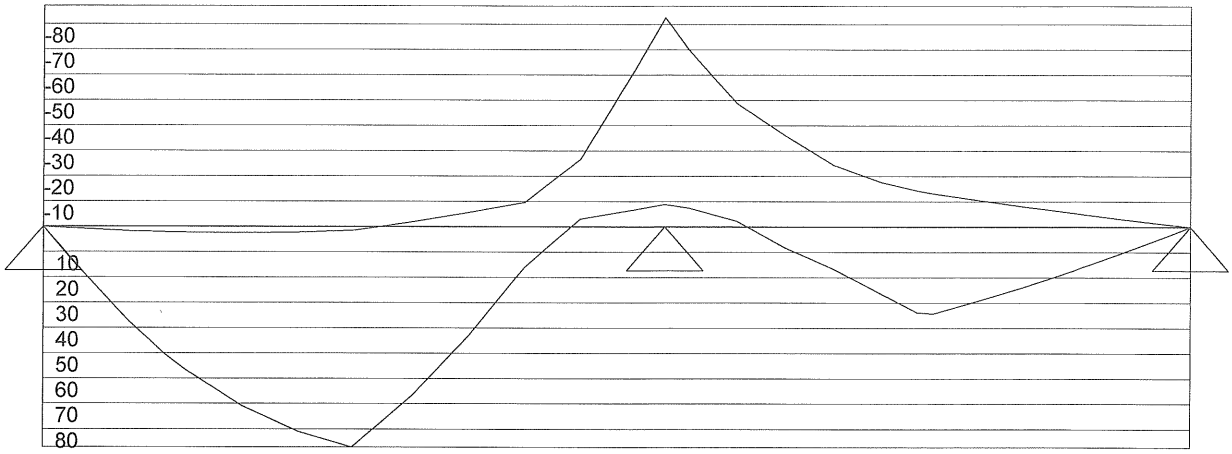

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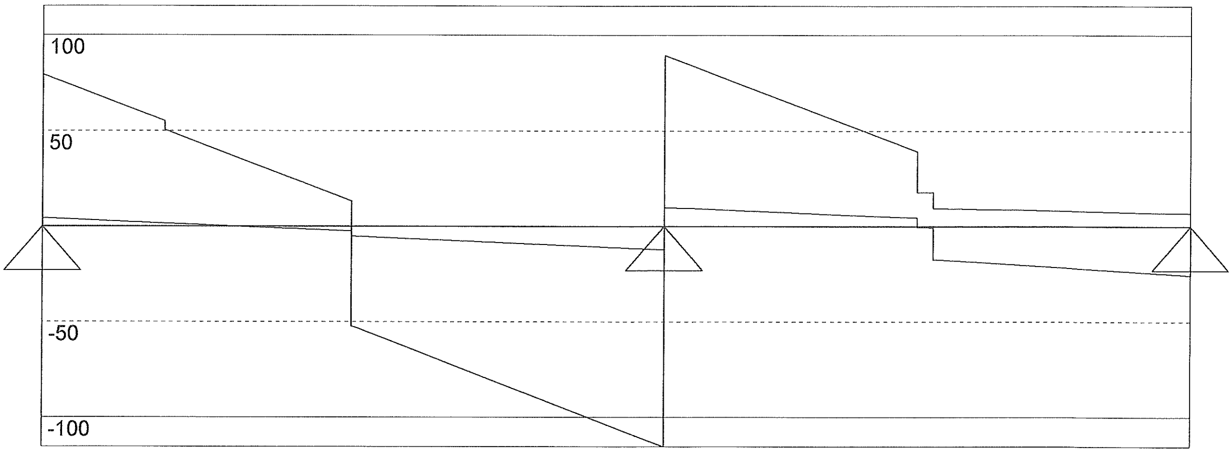

Load factor of dead load= 1 Load factor of imposed load= 1
Load width 1 (m) (by which the loads
has been multiplied during calculation)
Max/Min reactions of beam [kN]
79,439 205,008 25,666
4,281 22,173 -6,793

L40 380 x 390 B 2 Cf=0,97 Design method: Allowable stress design
Increasing factor of the allowable stress 1,03
Factored Moment/Moment capacity [kNm] 87,130 142,984 61 %
Factored shear force/shear capacity [kN] 115,429 116,534 99 %

GL (2) 7 1/2" x 15 3/8"

Deflection due to unfactored load (Deflection limit L/360)
6,6 mm (62 %) 1,6 mm (18 %)

Beam Id: LOT#70 MB 5 GLULAM

Date 19-06-2018

Structural Engineer:

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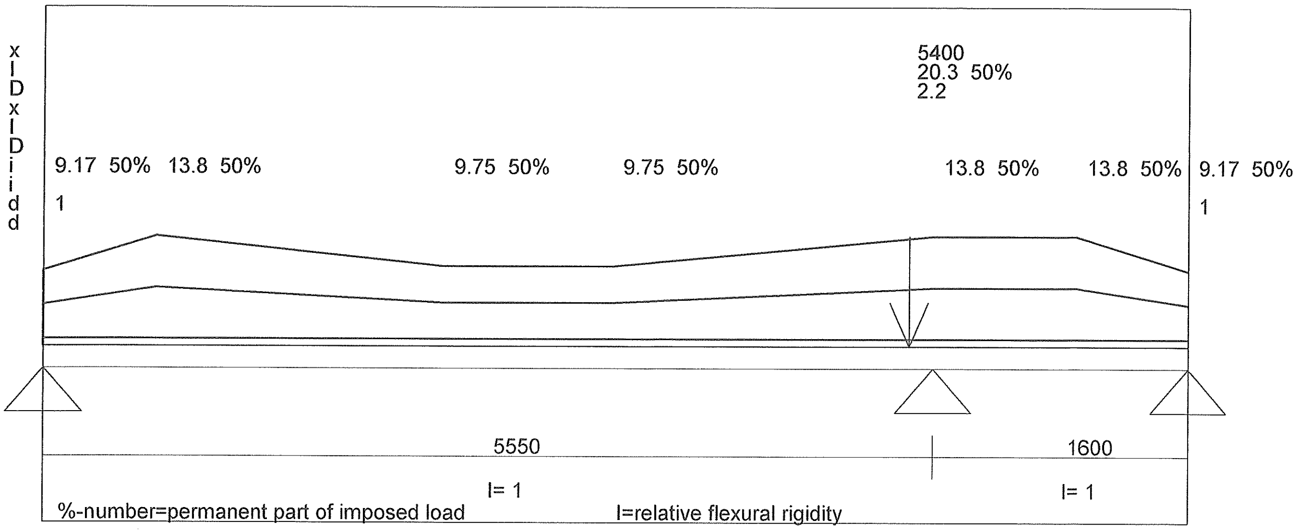

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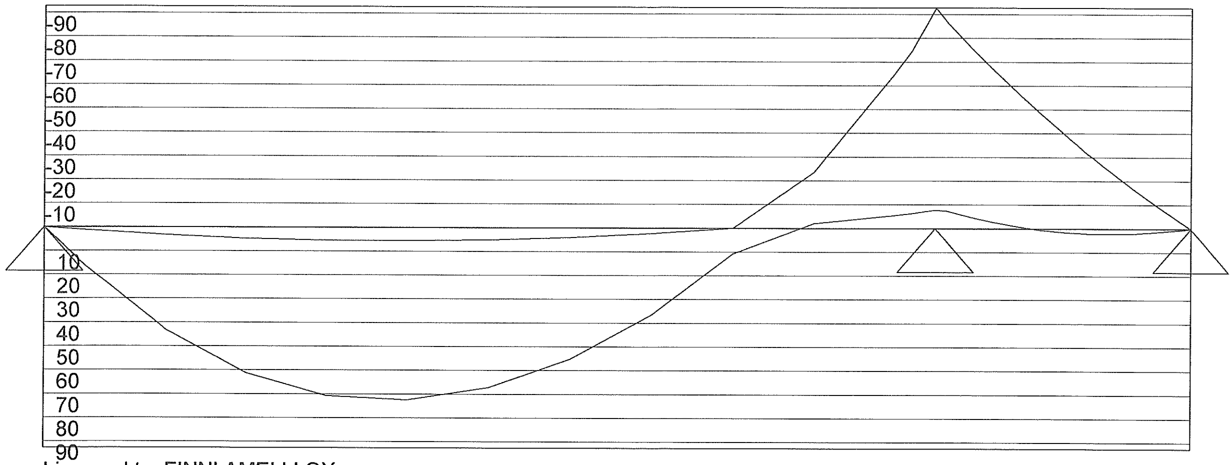

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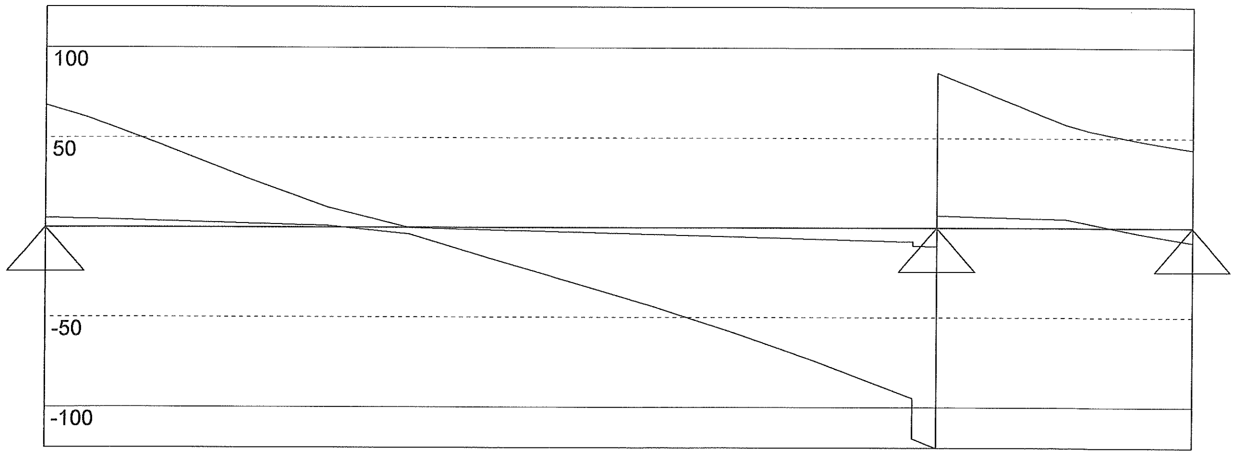

Load factor of dead load= 1 Load factor of imposed load= 1
Load width 2.45 (m) (by which the loads
has been multiplied during calculation)
Max/Min reactions of beam [kN]
67,927 209,092 8,202
5,246 17,155 -43,405

L40 380 x 430 B 2 Cf=0,96 Design method: Allowable stress design
Increasing factor of the allowable stress 1,02
Factored Moment/Moment capacity [kNm] 92,662 171,091 54 %
Factored shear force/shear capacity [kN] 122,747 127,850 96 %

GL (2) 7 1/2" x 17"

Deflection due to unfactored load (Deflection limit L/360)
9,0 mm (58 %) 0,0 mm (0 %)

Beam Id: LOT#70 MB 6 GLULAM

Date 19-06-2018

Structural Engineer:

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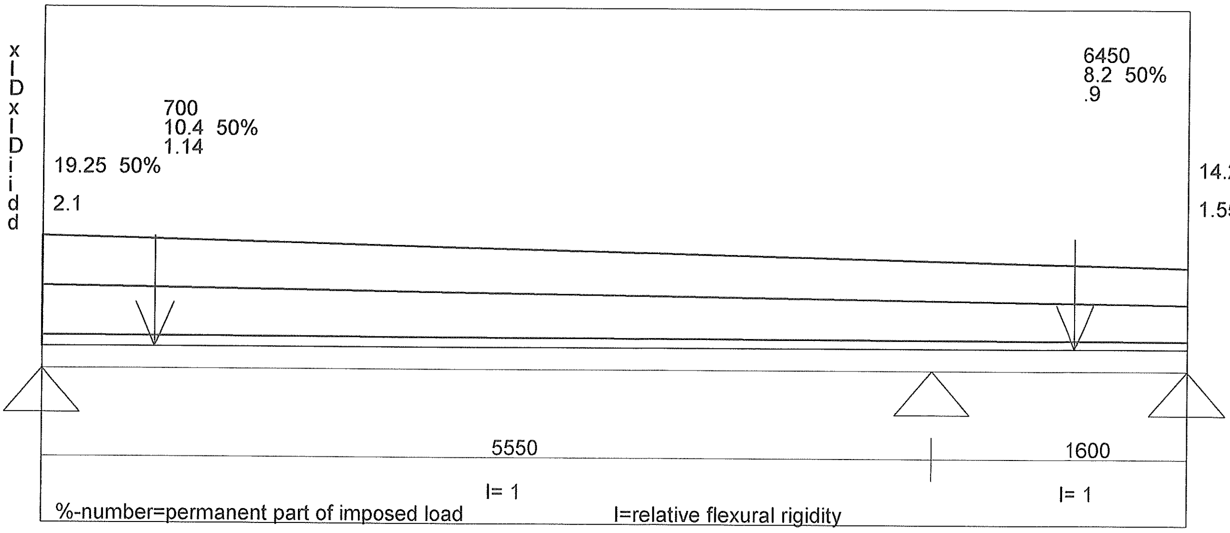

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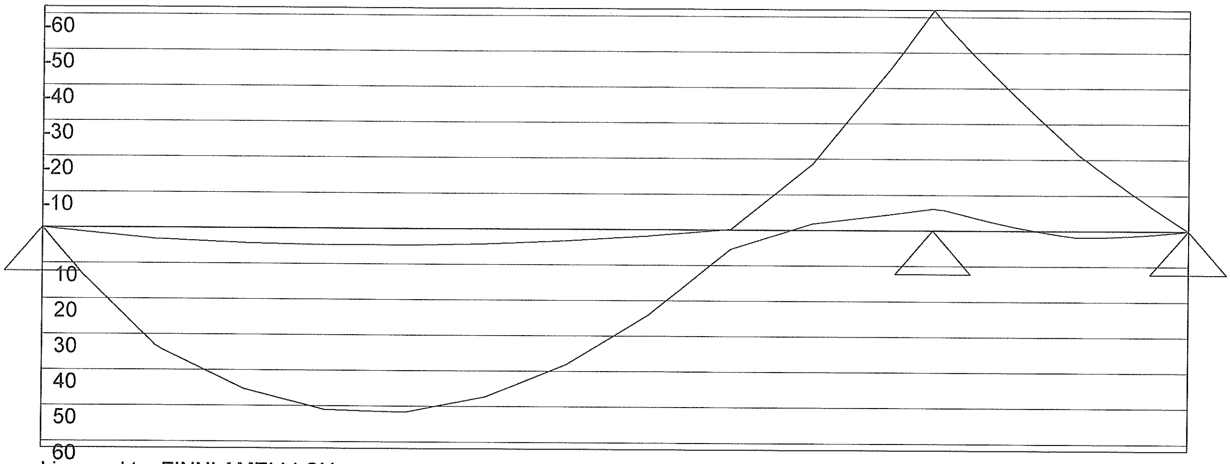

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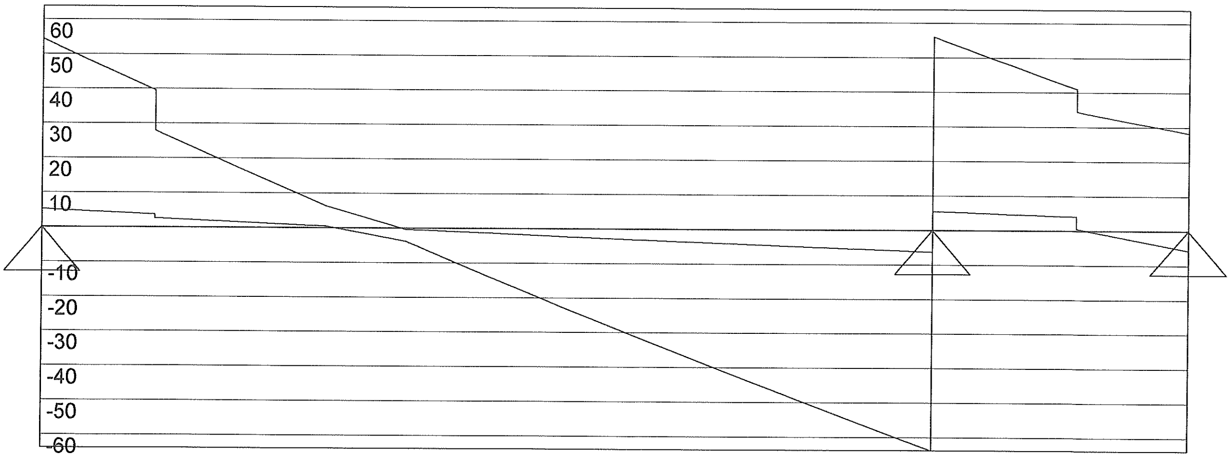

Load factor of dead load= 1 Load factor of imposed load= 1
Load width 1 (m) (by which the loads
has been multiplied during calculation)
Max/Min reactions of beam [kN]
54,274 119,761 5,612
5,186 11,786 -28,297

L40 380 x 430 B 2 Cf=0,96 Design method: Allowable stress design
Increasing factor of the allowable stress 1,03
Factored Moment/Moment capacity [kNm] 61,922 171,870 36 %
Factored shear force/shear capacity [kN] 63,808 128,433 50 %

GL (2) 7 1/2" x 17"

Deflection due to unfactored load (Deflection limit L/360)
6,3 mm (41 %) 0,0 mm (0 %)

Beam Id: LOT#70 MB 7 LVL

Date 19-06-2018

Structural Engineer:

