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TOKYO, JAPAN

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CALCULATIONS FOR

大茶貯蓄成形格納庫 應力計算書

昭和十四年八月

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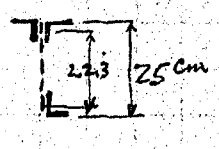
CALCULATIONS FOR
中央卸賣養成所格納庫

応力計算書

Purlin Span length = 4.17m spacing 0.83m c to c

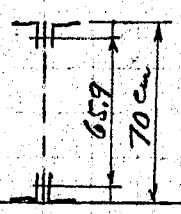
Load on purlin
Cement slate 18
purlin say 25
30cm snow say 30
misc. say 2
75 kg/m² × 0.83 = 62 kg/m.



Moment = 1/8 × 62 × 4.17² = 135 kgm
effective depth say 25 − 2@1.36 = 22.3 cm
flg. stress = 135×100 / 22.3 = 606 kg
flg. area req'd = 606/1400 = 0.43 cm² net
use
2L 50×50×4 = 7.78 cm² gr for Top flange.
1L " = 3.89 − 0.56 = 3.33 cm² net for bottom flange.

Rafter Span length = 10.0 metres, spacing 4.17 m

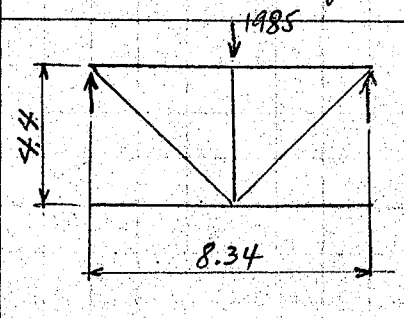
Load on Rafter. Roof and Snow 75
rafter say 10
85 × 4.17 = 355 kg/m



Moment = 1/8 × 355 × 10.0² = 4440 kgm
Effective depth say 70.0 − 2@2.05 = 65.9 cm
Chord stress = 4440×100 / 65.9 = 6740 kg C or T
S.R. = 6740 ÷ 1400 = 4.81 cm² net
use 2L 75×75×6 = 17.45 cm² − 2.64 = 14.81 cm² net
Compression chord.
l/r = 130/2.25 = 58 f = 1400(1 − 0.03×58) = 1160 kg/cm² c.
Compressive stress = 6740/17.45 = 386 kg/cm² c < 1160.

Diagonal Shear say = 355×10÷2 = 1775 kg Coef. = 1.36
max. diag. stress = 1775×1.36 = 2420 kg SR = 1.73 cm² net
use 1L 65×65×6 = 7.53 cm² − 1.14 = 6.39 cm² net
l/r = 83/1.1 = 76 f = 1400(1 − 0.03×76) = 1095 kg/cm² c.
Compressive stress = 2420/7.53 = 321 kg/cm² c.
両端 2 panels 共 1L 65×65×6, 他 1L 50×50×4 トス.

Sway Bracing



Span length = 8.33 m spacing 10.0 m c to c.
Concentration due to rafter = 1775 kg
panel concentration of sway 50×4.17 = 210
1985
Diag. length = √(4.4² + 4.17²) = 6.07 m
Coef. = 6.07/4.4 = 1.38
shear = 1985/2 = 993 kg.
Top chord stress = 993×4.17÷4.4 = 940 kg c.
Diagonal stress = 993×1.38 = 1370 kg T SR = 0.98 cm² net

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CALCULATIONS FOR
中央米負養成所格納庫

Top chord, 2L 75×75×6 = 17.45 cm² $l/r = \frac{417}{2.25} = 185$

$f = 1400(1-0.03\times185) = 623$ kg/cm² C

Compresive stress = $\frac{940}{17.45} = 54$ kg/cm²

Diagonal, use 1L 75×75×6 = 8.73 cm² gr - 1.32 = 7.41 net

Eaves strut

Span length = 8.34 m.

Load on strut

wall	2.0 @	25	= 50
strut say			50
			100 kg/m.

Rafter concentration at center = 1775 kg.

moment = $1/8 \times 100 \times 8.34^2$ = 870

$1/4 \times 1775 \times 8.34$ = 3700

4570 kgm

Chord stresses = $\frac{4570}{1.80}$ = 2540 kg C or T SR = 1.82 cm² net

Shear 100×4.17 = 417

$1775 \div 2$ = 888

1305 kg

Diagonal stress = 1305×1.16 = 1515 kg SR = 1.1 cm² net

Diag. length = $\sqrt{1.8^2+1.04^2} = 2.08$

coef. = $\frac{2.08}{1.80} = 1.16$

Chord section 2L 75×75×6 = 17.45 - 5.28 = 12.17 net

$l/r = \frac{417}{2.25} = 186$ $f = 1400\times(1-0.03\times186) = 622$ kg/cm² C

Stress = $\frac{2540}{1745} = 15$ kg/cm²

Diagonal section 1L 65×65×6 = 7.53 - 1.32 = 6.21 net

$l/r = \frac{208}{1.1} = 189$. $f = 1400(1-0.03\times189) = 607$ kg/cm² C

Stress = 1515 ÷ 7.53 = 201 kg/cm² C.

Roof Truss. Span length 40.0 meters spacing 8.34 m c to c.

Load on truss.

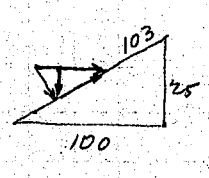
Cement slate	18
purlin	25
rafter	10
sway	5
main truss say	25
lateral bracing	10
30 cm snow	30
misc, say	2
	125 kg/m²

Panel concentration say 2.5 @ 125 × 8.34 = 2600 kg = P

Crane load 3,000 kg moving along center half of the span, = C

Wind load. velocity of wind V = 30 m/sec.

wind pressure $W = \frac{V^2}{10} = \frac{30^2}{10} = 90$ kg/m² on vertical plane.



Slope of roof 25/100

Vertical component = $W \times \frac{25}{103} \times \frac{100}{103} = 0.236W = 90\times0.236 = 21$