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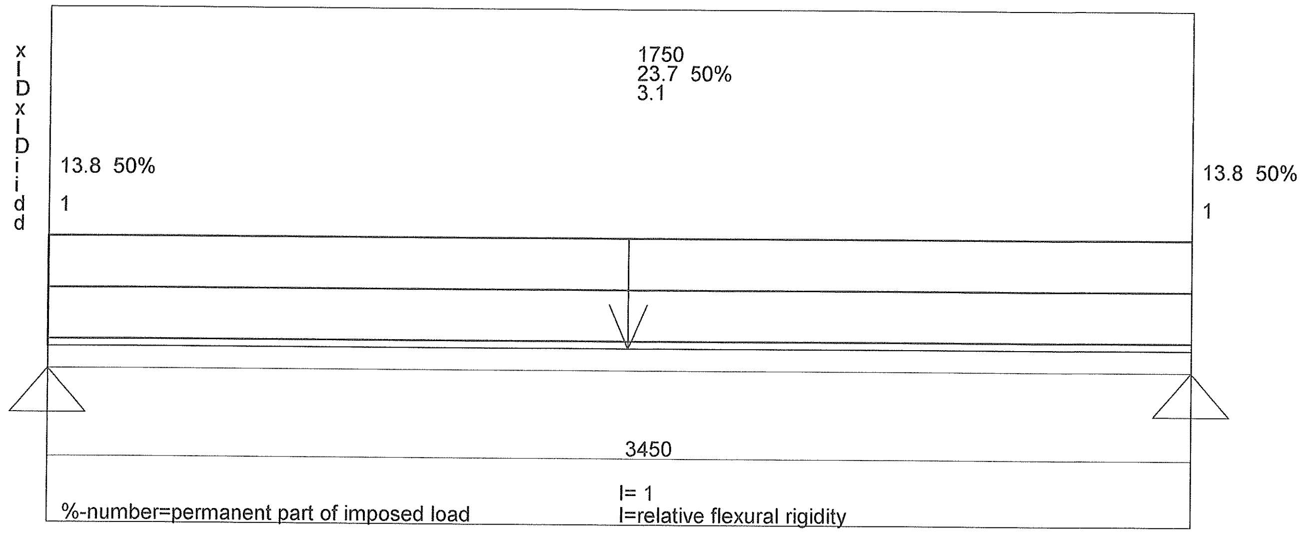

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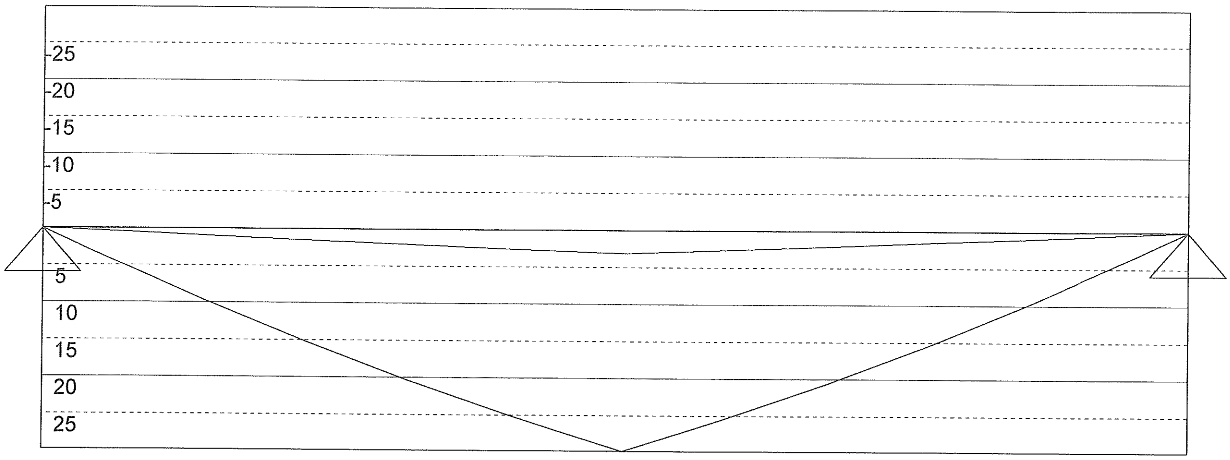

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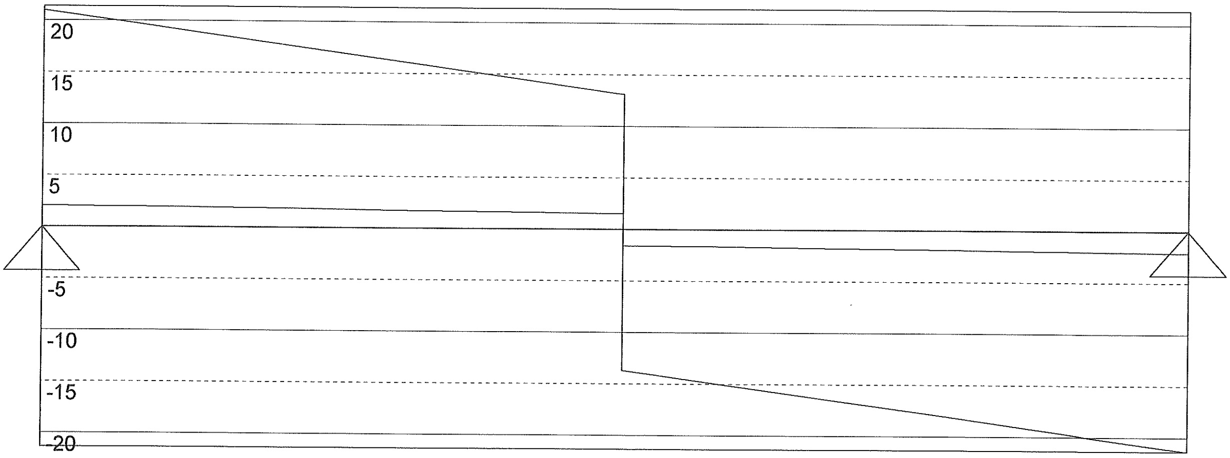

Load factor of dead load= 1 Load factor of imposed load= 1
Load width .305 (m) (by which the loads
has been multiplied during calculation)
Max/Min reactions of beam [kN]
20,991 21,381
2,054 2,099

KER 152 x 300 B 2 Cf=1,00 Design method: Allowable stress design
Increasing factor of the allowable stress 1,03
Factored Moment/Moment capacity [kNm] 29,825 42,071 71 %
Factored shear force/shear capacity [kN] 21,379 52,978 40 %

(4) LVL 1 1/2" x 11 7/8"

Deflection due to unfactored load (Deflection limit L/360)
8,8 mm (92 %)

Beam Id: LOT#70 MB 13 GLU LAM

Date 19-06-2018

Structural Engineer:

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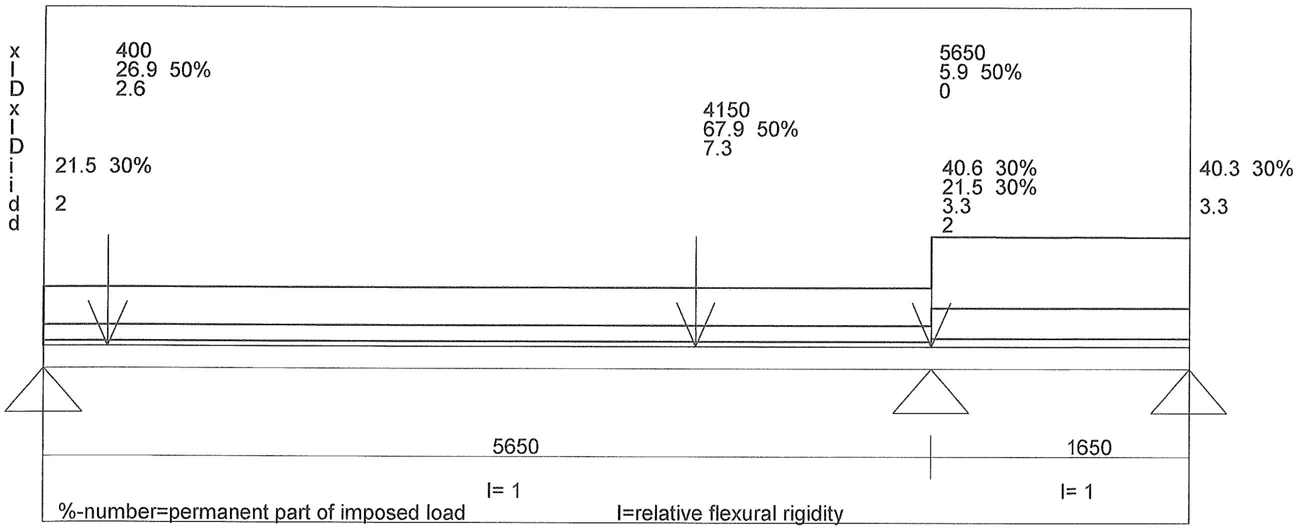

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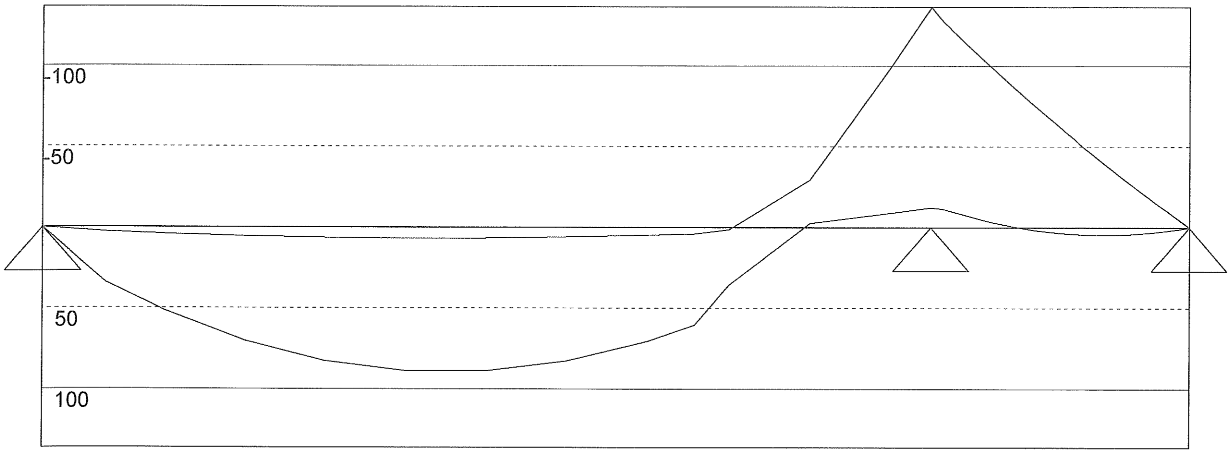

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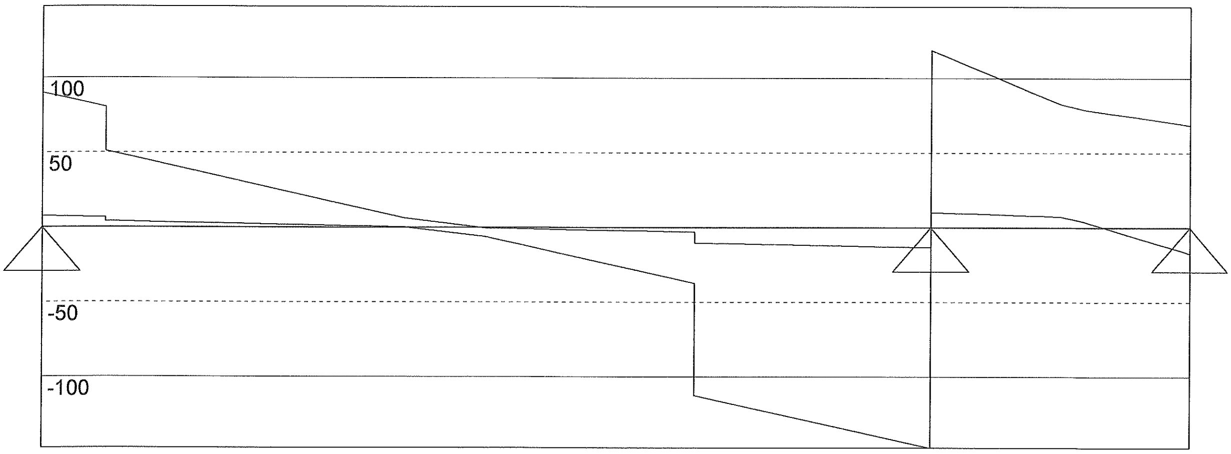

Load factor of dead load= 1 Load factor of imposed load= 1
Load width 1 (m) (by which the loads
has been multiplied during calculation)
Max/Min reactions of beam [kN]
89,999 272,494 17,303
7,447 23,510 -68,593

L40 380 x 515 B 2 Cf=0,94 Design method: Allowable stress design
Increasing factor of the allowable stress 1,02
Factored Moment/Moment capacity [kNm] 136,336 240,880 57 %
Factored shear force/shear capacity [kN] 147,826 153,335 96 %

(2) GL 7 1/2" x 20 1/4"

Deflection due to unfactored load (Deflection limit L/360)
7,4 mm (47 %) 0,0 mm (0 %)

Beam Id: LOT#70 HDR #1

Date 20-06-2018

Structural Engineer:

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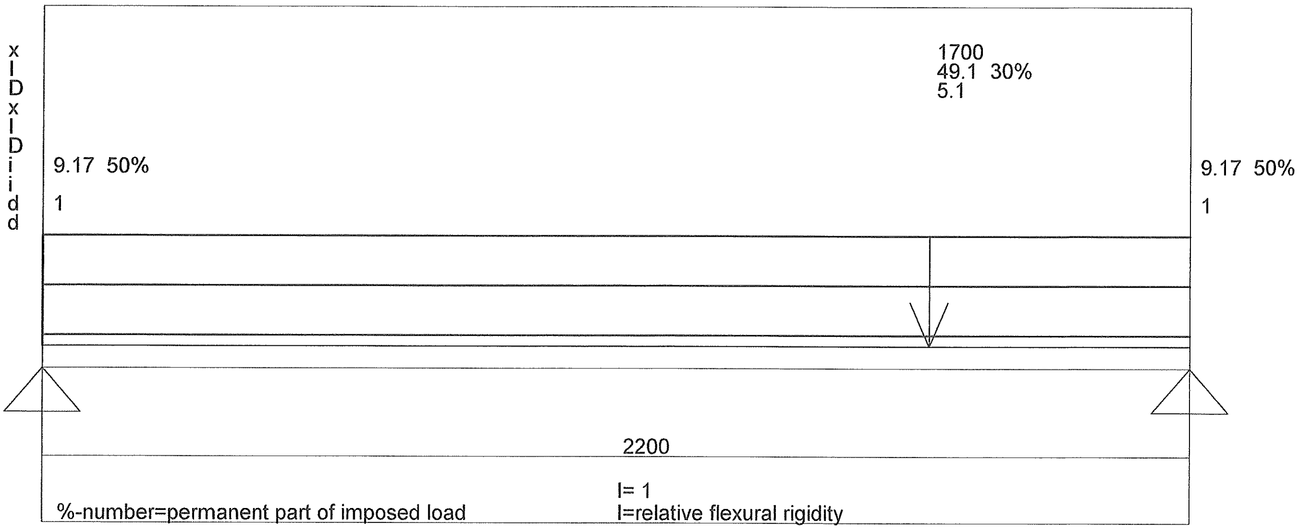

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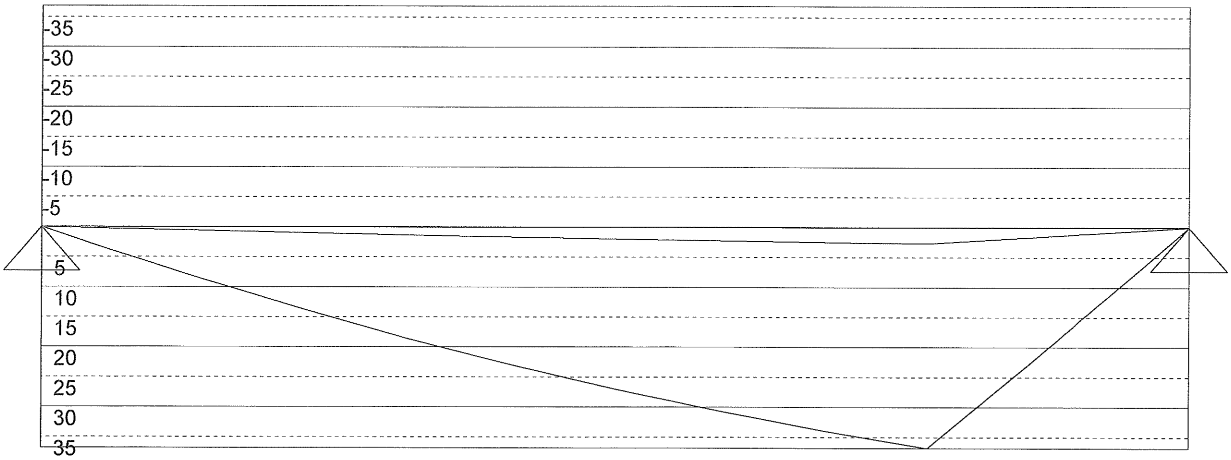

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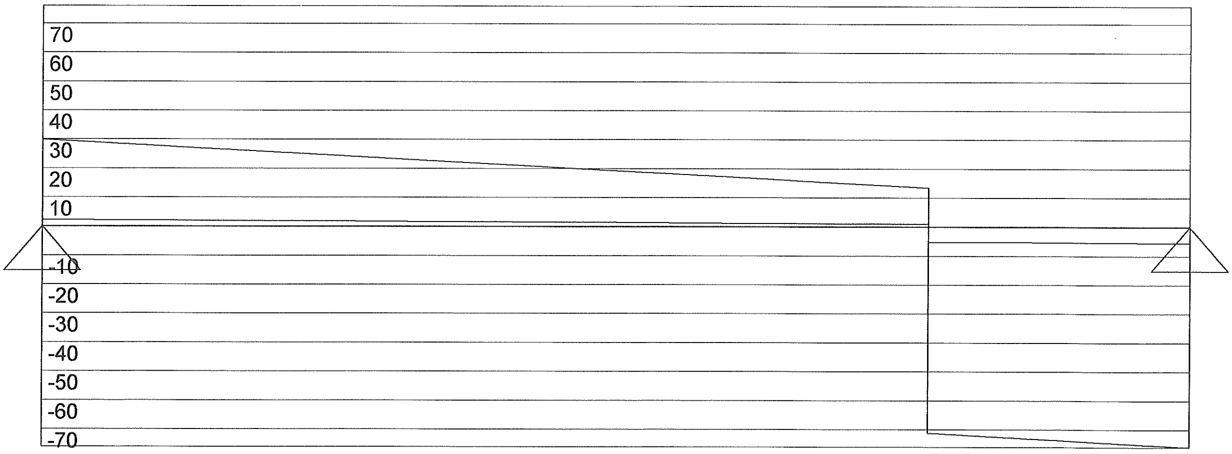

Load factor of dead load= 1.2 Load factor of imposed load= 1.6
Load width .61 (m) (by which the loads
has been multiplied during calculation)
Max/Min reactions of beam [kN]
29,893 76,083
2,196 5,534

T24 328 x 260 B 2 Cf=1,00 Design method: Ultimate limit design
Factored Moment/Moment capacity [kNm] 36,831 56,853 65 %
Factored shear force/shear capacity [kN] 76,081 87,467 87 %

(2) L065

Deflection due to unfactored load (Deflection limit L/360)
3,1 mm (50 %)

Beam Id: LOT#70 HDR #2

Date 20-06-2018

Structural Engineer:

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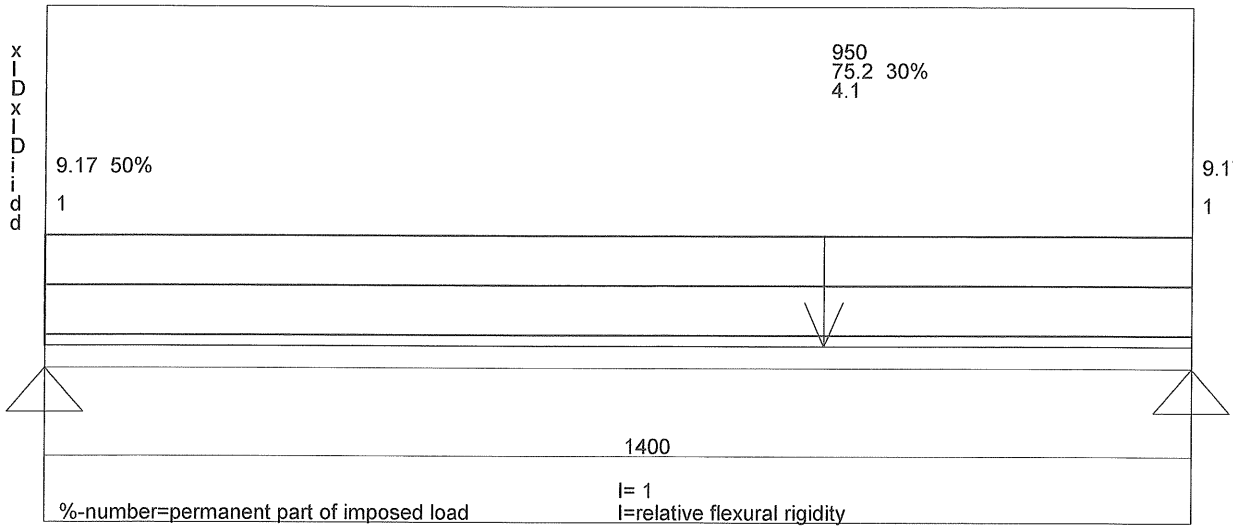

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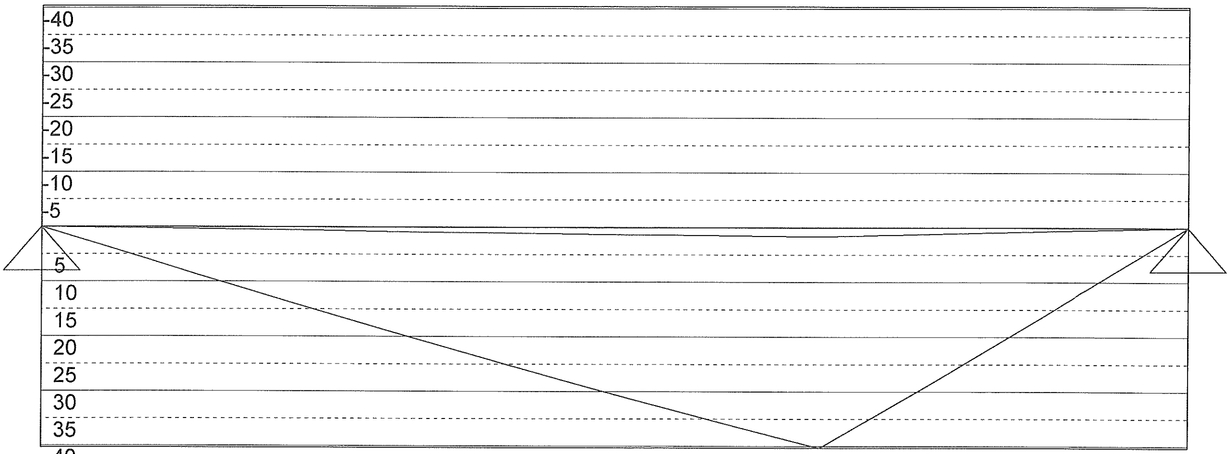

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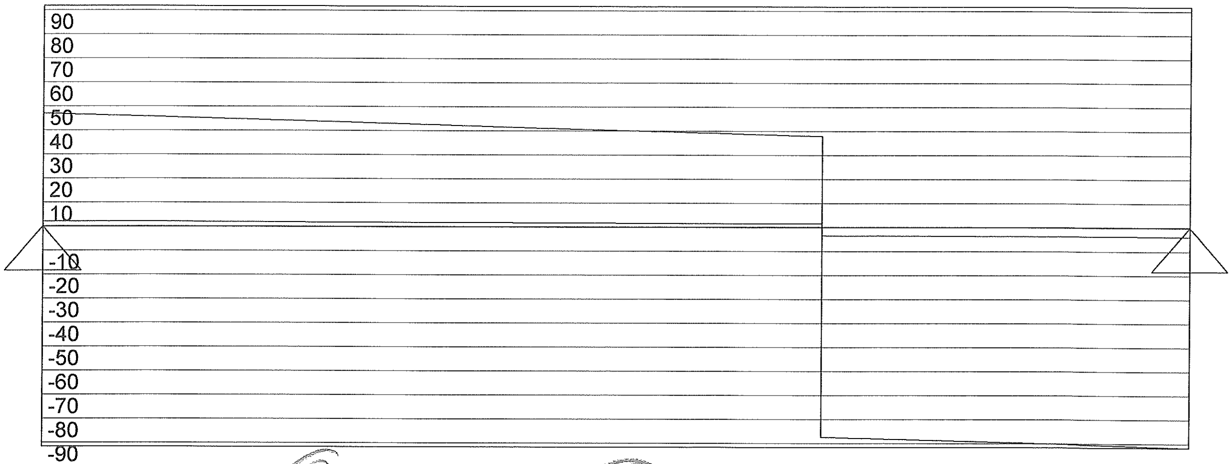

Load factor of dead load= 1.2 Load factor of imposed load= 1.6
Load width .61 (m) (by which the loads
has been multiplied during calculation)
Max/Min reactions of beam [kN]
47,031 91,761
2,094 3,851

T24 492 x 260 B 2 Cf=1,00 Design method: Ultimate limit design
Factored Moment/Moment capacity [kNm] 40,312 85,280 47 %
Factored shear force/shear capacity [kN] 91,759 131,200 70 %

(3) Logs

Deflection due to unfactored load (Deflection limit L/360)
0,9 mm (22 %)

Beam Id: LOT#70 HDR #3

Date 20-06-2018

Structural Engineer:

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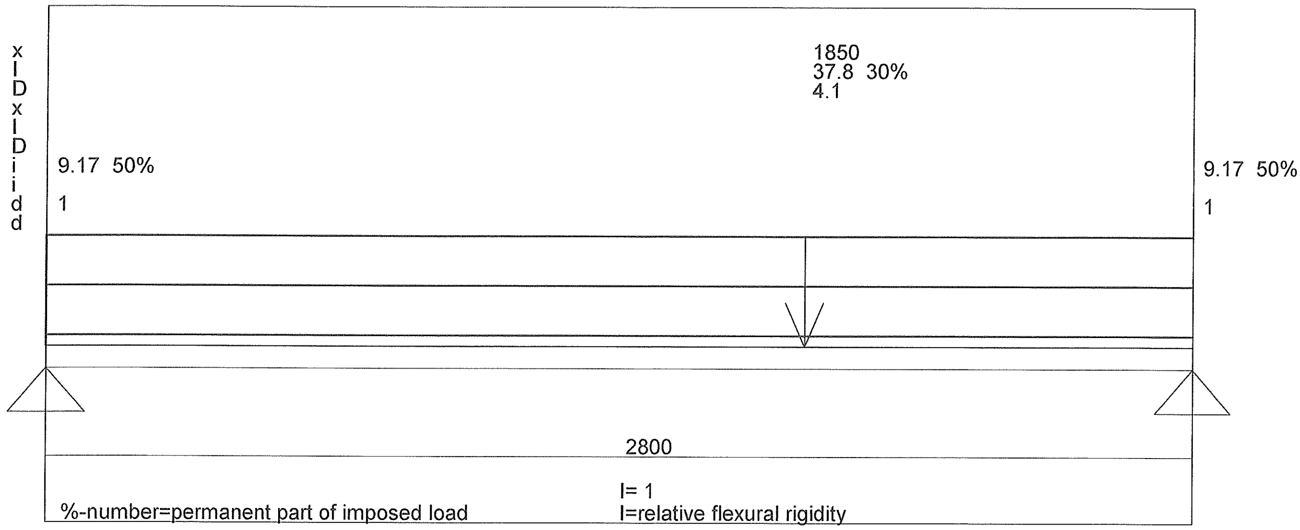

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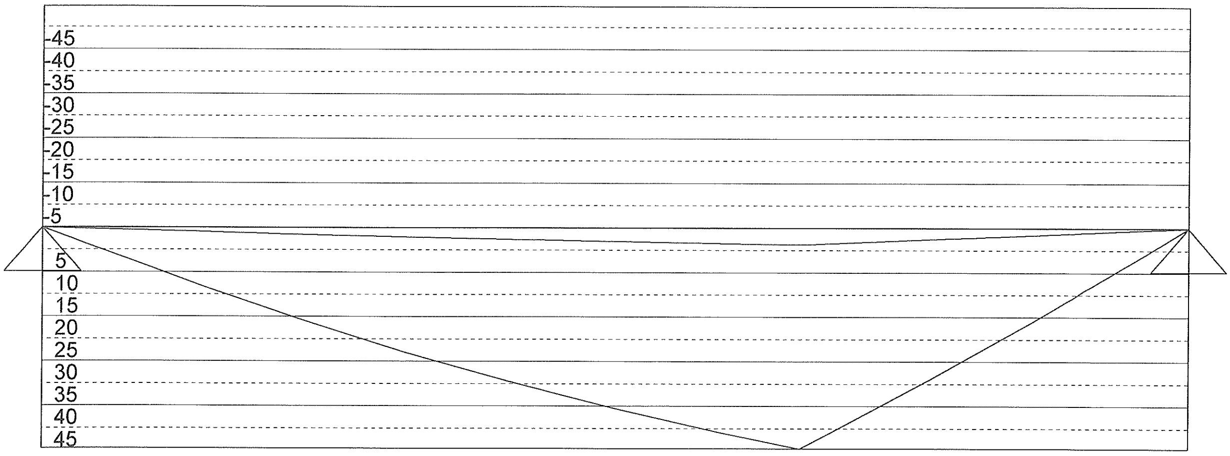

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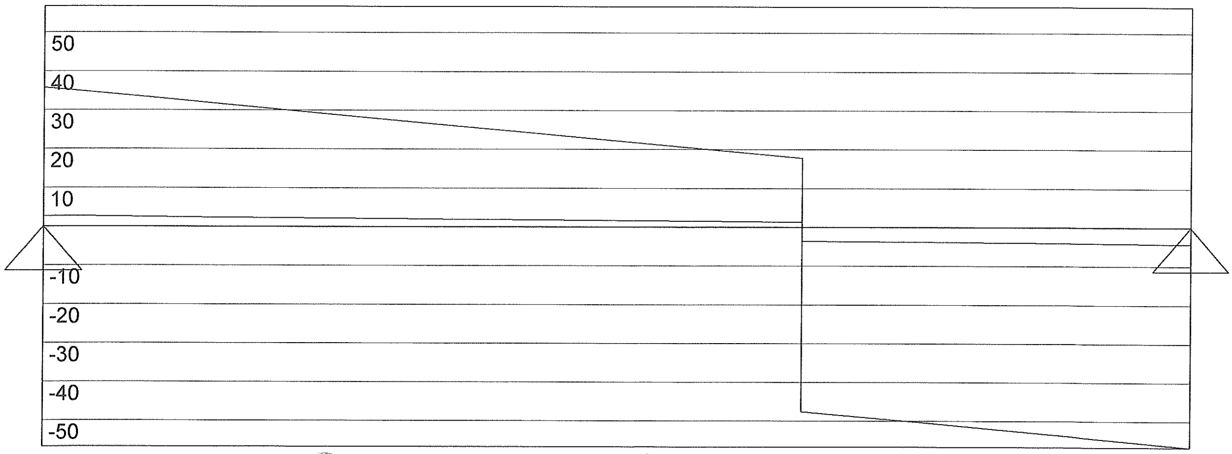

Load factor of dead load= 1.2 Load factor of imposed load=1.6
Load width .61 (m) (by which the loads
has been multiplied during calculation)
Max/Min reactions of beam [kN]
35,741 56,764
2,694 4,275

T24 328 x 260 B 2 Cf=1,00 Design method: Ultimate limit design
Factored Moment/Moment capacity [kNm] 49,557 56,853 87 %
Factored shear force/shear capacity [kN] 56,761 87,467 65 %

2 Logs

Deflection due to unfactored load (Deflection limit L/360)
6,9 mm (88 %)

Beam Id: LOT#70 HDR # 4

Date 20-06-2018

Structural Engineer:

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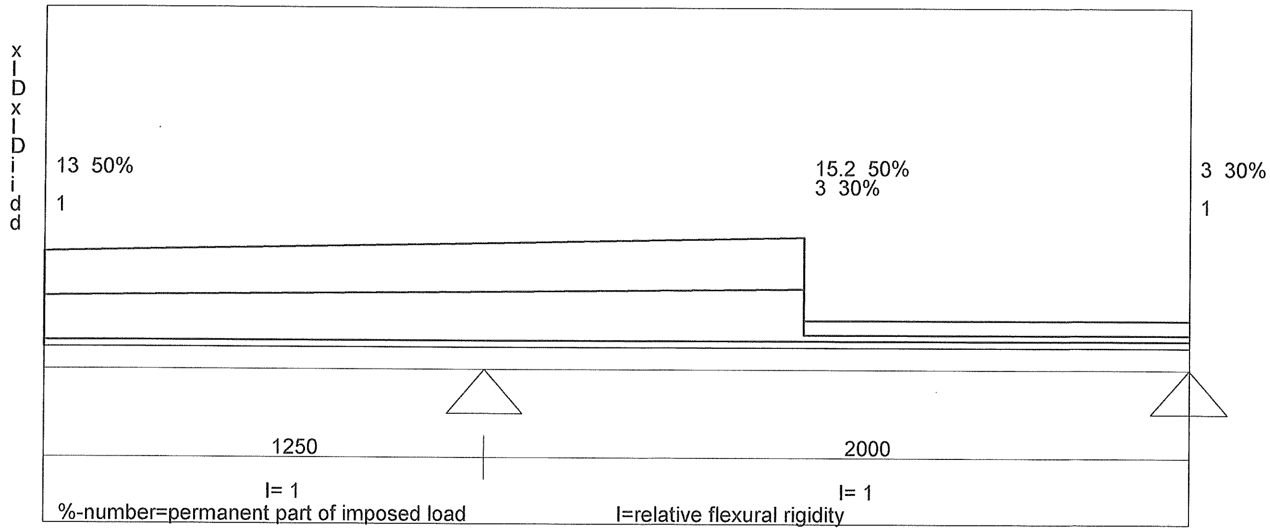

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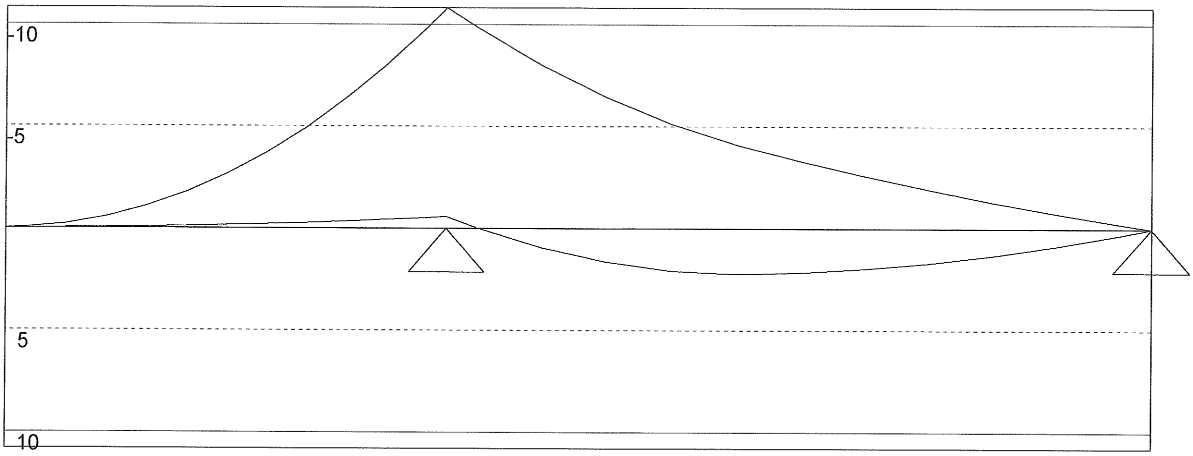

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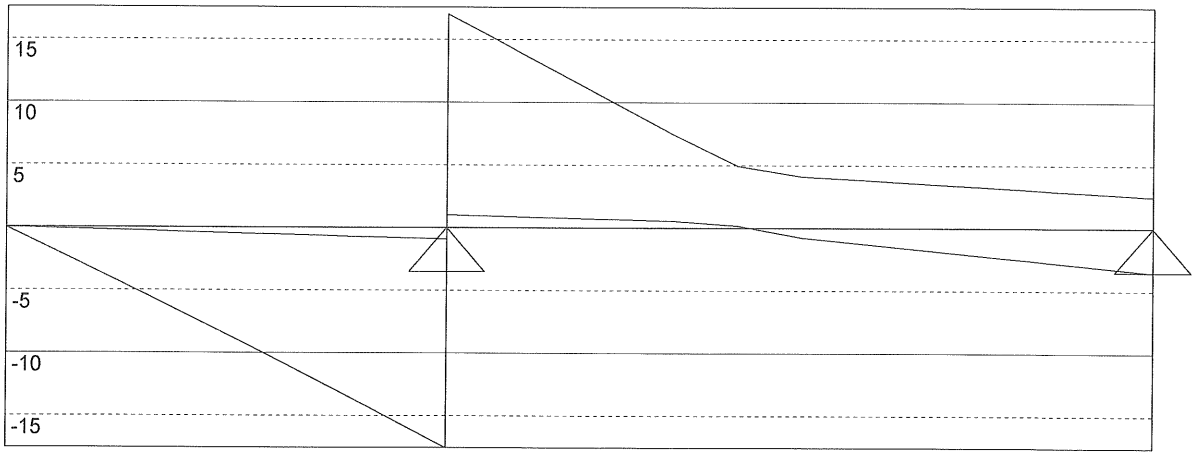

Load factor of dead load=1.2 Load factor of imposed load= 1.6
Load width .61 (m) (by which the loads
has been multiplied during calculation)
Max/Min reactions of beam [kN]
34,577 3,552
1,933 -2,500

T24 164 x 260 B 2 Cf=1,00 Design method: Ultimate limit design
Factored Moment/Moment capacity [kNm] 10,807 28,427 38 %
Factored shear force/shear capacity [kN] 17,553 43,733 40 %

(1) LO6

Deflection due to unfactored load (Deflection limit L/360)/L/180)!
4,5 mm (65 %) 0,4 mm (6 %)

Beam Id: LOT#70 HDR #5

Date 20-06-2018

Structural Engineer:

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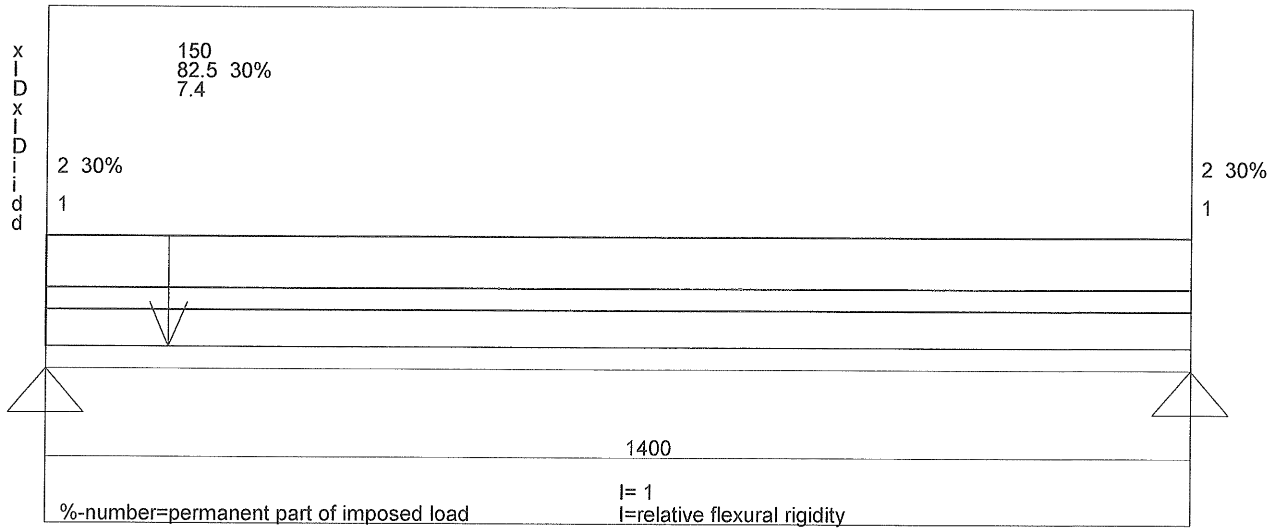

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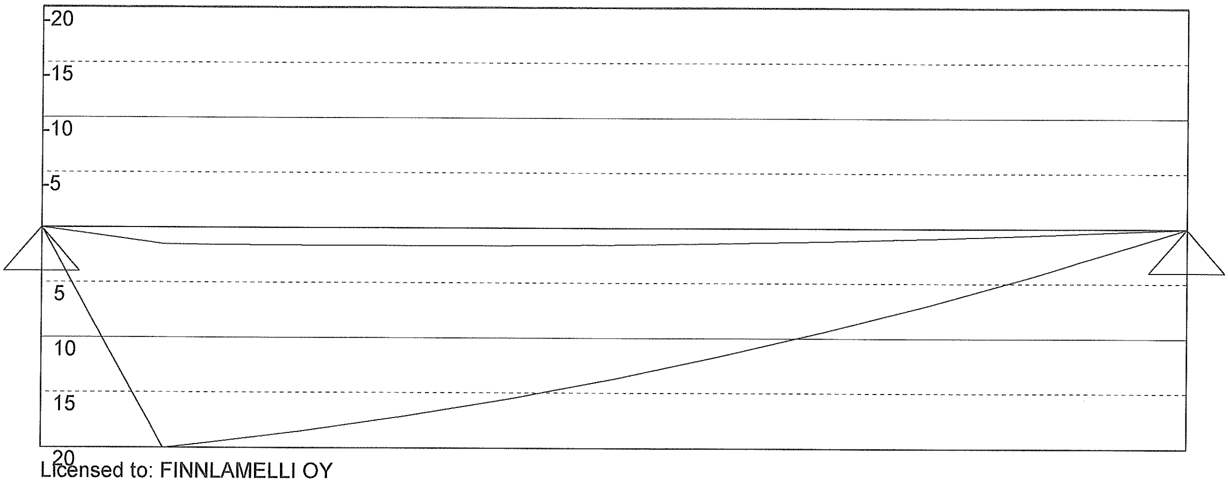

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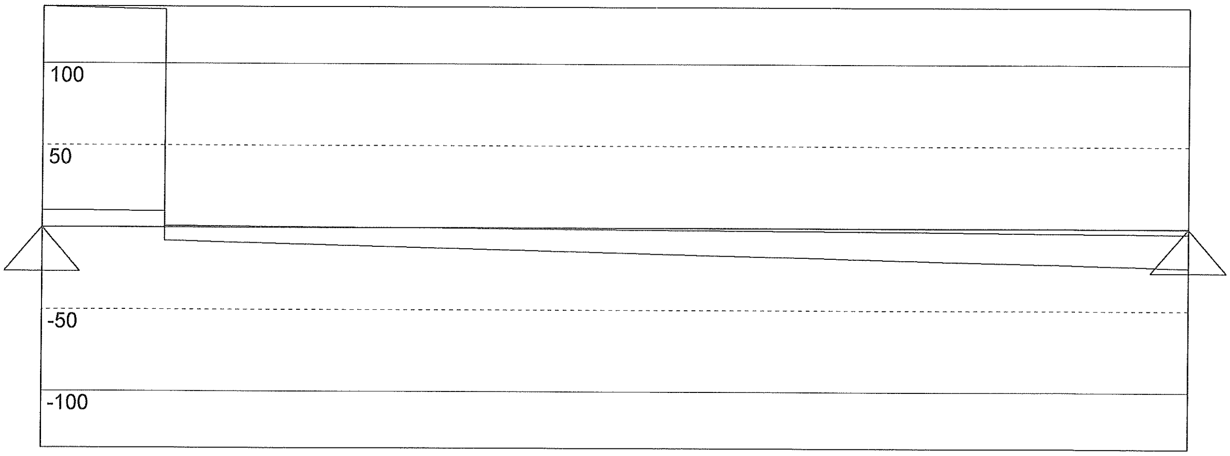

Load factor of dead load= 1.2 Load factor of imposed load= 1.6
Load width 2.9 (m) (by which the loads
has been multiplied during calculation)
Max/Min reactions of beam [kN]
134,715 24,026
10,364 3,387

T24 656 x 260 B 2 Cf=1,00 Design method: Ultimate limit design
Factored Moment/Moment capacity [kNm] 20,064 113,707 18 %
Factored shear force/shear capacity [kN] 134,715 174,933 77 %

(4) LOGS

Deflection due to unfactored load (Deflection limit L/360)
0,3 mm (8 %)

SIMPSON Strong-Tie®

Anchor Designer™
Software
Version 2.6.6682.21

Company:	Ensign Engineering	Date:	6/11/2018
Engineer:	Alex Hawkins, P.E.	Page:	1/5
Project:			
Address:	45 W 10000 S Ste. 500		
Phone:	801-255-0529		
E-mail:	ahawkins@ensigneng.com		

1.Project information

Customer company:
Customer contact name:
Customer e-mail:
Comment:

Project description:
Location: Utah
Fastening description:

2. Input Data & Anchor Parameters

General
Design method:ACI 318-14
Units: Imperial units

Anchor Information:
Anchor type: Concrete screw
Material: Carbon Steel
Diameter (inch): 0.750
Nominal Embedment depth (inch): 6.000
Effective Embedment depth, h_{ef} (inch): 4.640
Code report: ICC-ES ESR-2713
Anchor category: 1
Anchor ductility: No
h_{min} (inch): 9.58
c_{ac} (inch): 7.00
C_{min} (inch): 1.75
S_{min} (inch): 3.00

Base Material
Concrete: Normal-weight
Concrete thickness, h (inch): 120.00
State: Cracked
Compressive strength, f'c (psi): 3000
$\Psi_{c,V}$: 1.0
Reinforcement condition: B tension, B shear
Supplemental reinforcement: Not applicable
Reinforcement provided at corners: No
Ignore concrete breakout in tension: No
Ignore concrete breakout in shear: No
Ignore 6do requirement: Not applicable
Build-up grout pad: No

Base Plate
Length x Width x Thickness (inch): 10.00 x 10.00 x 0.81

Recommended Anchor
Anchor Name: Titen HD® - 3/4"Ø Titen HD, hnom:6" (152mm)
Code Report: ICC-ES ESR-2713

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Software
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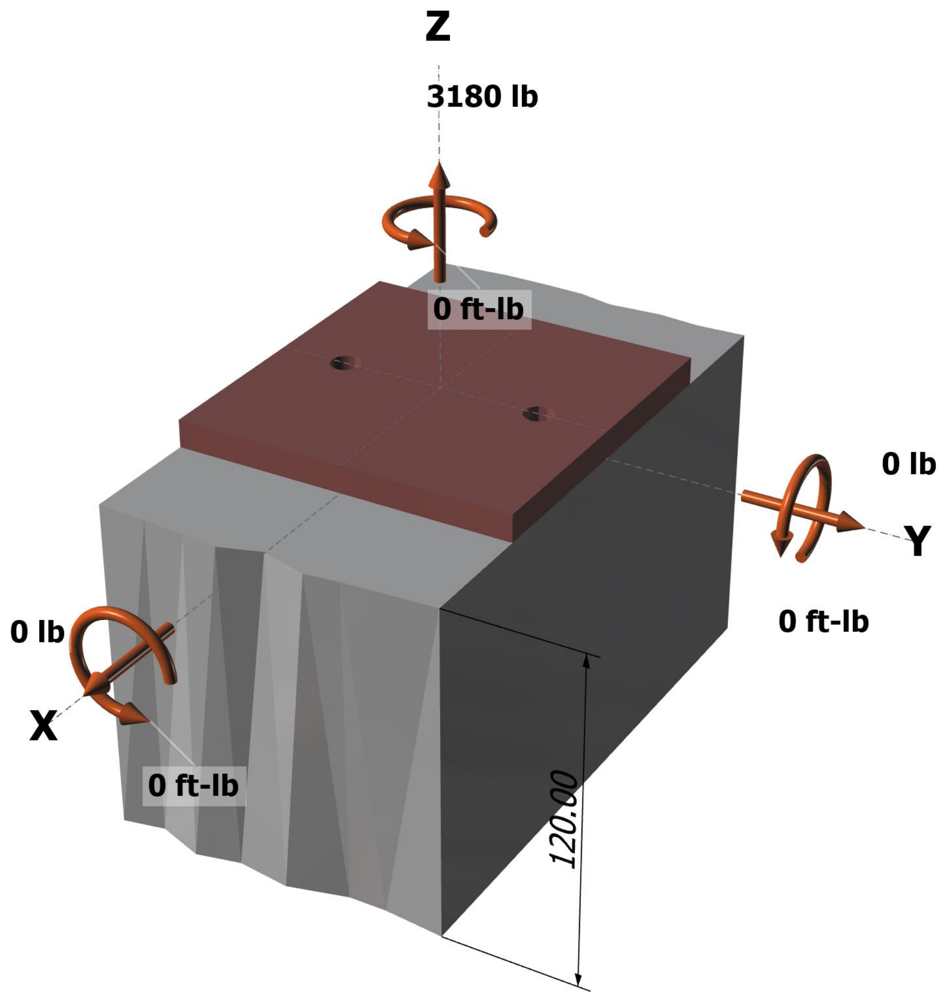
Company:	Ensign Engineering	Date:	6/11/2018
Engineer:	Alex Hawkins, P.E.	Page:	2/5
Project:			
Address:	45 W 10000 S Ste. 500		
Phone:	801-255-0529		
E-mail:	ahawkins@ensigneng.com		

Load and Geometry
Load factor source: ACI 318 Section 5.3
Load combination: not set
Seismic design: No
Anchors subjected to sustained tension: Not applicable
Apply entire shear load at front row: No
Anchors only resisting wind and/or seismic loads: No

Strength level loads:

N_{ua} [lb]: 3180
V_{uax} [lb]: 0
V_{uay} [lb]: 0
M_{ux} [ft-lb]: 0
M_{uy} [ft-lb]: 0
M_{uz} [ft-lb]: 0

<Figure 1>



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Engineer:	Alex Hawkins, P.E.	Page:	3/5
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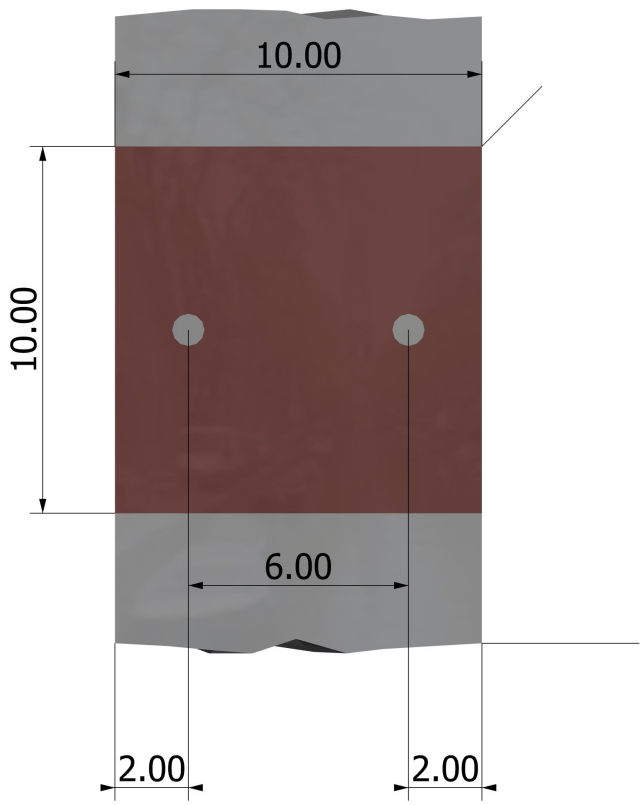
Address:	45 W 10000 S Ste. 500		
Phone:	801-255-0529		
E-mail:	ahawkins@ensigneng.com		

<Figure 2>



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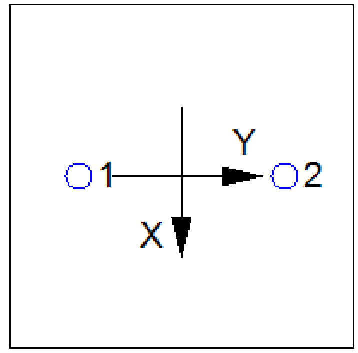
Company:	Ensign Engineering	Date:	6/11/2018
Engineer:	Alex Hawkins, P.E.	Page:	4/5
Project:			
Address:	45 W 10000 S Ste. 500		
Phone:	801-255-0529		
E-mail:	ahawkins@ensigneng.com		

3. Resulting Anchor Forces

Anchor	Tension load, N_{ua} (lb)	Shear load x, V_{uax} (lb)	Shear load y, V_{uay} (lb)	Shear load combined, $\sqrt{(V_{uax})^2+(V_{uay})^2}$ (lb)
1	1590.0	0.0	0.0	0.0
2	1590.0	0.0	0.0	0.0
Sum	3180.0	0.0	0.0	0.0

Maximum concrete compression strain (‰): 0.00
Maximum concrete compression stress (psi): 0
Resultant tension force (lb): 3180
Resultant compression force (lb): 0
Eccentricity of resultant tension forces in x-axis, e'_{Nx} (inch): 0.00
Eccentricity of resultant tension forces in y-axis, e'_{Ny} (inch): 0.00

<Figure 3>



4. Steel Strength of Anchor in Tension (Sec. 17.4.1)

N_{sa} (lb)	ϕ	ϕN_{sa} (lb)
45540	0.65	29601

5. Concrete Breakout Strength of Anchor in Tension (Sec. 17.4.2)

$N_b = k_c \lambda_a \sqrt{f'_c} h_{ef}^{1.5}$ (Eq. 17.4.2.2a)

k_c	λ_a	f'_c (psi)	h_{ef} (in)	N_b (lb)
17.0	1.00	3000	4.640	9307

$\phi N_{cbg} = \phi\, (A_{Nc} / A_{Nco})\, \Psi_{ec,N} \Psi_{ed,N} \Psi_{c,N} \Psi_{cp,N} N_b$ (Sec. 17.3.1 & Eq. 17.4.2.1b)

A_{Nc} (in^2)	A_{Nco} (in^2)	$c_{a,min}$ (in)	$\Psi_{ec,N}$	$\Psi_{ed,N}$	$\Psi_{c,N}$	$\Psi_{cp,N}$	N_b (lb)	ϕ	ϕN_{cbg} (lb)
139.20	193.77	2.00	1.000	0.786	1.00	1.000	9307	0.65	3417

6. Pullout Strength of Anchor in Tension (Sec. 17.4.3)

$\phi N_{pn} = \phi \Psi_{c,P} \lambda_a N_p (f'_c / 2{,}500)^n$ (Sec. 17.3.1, Eq. 17.4.3.1 & Code Report)

$\Psi_{c,P}$	λ_a	N_p (lb)	f'_c (psi)	n	ϕ	ϕN_{pn} (lb)
1.0	1.00	6820	3000	0.50	0.65	4856

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Company:	Ensign Engineering	Date:	6/11/2018
Engineer:	Alex Hawkins, P.E.	Page:	5/5
Project:			
Address:	45 W 10000 S Ste. 500		
Phone:	801-255-0529		
E-mail:	ahawkins@ensigneng.com		

11. Results

11. Interaction of Tensile and Shear Forces (Sec. D.7)?

Tension	Factored Load, N_{ua} (lb)	Design Strength, $øN_n$ (lb)	Ratio	Status
Steel	1590	29601	0.05	Pass
Concrete breakout	**3180**	**3417**	**0.93**	**Pass (Governs)**
Pullout	1590	4856	0.33	Pass

3/4"Ø Titen HD, hnom:6" (152mm) meets the selected design criteria.

12. Warnings

- Designer must exercise own judgement to determine if this design is suitable.

- Refer to manufacturer's product literature for hole cleaning and installation instructions.

Beam Id: Lot # 70 - RB /

Date 17-05-2018

Structural Engineer:

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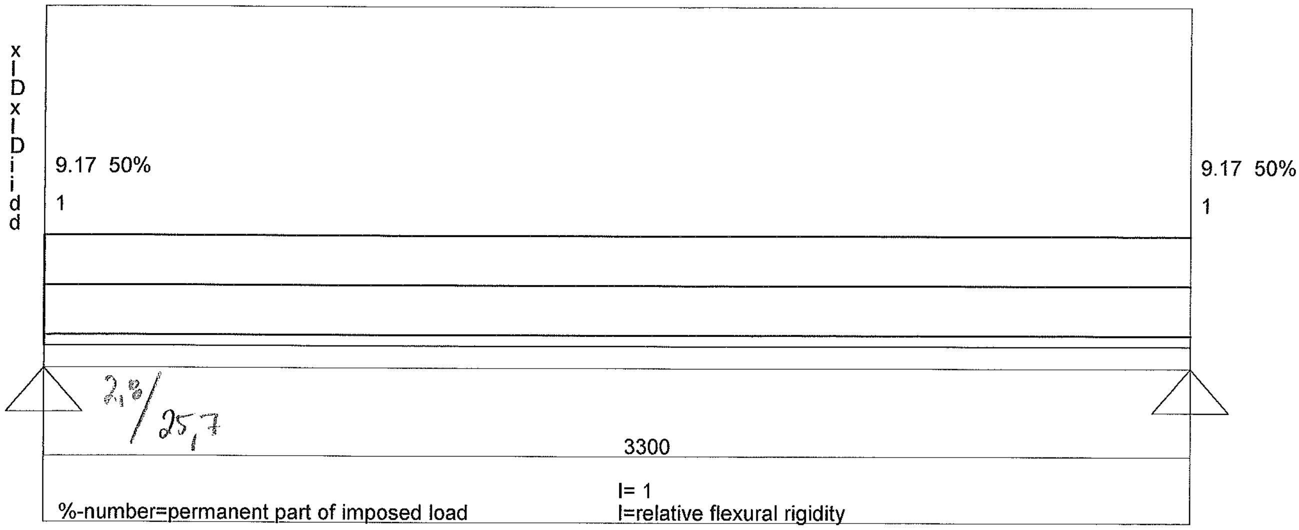

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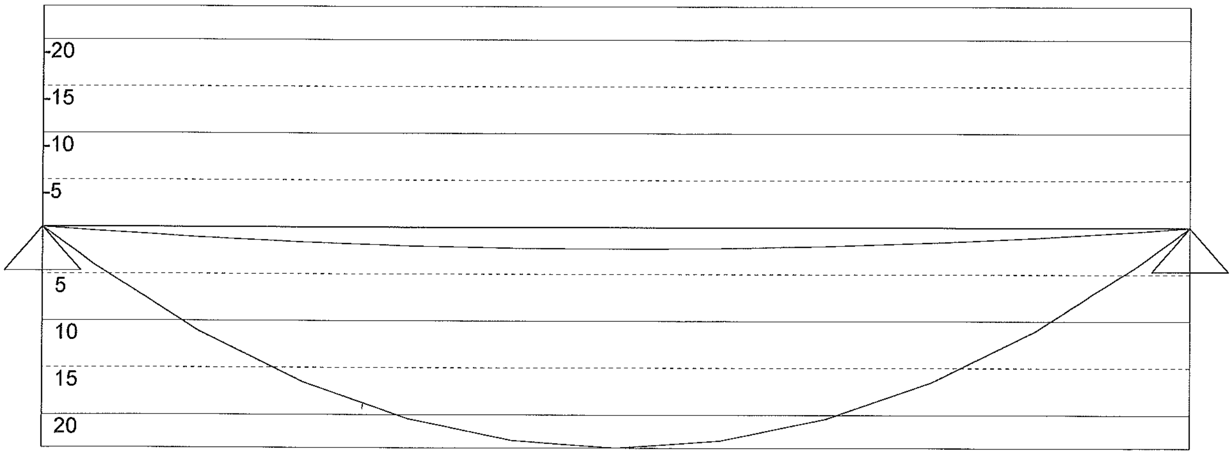

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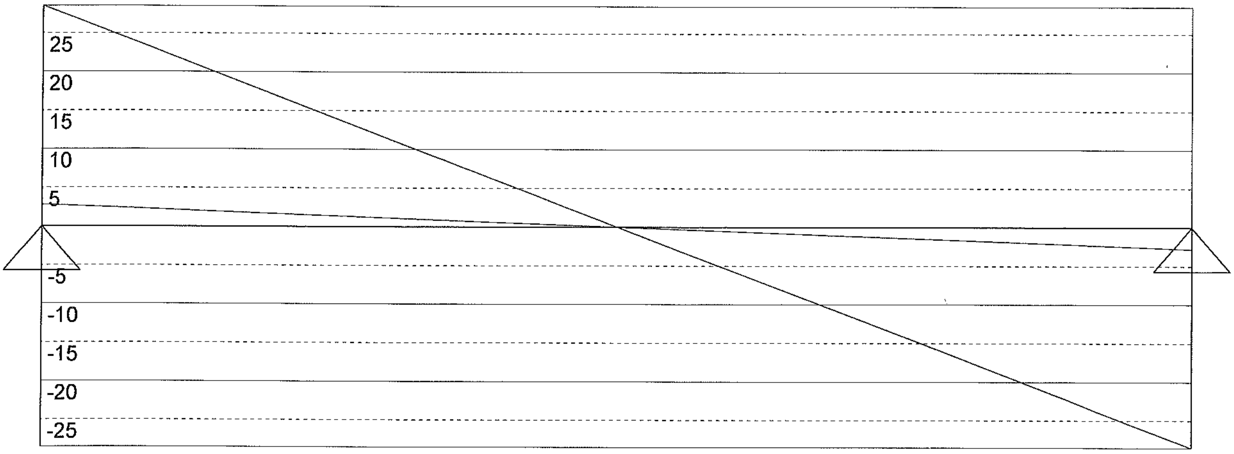

Load factor of dead load= 1 Load factor of imposed load= 1
Load width 1.7 (m) (by which the loads
has been multiplied during calculation)
Max/Min reactions of beam [kN]
28,521 28,527
2,804 2,805

KER 114 x 300 B 2 Cf=1,00 Design method: Allowable stress design
Increasing factor of the allowable stress 1,03
Factored Moment/Moment capacity [kNm] 23,535 31,556 75 %
Factored shear force/shear capacity [kN] 28,521 39,737 72 %

(3) LVL | 1 1/2 x 11 7/8 ✓

Deflection due to unfactored load (Deflection limit L/240)
10,0 mm (73 %)

Beam Id: Lot # 70 - RB 2

Date 18-05-2018

Structural Engineer:

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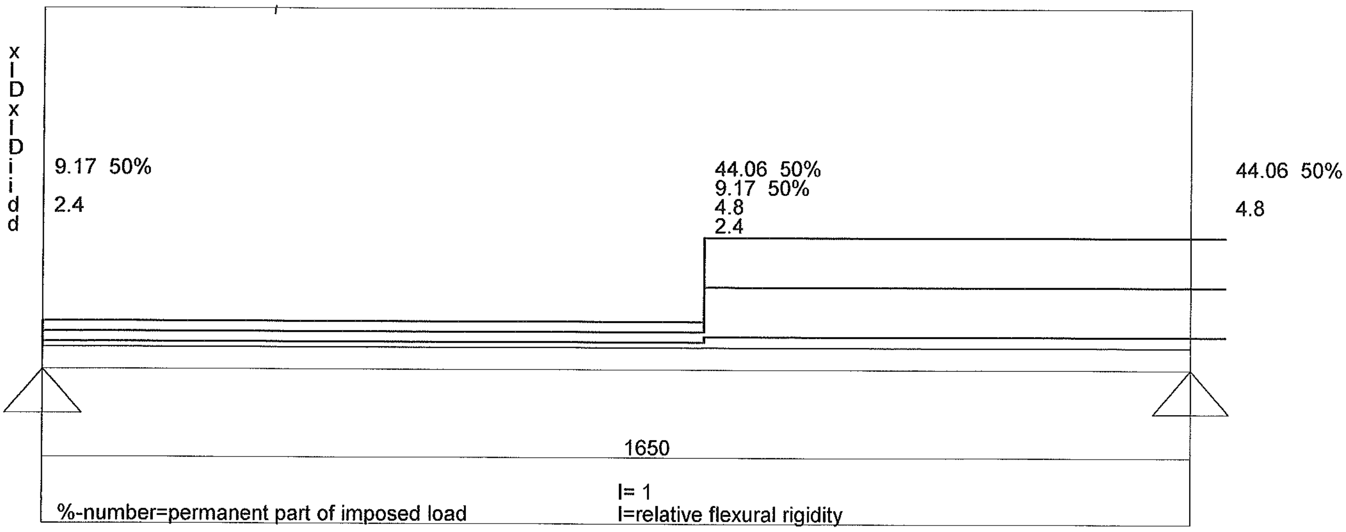

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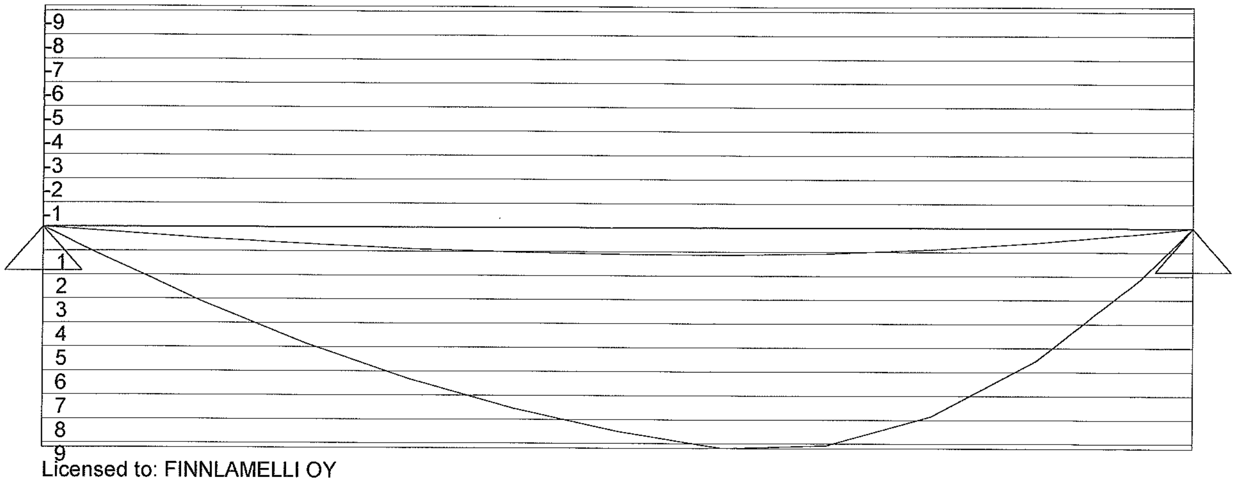

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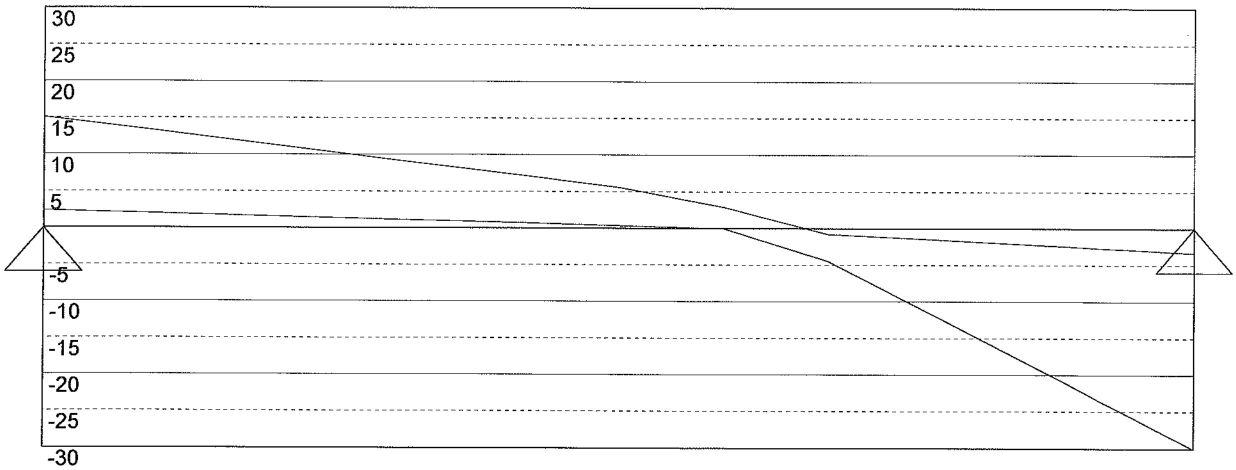

Load factor of dead load= 1 Load factor of imposed load= 1
Load width 1 (m) (by which the loads
has been multiplied during calculation)
Max/Min reactions of beam [kN]
15,080 30,111
2,336 3,304

KER 114 x 300 B 2 Cf=1,00 Design method: Allowable stress design
Increasing factor of the allowable stress 1,03
Factored Moment/Moment capacity [kNm] 9,194 31,648 29 %
Factored shear force/shear capacity [kN] 30,103 39,853 76 %

(3) LVL $1\frac{1}{2} \times 11\frac{7}{8}$ ✓

Deflection due to unfactored load (Deflection limit L/240)
0,9 mm (14 %)

Beam Id: Lot # 70 - RB 3

Date 18-05-2018

Structural Engineer:

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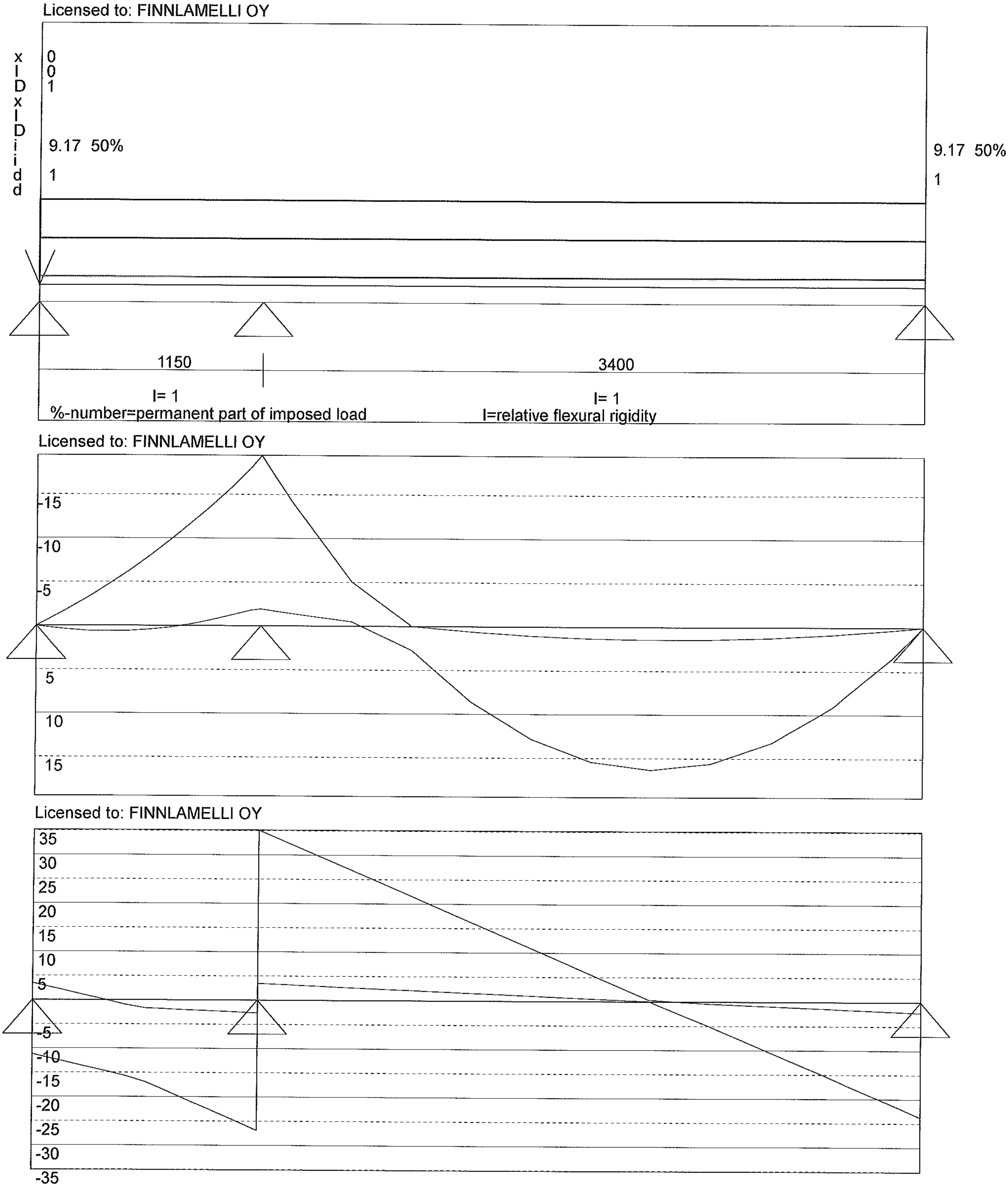


Load factor of dead load= 1 Load factor of imposed load= 1
Load width 1.7 (m) (by which the loads
has been multiplied during calculation)
Max/Min reactions of beam [kN]
4,510 61,976 23,762
-10,184 6,094 2,231

KER 114 x 300 B 2 Cf=1,00 Design method: Allowable stress design
Increasing factor of the allowable stress 1,03
Factored Moment/Moment capacity [kNm] 19,462 31,556 62 %
Factored shear force/shear capacity [kN] 35,111 39,737 88 %

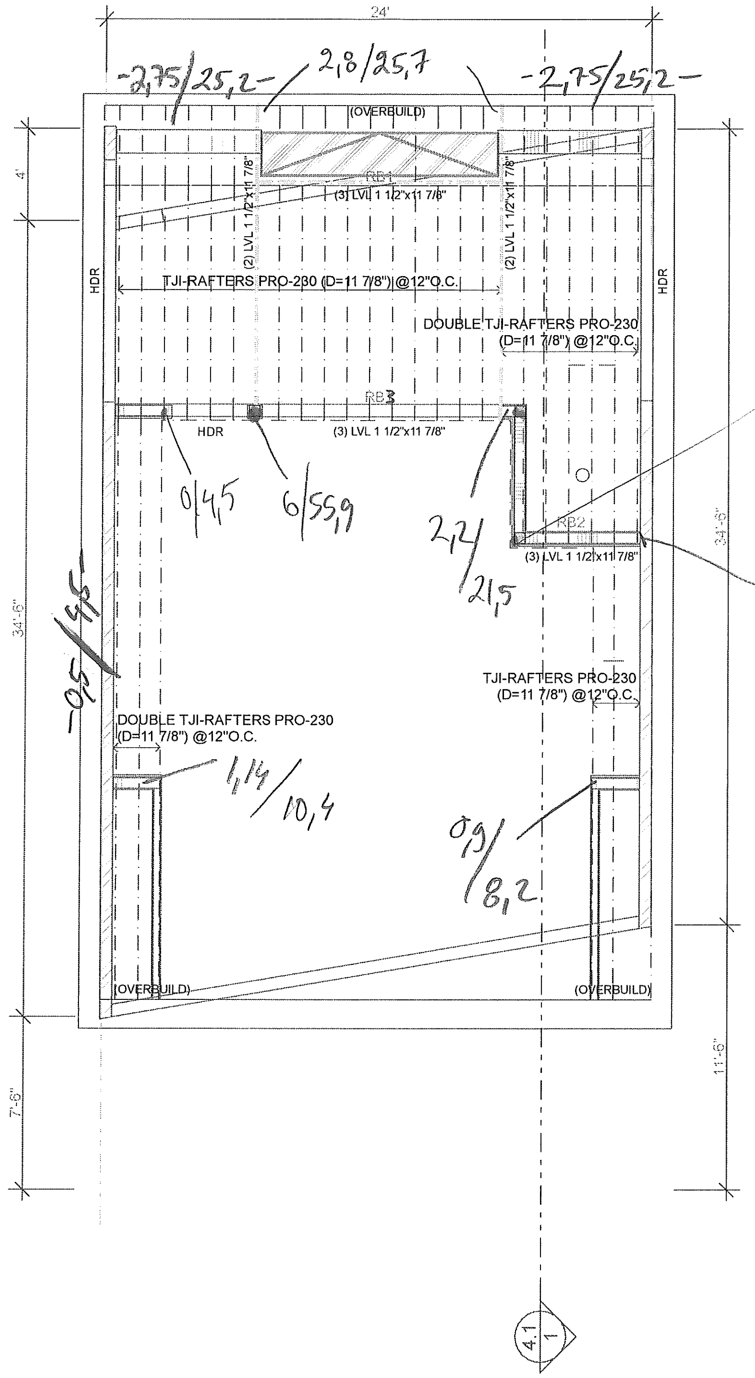
(3) LVL 1 1/2 x 11 7/8 ✓

Deflection due to unfactored load (Deflection limit L/240)
0,0 mm (0 %) 6,1 mm (43 %)

LOT 70

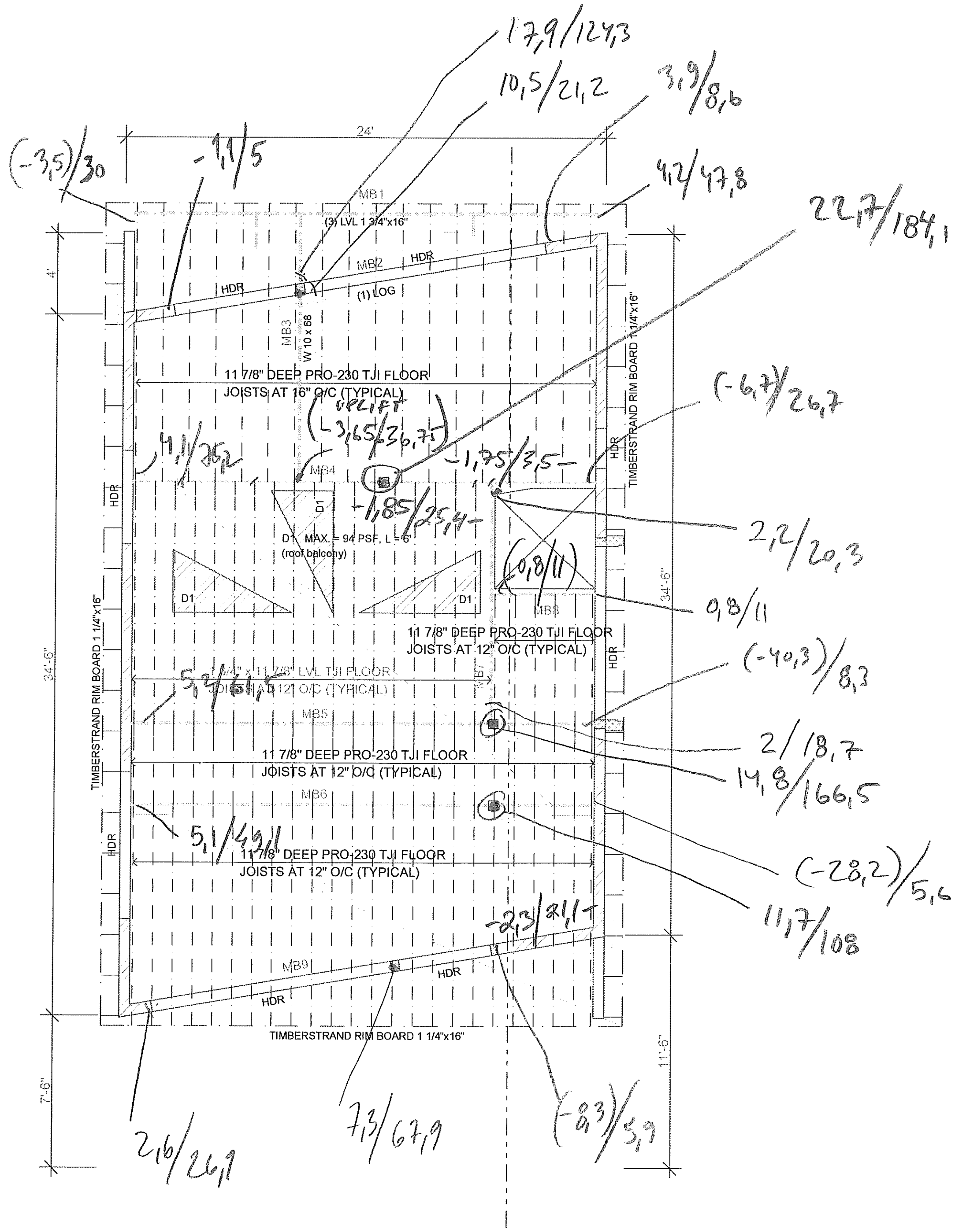
ROOF LOADS

SNOW = 188 psf = 9,17 kN/m



LOT 70

ROOF "LEVEL" LOADS
(FLOOR)

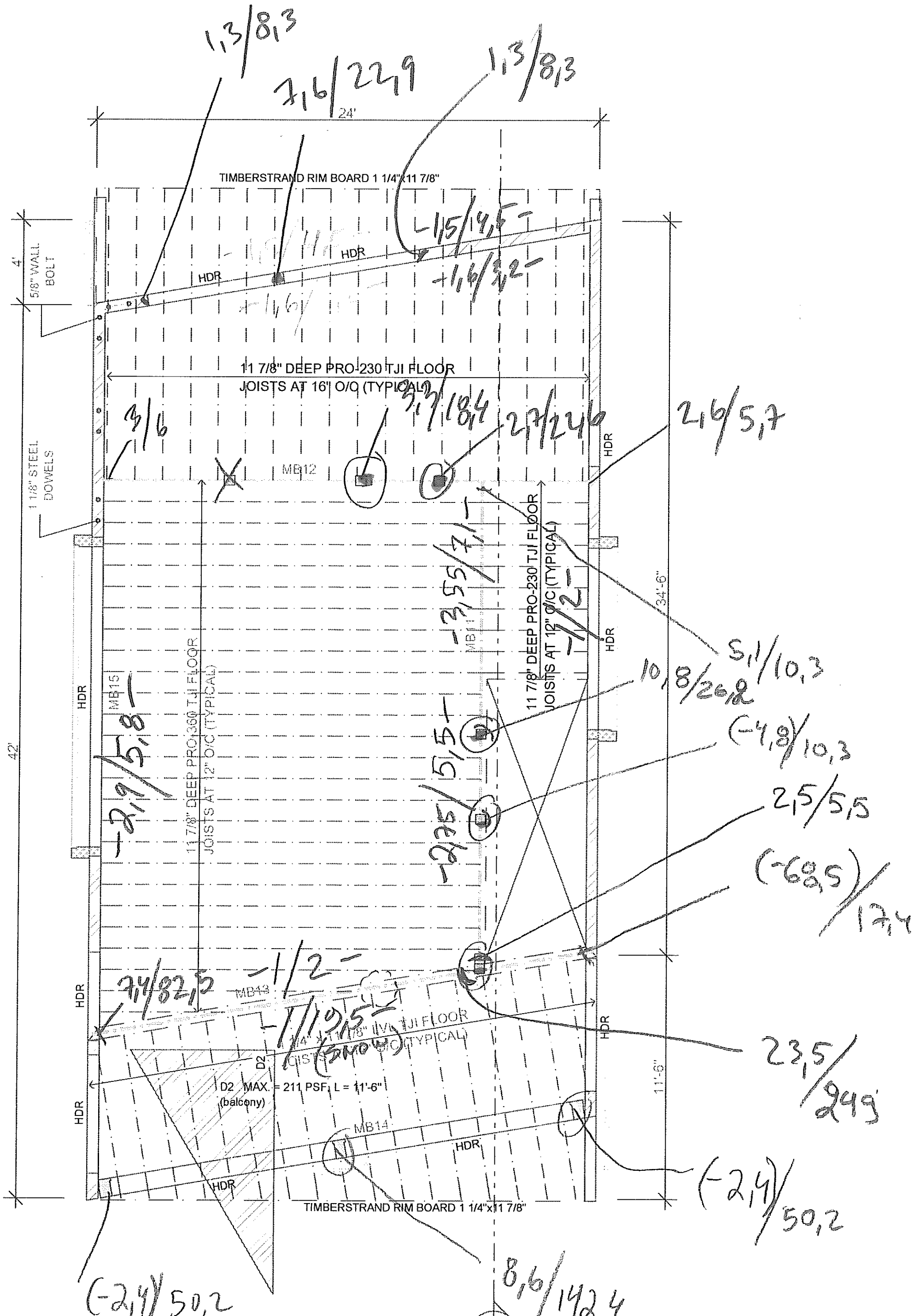


1 S4 UPPER LEVEL FRAMING PLAN

4.1 1

Lot 70

UPPER FLOOR LOADS

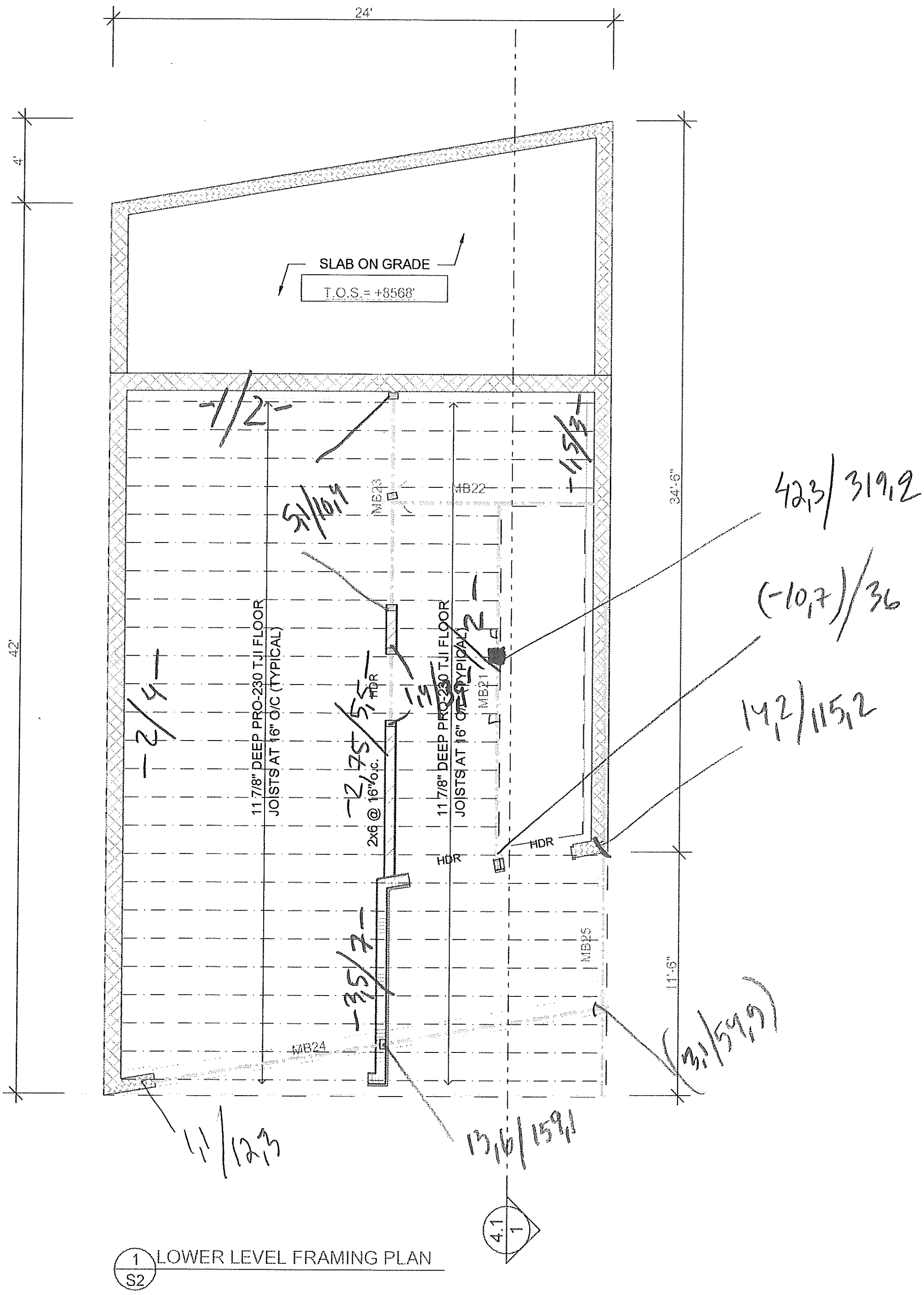


1 S3 MAIN LEVEL FRAMING PLAN

4.1 1

LOT 70

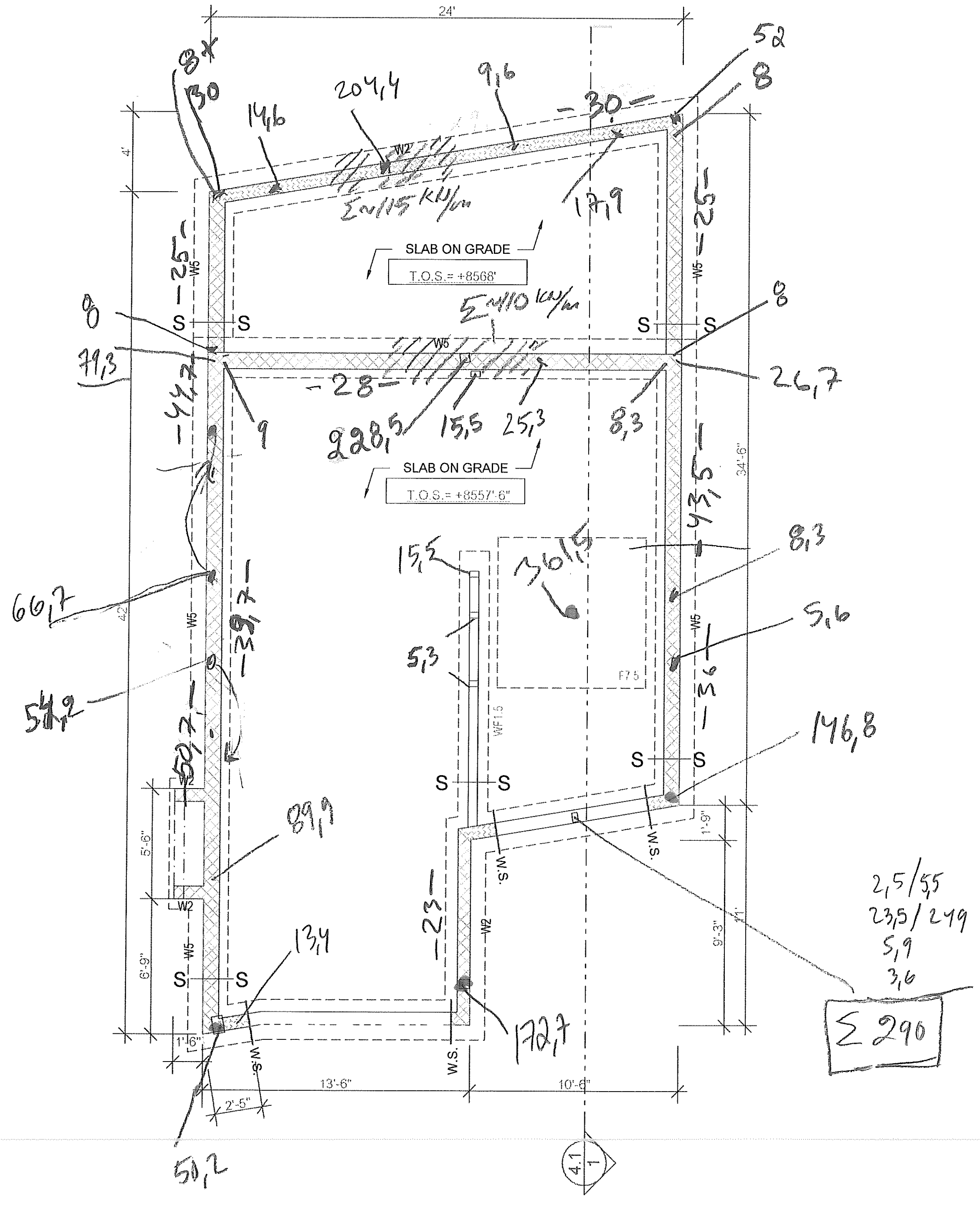
MAIN FLOOR LOADS



LOWER LEVEL FRAMING PLAN
1
S2

Lot 70

FOOTING LOADS



1 S1 FOOTING AND FOUNDATION PLAN

Ensign Engineering
45 West 10000 South, Suite 500
Sandy, Utah 84070
P: (801) 255-0529

Wood Beam

Lic. # : KW-06004069 **Licensee : ENSIGN ENGINEERING**

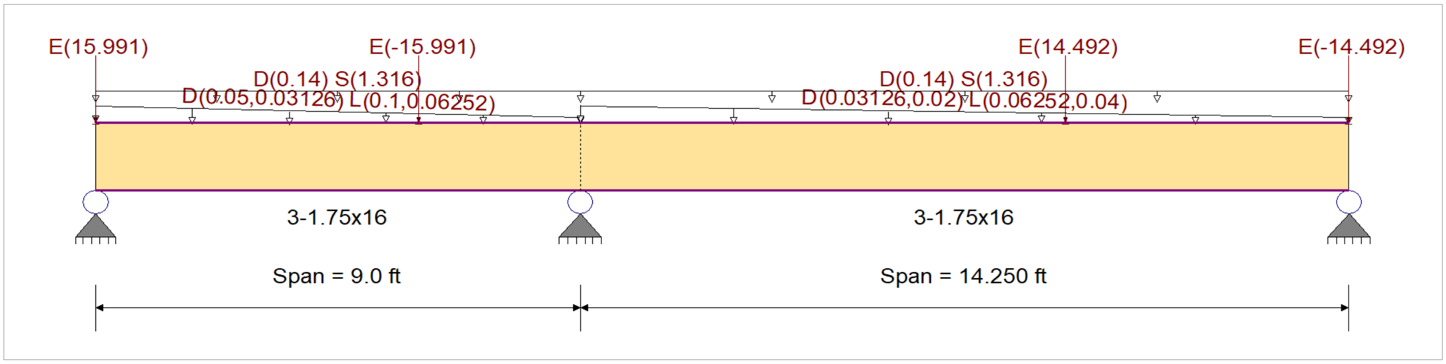
Description : MB1

CODE REFERENCES

Calculations per NDS 2015, IBC 2015, CBC 2016, ASCE 7-10
Load Combination Set : ASCE 7-10

Material Properties

Analysis Method :	Allowable Stress Design	Fb +	2,600.0 psi	*E : Modulus of Elasticity*	
Load Combination :	ASCE 7-10	Fb -	2,600.0 psi	Ebend- xx	2,000.0 ksi
		Fc - Prll	2,510.0 psi	Eminbend - xx	1,016.54 ksi
Wood Species	: Trus Joist	Fc - Perp	750.0 psi		
Wood Grade	: MicroLam LVL 2.0 E	Fv	285.0 psi		
		Ft	1,555.0 psi	Density	42.0 pcf
Beam Bracing	: Beam is Fully Braced against lateral-torsional buckling				



Applied Loads

Service loads entered. Load Factors will be applied for calculations.

Load for Span Number 1

Varying Uniform Load : D= 0.020->0.020, L= 0.040->0.040 ksf, Extent = 0.0 -->> 9.0 ft, Trib Width = 2.50->1.563 ft, (Floor)

Uniform Load : D = 0.020, S = 0.1880 ksf, Tributary Width = 7.0 ft, (Roof)

Point Load : E = 15.991 k @ 0.0 ft, (Hold Down)

Point Load : E = -15.991 k @ 6.0 ft, (Hold Down)

Load for Span Number 2

Varying Uniform Load : D= 0.020->0.020, L= 0.040->0.040 ksf, Extent = 0.0 -->> 14.250 ft, Trib Width = 1.563->1.0 ft, (Floor)

Uniform Load : D = 0.020, S = 0.1880 ksf, Tributary Width = 7.0 ft, (Roof)

Point Load : E = 14.492 k @ 9.0 ft, (Hold Down)

Point Load : E = -14.492 k @ 14.250 ft, (Hold Down)

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THE STANDARD IN ENGINEERING

Project Title: Powder Mountain
Engineer: Alex Hawkins
Project ID: 8332
Project Descr:

Printed: 1 AUG 2018, 2:11PM

File = P:\8332 Powder Mountain 70\Structural Calcs\3 Sided Diaphragms.ec6

Wood Beam

Lic. # : KW-06004069 **Licensee : ENSIGN ENGINEERING**

Description : MB1

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio	=	**0.573** : 1	Maximum Shear Stress Ratio	=	**0.583** : 1
Section used for this span		**3-1.75x16**	Section used for this span		**3-1.75x16**
fb : Actual	=	2,293.97 psi	fv : Actual	=	191.17 psi
FB : Allowable	=	4,000.38 psi	Fv : Allowable	=	327.75 psi
Load Combination		+D+0.750L+0.750S+0.5250E	Load Combination		+D+S
Location of maximum on span	=	8.996 ft	Location of maximum on span	=	9.000 ft
Span # where maximum occurs	=	Span # 2	Span # where maximum occurs	=	Span # 1

Maximum Deflection

Max Downward Transient Deflection	0.328 in	Ratio =	521 >=360
Max Upward Transient Deflection	0.000 in	Ratio =	0 <360
Max Downward Total Deflection	0.341 in	Ratio =	501 >=240
Max Upward Total Deflection	0.000 in	Ratio =	0 <240

Maximum Forces & Stresses for Load Combinations

Load Combination		Max Stress Ratios									Moment Values			Shear Values		
Segment Length	Span #	M	V	C_d	$C_{F/V}$	C_i	C_r	C_m	C_t	C_L	M	fb	F'b	V	fv	F'v
D Only													0.00	0.00	0.00	0.00
Length = 9.0 ft	1	0.078	0.084	0.90	0.962	1.00	1.00	1.00	1.00	1.00	3.29	176.15	2250.22	1.21	21.55	256.50
Length = 14.250 ft	2	0.078	0.084	0.90	0.962	1.00	1.00	1.00	1.00	1.00	3.29	176.15	2250.22	1.21	21.55	256.50
+D+L					0.962	1.00	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 9.0 ft	1	0.095	0.100	1.00	0.962	1.00	1.00	1.00	1.00	1.00	4.41	236.29	2500.24	1.60	28.56	285.00
Length = 14.250 ft	2	0.095	0.100	1.00	0.962	1.00	1.00	1.00	1.00	1.00	4.41	236.29	2500.24	1.60	28.56	285.00
+D+S					0.962	1.00	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 9.0 ft	1	0.539	0.583	1.15	0.962	1.00	1.00	1.00	1.00	1.00	28.92	1,549.25	2875.28	10.71	191.17	327.75
Length = 14.250 ft	2	0.539	0.583	1.15	0.962	1.00	1.00	1.00	1.00	1.00	28.92	1,549.25	2875.28	10.71	191.17	327.75
+D+0.750L					0.962	1.00	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 9.0 ft	1	0.071	0.075	1.25	0.962	1.00	1.00	1.00	1.00	1.00	4.13	221.26	3125.30	1.50	26.80	356.25
Length = 14.250 ft	2	0.071	0.075	1.25	0.962	1.00	1.00	1.00	1.00	1.00	4.13	221.26	3125.30	1.50	26.80	356.25
+D+0.750L+0.750S					0.962	1.00	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 9.0 ft	1	0.435	0.470	1.15	0.962	1.00	1.00	1.00	1.00	1.00	23.35	1,251.08	2875.28	8.63	154.02	327.75
Length = 14.250 ft	2	0.435	0.470	1.15	0.962	1.00	1.00	1.00	1.00	1.00	23.35	1,251.08	2875.28	8.63	154.02	327.75
+D+0.70E					0.962	1.00	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 9.0 ft	1	0.368	0.237	1.60	0.962	1.00	1.00	1.00	1.00	1.00	27.47	1,471.41	4000.38	6.05	108.05	456.00
Length = 14.250 ft	2	0.452	0.260	1.60	0.962	1.00	1.00	1.00	1.00	1.00	33.77	1,809.27	4000.38	6.64	118.62	456.00
+D+0.750L+0.750S+0.5250E					0.962	1.00	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 9.0 ft	1	0.382	0.462	1.60	0.962	1.00	1.00	1.00	1.00	1.00	28.52	1,527.67	4000.38	11.79	210.55	456.00
Length = 14.250 ft	2	0.573	0.462	1.60	0.962	1.00	1.00	1.00	1.00	1.00	42.82	2,293.97	4000.38	11.79	210.55	456.00
+0.60D					0.962	1.00	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 9.0 ft	1	0.026	0.028	1.60	0.962	1.00	1.00	1.00	1.00	1.00	1.97	105.69	4000.38	0.72	12.93	456.00
Length = 14.250 ft	2	0.026	0.028	1.60	0.962	1.00	1.00	1.00	1.00	1.00	1.97	105.69	4000.38	0.72	12.93	456.00
+0.60D+0.70E					0.962	1.00	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 9.0 ft	1	0.365	0.247	1.60	0.962	1.00	1.00	1.00	1.00	1.00	27.24	1,459.31	4000.38	6.31	112.67	456.00
Length = 14.250 ft	2	0.438	0.249	1.60	0.962	1.00	1.00	1.00	1.00	1.00	32.70	1,751.62	4000.38	6.36	113.48	456.00

Overall Maximum Deflections

Load Combination	Span	Max. "-" Defl	Location in Span	Load Combination	Max. "+" Defl	Location in Span
	1	0.0000	0.000	E Only	-0.1262	4.978
+D+0.750L+0.750S+0.5250E	2	0.3407	7.881		0.0000	4.978

Vertical Reactions

Support notation : Far left is #1 Values in KIPS

Load Combination	Support 1	Support 2	Support 3
Overall MAXimum	9.568	22.533	8.514
Overall MINimum	9.568	-3.539	-6.029
D Only	0.462	2.588	0.936
+D+L	0.731	3.521	1.196
+D+S	3.536	22.533	8.514
+D+0.750L	0.663	3.288	1.131
+D+0.750L+0.750S	2.969	18.247	6.814

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Wood Beam

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Description : MB1

Vertical Reactions

Support notation : Far left is #1 Values in KIPS

Load Combination	Support 1	Support 2	Support 3
+D+0.70E	7.159	0.111	-3.284
+D+0.750L+0.750S+0.5250E	7.992	16.389	3.649
+0.60D	0.277	1.553	0.562
+0.60D+0.70E	6.975	-0.924	-3.659
L Only	0.269	0.933	0.260
S Only	3.074	19.945	7.578
E Only	9.568	-3.539	-6.029

Ensign Engineering
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Project Title: Powder Mountain
Engineer: Alex Hawkins
Project ID: 8332
Project Descr:

Steel Beam

Lic. # : KW-06004069 **Licensee : ENSIGN ENGINEERING**

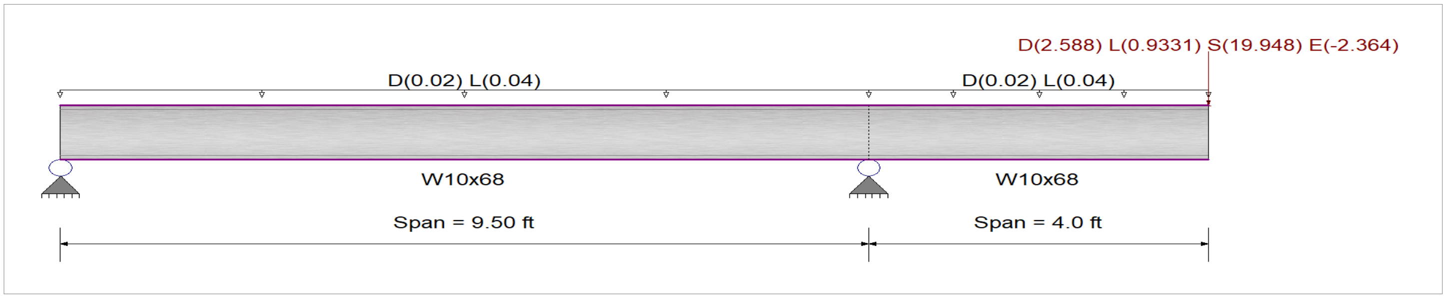
Description : MB3

CODE REFERENCES

Calculations per AISC 360-10, IBC 2015, CBC 2016, ASCE 7-10
Load Combination Set : ASCE 7-10

Material Properties

Analysis Method :	Allowable Strength Design	Fy : Steel Yield :	50.0 ksi
Beam Bracing :	Beam is Fully Braced against lateral-torsional buckling	E: Modulus :	29,000.0 ksi
Bending Axis :	Major Axis Bending		



Applied Loads

Service loads entered. Load Factors will be applied for calculations.

Beam self weight NOT internally calculated and added
Load for Span Number 1
Uniform Load : D = 0.020, L = 0.040 ksf, Tributary Width = 1.0 ft, (Floor)

Load for Span Number 2
Uniform Load : D = 0.020, L = 0.040 ksf, Tributary Width = 1.0 ft, (Floor)

Point Load : D = 2.588, L = 0.9331, S = 19.948, E = -2.364 k @ 4.0 ft, (MB1)

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio =	**0.424** : 1	Maximum Shear Stress Ratio =	**0.231** : 1
Section used for this span	**W10x68**	Section used for this span	**W10x68**
Ma : Applied	90.304 k-ft	Va : Applied	22.616 k
Mn / Omega : Allowable	212.824 k-ft	Vn/Omega : Allowable	97.760 k
Load Combination	+D+S	Load Combination	+D+S
Location of maximum on span	9.500 ft	Location of maximum on span	9.500 ft
Span # where maximum occurs	Span # 1	Span # where maximum occurs	Span # 1

Maximum Deflection

Max Downward Transient Deflection	0.217 in	Ratio =	442 >=360
Max Upward Transient Deflection	-0.070 in	Ratio =	1,623 >=360
Max Downward Total Deflection	0.245 in	Ratio =	392 >=240
Max Upward Total Deflection	-0.079 in	Ratio =	1440 >=240

Maximum Forces & Stresses for Load Combinations

Load Combination / Segment Length	Span #	Max Stress Ratios M	Max Stress Ratios V	Mmax +	Mmax -	Ma Max	Mnx	Mnx/Omega	Cb	Rm	Va Max	Vnx	Vnx/Omega
D Only													
Dsgn. L = 9.50 ft	1	0.049	0.027		-10.51	10.51	355.42	212.82	1.00	1.00	2.67	146.64	97.76
Dsgn. L = 4.00 ft	2	0.049	0.027		-10.51	10.51	355.42	212.82	1.00	1.00	2.67	146.64	97.76
+D+L													
Dsgn. L = 9.50 ft	1	0.068	0.038		-14.56	14.56	355.42	212.82	1.00	1.00	3.76	146.64	97.76
Dsgn. L = 4.00 ft	2	0.068	0.038		-14.56	14.56	355.42	212.82	1.00	1.00	3.76	146.64	97.76
+D+S													
Dsgn. L = 9.50 ft	1	0.424	0.231		-90.30	90.30	355.42	212.82	1.00	1.00	22.62	146.64	97.76
Dsgn. L = 4.00 ft	2	0.424	0.231		-90.30	90.30	355.42	212.82	1.00	1.00	22.62	146.64	97.76
+D+0.750L													
Dsgn. L = 9.50 ft	1	0.064	0.036		-13.55	13.55	355.42	212.82	1.00	1.00	3.49	146.64	97.76
Dsgn. L = 4.00 ft	2	0.064	0.036		-13.55	13.55	355.42	212.82	1.00	1.00	3.49	146.64	97.76
+D+0.750L+0.750S													
Dsgn. L = 9.50 ft	1	0.345	0.189		-73.40	73.40	355.42	212.82	1.00	1.00	18.45	146.64	97.76
Dsgn. L = 4.00 ft	2	0.345	0.189		-73.40	73.40	355.42	212.82	1.00	1.00	18.45	146.64	97.76
+D+0.70E													

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Project Title: Powder Mountain
Engineer: Alex Hawkins
Project ID: 8332
Project Descr:

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Steel Beam

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Description : MB3

Load Combination		Max Stress Ratios		Summary of Moment Values							Summary of Shear Values		
Segment Length	Span #	M	V	Mmax +	Mmax -	Ma Max	Mnx	Mnx/Omega	Cb	Rm	Va Max	Vnx	Vnx/Omega
Dsgn. L = 9.50 ft	1	0.018	0.010		-3.89	3.89	355.42	212.82	1.00	1.00	1.01	146.64	97.76
Dsgn. L = 4.00 ft	2	0.018	0.010		-3.89	3.89	355.42	212.82	1.00	1.00	1.01	146.64	97.76
+D+0.750L+0.750S+0.5250E													
Dsgn. L = 9.50 ft	1	0.322	0.176		-68.43	68.43	355.42	212.82	1.00	1.00	17.21	146.64	97.76
Dsgn. L = 4.00 ft	2	0.322	0.176		-68.43	68.43	355.42	212.82	1.00	1.00	17.21	146.64	97.76
+0.60D													
Dsgn. L = 9.50 ft	1	0.030	0.016		-6.31	6.31	355.42	212.82	1.00	1.00	1.60	146.64	97.76
Dsgn. L = 4.00 ft	2	0.030	0.016		-6.31	6.31	355.42	212.82	1.00	1.00	1.60	146.64	97.76
+0.60D+0.70E													
Dsgn. L = 9.50 ft	1	0.002	0.001	0.34		0.34	355.42	212.82	1.00	1.00	0.09	146.64	97.76
Dsgn. L = 4.00 ft	2	0.001	0.001	0.31		0.31	355.42	212.82	1.00	1.00	0.10	146.64	97.76

Overall Maximum Deflections

Load Combination	Span	Max. "-" Defl	Location in Span	Load Combination	Max. "+" Defl	Location in Span
	1	0.0000	0.000	+D+S	-0.0792	5.510
+D+S	2	0.2451	4.000		0.0000	5.510

Vertical Reactions

Support notation : Far left is #1 Values in KIPS

Load Combination	Support 1	Support 2	Support 3
Overall MAXimum	-9.411	32.217	
Overall MINimum	0.090	-0.030	
D Only	-1.012	3.870	
+D+L	-1.248	5.579	
+D+S	-9.411	32.217	
+D+0.750L	-1.189	5.152	
+D+0.750L+0.750S	-7.488	26.412	
+D+0.70E	-0.315	1.518	
+D+0.750L+0.750S+0.5250E	-6.966	24.648	
+0.60D	-0.607	2.322	
+0.60D+0.70E	0.090	-0.030	
L Only	-0.237	1.710	
S Only	-8.399	28.347	
E Only	0.995	-3.359	

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Project Title: Powder Mountain
Engineer: Alex Hawkins
Project ID: 8332
Project Descr:

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Steel Beam

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Description : MB4

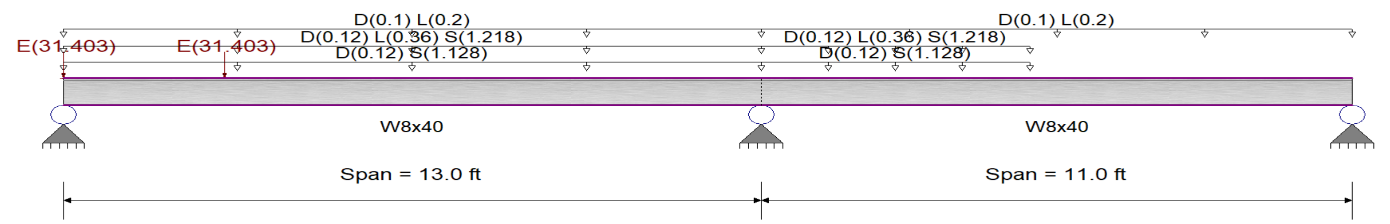
CODE REFERENCES

Calculations per AISC 360-10, IBC 2015, CBC 2016, ASCE 7-10
Load Combination Set : ASCE 7-10

Material Properties

Analysis Method : Allowable Strength Design
Beam Bracing : Beam is Fully Braced against lateral-torsional buckling
Bending Axis : Major Axis Bending

Fy : Steel Yield : 50.0 ksi
E: Modulus : 29,000.0 ksi



Applied Loads

Service loads entered. Load Factors will be applied for calculations.

Beam self weight NOT internally calculated and added
Load for Span Number 1
Uniform Load : D = 0.020, S = 0.1880 ksf, Tributary Width = 6.0 ft, (Roof)

Uniform Load : D = 0.020, L = 0.060, S = 0.2030 ksf, Tributary Width = 6.0 ft, (Balcony)

Uniform Load : D = 0.020, L = 0.040 ksf, Tributary Width = 5.0 ft, (Floor)

Point Load : E = 31.403 k @ 3.0 ft, (Hold Down)

Point Load : E = 31.403 k @ 0.0 ft, (Hold Down)

Load for Span Number 2
Uniform Load : D = 0.020, S = 0.1880 ksf, Extent = 0.0 -->> 5.0 ft, Tributary Width = 6.0 ft, (Roof)

Uniform Load : D = 0.020, L = 0.060, S = 0.2030 ksf, Extent = 0.0 -->> 5.0 ft, Tributary Width = 6.0 ft, (Balcony)

Uniform Load : D = 0.020, L = 0.040 ksf, Tributary Width = 5.0 ft, (Floor)

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio =	**0.648** : 1	Maximum Shear Stress Ratio =	**0.423** : 1
Section used for this span	**W8x40**	Section used for this span	**W8x40**
Ma : Applied	64.303 k-ft	Va : Applied	25.135 k
Mn / Omega : Allowable	99.301 k-ft	Vn/Omega : Allowable	59.40 k
Load Combination	+D+0.750L+0.750S+0.5250E	Load Combination	+D+0.750L+0.750S+0.5250E
Location of maximum on span	3.432 ft	Location of maximum on span	0.000 ft
Span # where maximum occurs	Span # 1	Span # where maximum occurs	Span # 1

Maximum Deflection

Max Downward Transient Deflection	0.290 in	Ratio =	537 >=480
Max Upward Transient Deflection	-0.077 in	Ratio =	1,707 >=480
Max Downward Total Deflection	0.372 in	Ratio =	419 >=360
Max Upward Total Deflection	-0.069 in	Ratio =	1908 >=360

Maximum Forces & Stresses for Load Combinations

Load Combination		Max Stress Ratios		Summary of Moment Values							Summary of Shear Values		
Segment Length	Span #	M	V	Mmax +	Mmax -	Ma Max	Mnx	Mnx/Omega	Cb	Rm	Va Max	Vnx	Vnx/Omega
D Only													
Dsgn. L = 13.00 ft	1	0.054	0.044	4.73	-5.40	5.40	165.83	99.30	1.00	1.00	2.63	89.10	59.40

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Steel Beam

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Description : MB4

Load Combination		Max Stress Ratios		Summary of Moment Values							Summary of Shear Values		
Segment Length	Span #	M	V	Mmax +	Mmax -	Ma Max	Mnx	Mnx/Omega	Cb	Rm	Va Max	Vnx	Vnx/Omega
Dsgn. L = 11.00 ft	2	0.054	0.033	0.55	-5.40	5.40	165.83	99.30	1.00	1.00	1.97	89.10	59.40
+D+L													
Dsgn. L = 13.00 ft	1	0.145	0.117	12.48	-14.43	14.43	165.83	99.30	1.00	1.00	6.96	89.10	59.40
Dsgn. L = 11.00 ft	2	0.145	0.089	1.73	-14.43	14.43	165.83	99.30	1.00	1.00	5.28	89.10	59.40
+D+S													
Dsgn. L = 13.00 ft	1	0.406	0.346	38.39	-40.27	40.27	165.83	99.30	1.00	1.00	20.56	89.10	59.40
Dsgn. L = 11.00 ft	2	0.406	0.239		-40.27	40.27	165.83	99.30	1.00	1.00	14.20	89.10	59.40
+D+0.750L													
Dsgn. L = 13.00 ft	1	0.123	0.099	10.54	-12.17	12.17	165.83	99.30	1.00	1.00	5.88	89.10	59.40
Dsgn. L = 11.00 ft	2	0.123	0.075	1.44	-12.17	12.17	165.83	99.30	1.00	1.00	4.45	89.10	59.40
+D+0.750L+0.750S													
Dsgn. L = 13.00 ft	1	0.386	0.325	35.79	-38.33	38.33	165.83	99.30	1.00	1.00	19.32	89.10	59.40
Dsgn. L = 11.00 ft	2	0.386	0.229	0.44	-38.33	38.33	165.83	99.30	1.00	1.00	13.63	89.10	59.40
+D+0.70E													
Dsgn. L = 13.00 ft	1	0.509	0.293	50.59	-22.31	50.59	165.83	99.30	1.00	1.00	17.40	89.10	59.40
Dsgn. L = 11.00 ft	2	0.225	0.059		-22.31	22.31	165.83	99.30	1.00	1.00	3.51	89.10	59.40
+D+0.750L+0.750S+0.5250E													
Dsgn. L = 13.00 ft	1	0.648	0.423	64.30	-51.01	64.30	165.83	99.30	1.00	1.00	25.13	89.10	59.40
Dsgn. L = 11.00 ft	2	0.514	0.249		-51.01	51.01	165.83	99.30	1.00	1.00	14.78	89.10	59.40
+0.60D													
Dsgn. L = 13.00 ft	1	0.033	0.027	2.84	-3.24	3.24	165.83	99.30	1.00	1.00	1.58	89.10	59.40
Dsgn. L = 11.00 ft	2	0.033	0.020	0.33	-3.24	3.24	165.83	99.30	1.00	1.00	1.18	89.10	59.40
+0.60D+0.70E													
Dsgn. L = 13.00 ft	1	0.494	0.281	49.04	-20.15	49.04	165.83	99.30	1.00	1.00	16.69	89.10	59.40
Dsgn. L = 11.00 ft	2	0.203	0.046		-20.15	20.15	165.83	99.30	1.00	1.00	2.72	89.10	59.40

Overall Maximum Deflections

Load Combination	Span	Max. "-" Defl	Location in Span	Load Combination	Max. "+" Defl	Location in Span
+D+0.750L+0.750S+0.5250E	1	0.3723	5.668		0.0000	0.000
	2	0.0000	5.668	E Only	-0.0773	4.664

Vertical Reactions

Support notation : Far left is #1 Values in KIPS

Load Combination	Support 1	Support 2	Support 3
Overall MAXimum	53.701	38.886	-2.196
Overall MINimum	1.077	2.757	-0.173
D Only	1.794	4.594	0.331
+D+L	4.740	12.240	1.020
+D+S	14.361	34.760	-0.173
+D+0.750L	4.004	10.329	0.848
+D+0.750L+0.750S	13.429	32.953	0.470
+D+0.70E	39.385	12.505	-1.206
+D+0.750L+0.750S+0.5250E	41.622	38.886	-0.683
+0.60D	1.077	2.757	0.199
+0.60D+0.70E	38.667	10.667	-1.338
L Only	2.946	7.646	0.689
S Only	12.567	30.165	-0.504
E Only	53.701	11.301	-2.196

Steel Column

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Description : Steel Column

Code References

Calculations per AISC 360-10, IBC 2015, CBC 2016, ASCE 7-10
Load Combinations Used : ASCE 7-10

General Information

Steel Section Name :	**HSS4x4x3/8**	Overall Column Height	9.50 ft
Analysis Method :	Allowable Strength	Top & Bottom Fixity	Top & Bottom Pinned
Steel Stress Grade			
Fy : Steel Yield	46.0 ksi		
E : Elastic Bending Modulus	29,000.0 ksi		

Brace condition for deflection (buckling) along columns :
X-X (width) axis :
Unbraced Length for X-X Axis buckling = 9.50 ft, K = 1.0
Y-Y (depth) axis :
Unbraced Length for Y-Y Axis buckling = 9.50 ft, K = 1.0

Applied Loads

Service loads entered. Load Factors will be applied for calculations.

Column self weight included : 163.376 lbs * Dead Load Factor
AXIAL LOADS . . .
Axial Load at 9.50 ft, D = 9.509, L = 71.759 k

DESIGN SUMMARY

Bending & Shear Check Results

PASS Max. Axial+Bending Stress Ratio =	**0.9269**	: 1
Load Combination	+D+L	
Location of max.above base	0.0	ft
At maximum location values are . . .		
Pa : Axial	81.431	k
Pn / Omega : Allowable	87.856	k
Ma-x : Applied	0.0	k-ft
Mn-x / Omega : Allowable	14.668	k-ft
Ma-y : Applied	0.0	k-ft
Mn-y / Omega : Allowable	14.668	k-ft
PASS Maximum Shear Stress Ratio =	**0.0**	: 1
Load Combination		
Location of max.above base	0.0	ft
At maximum location values are . . .		
Va : Applied	0.0	k
Vn / Omega : Allowable	0.0	k

Maximum Load Reactions . .

Top along X-X	0.0 k
Bottom along X-X	0.0 k
Top along Y-Y	0.0 k
Bottom along Y-Y	0.0 k

Maximum Load Deflections . . .

Along Y-Y 0.0 in at 0.0ft above base
for load combination :

Along X-X 0.0 in at 0.0ft above base
for load combination :

Load Combination Results

Load Combination	Maximum Axial + Bending Stress Ratios: Stress Ratio	Status	Location	Maximum Shear Ratios: Stress Ratio	Status	Location
D Only	0.110	PASS	0.00 ft	0.000	PASS	0.00 ft
+D+L	0.927	PASS	0.00 ft	0.000	PASS	0.00 ft
+D+0.750L	0.723	PASS	0.00 ft	0.000	PASS	0.00 ft
+0.60D	0.066	PASS	0.00 ft	0.000	PASS	0.00 ft

Maximum Reactions

Note: Only non-zero reactions are listed.

Load Combination	Axial Reaction @ Base	X-X Axis Reaction k @ Base	X-X Axis Reaction @ Top	Y-Y Axis Reaction @ Base	Y-Y Axis Reaction @ Top	Mx - End Moments k-ft @ Base	Mx - End Moments @ Top	My - End Moments @ Base	My - End Moments @ Top
D Only	9.672								
+D+L	81.431								
+D+0.750L	63.492								
+0.60D	5.803								
L Only	71.759								

Extreme Reactions

Item	Extreme Value	Axial Reaction @ Base	X-X Axis Reaction k @ Base	X-X Axis Reaction @ Top	Y-Y Axis Reaction @ Base	Y-Y Axis Reaction @ Top	Mx - End Moments k-ft @ Base	Mx - End Moments @ Top	My - End Moments @ Base	My - End Moments @ Top
Axial @ Base	Maximum	81.431								
"	Minimum	5.803								

Steel Column

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Description : Steel Column

Extreme Reactions

Item	Extreme Value	Axial Reaction @ Base	X-X Axis Reaction k @ Base	X-X Axis Reaction @ Top	Y-Y Axis Reaction @ Base	Y-Y Axis Reaction @ Top	Mx - End Moments k-ft @ Base	Mx - End Moments @ Top	My - End Moments @ Base	My - End Moments @ Top
Reaction, X-X Axis Base	Maximum	9.672								
"	Minimum	9.672								
Reaction, Y-Y Axis Base	Maximum	9.672								
"	Minimum	9.672								
Reaction, X-X Axis Top	Maximum	9.672								
"	Minimum	9.672								
Reaction, Y-Y Axis Top	Maximum	9.672								
"	Minimum	9.672								
Moment, X-X Axis Base	Maximum	9.672								
"	Minimum	9.672								
Moment, Y-Y Axis Base	Maximum	9.672								
"	Minimum	9.672								
Moment, X-X Axis Top	Maximum	9.672								
"	Minimum	9.672								
Moment, Y-Y Axis Top	Maximum	9.672								
"	Minimum	9.672								

Maximum Deflections for Load Combinations

Load Combination	Max. X-X Deflection		Distance		Max. Y-Y Deflection		Distance	
D Only	0.0000	in	0.000	ft	0.000	in	0.000	ft
+D+L	0.0000	in	0.000	ft	0.000	in	0.000	ft
+D+0.750L	0.0000	in	0.000	ft	0.000	in	0.000	ft
+0.60D	0.0000	in	0.000	ft	0.000	in	0.000	ft
L Only	0.0000	in	0.000	ft	0.000	in	0.000	ft

Steel Section Properties : HSS4x4x3/8

Depth	=	4.000	in	I xx	=	10.30	in^4	J	=	17.500	in^4
Design Thick	=	0.349	in	S xx	=	5.13	in^3				
Width	=	4.000	in	R xx	=	1.470	in				
Wall Thick	=	0.375	in	Zx	=	6.390	in^3				
Area	=	4.780	in^2	I yy	=	10.300	in^4	C	=	9.140	in^3
Weight	=	17.197	plf	S yy	=	5.130	in^3				
				R yy	=	1.470	in				
Ycg	=	0.000	in								

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Project Title: Powder Mountain
Engineer: Alex Hawkins
Project ID: 8332
Project Descr:

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File = P:\8332 Powder Mountain 70\Structural Calcs\3 Sided Diaphragms.ec6

Steel Column

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Description : Steel Column

Sketches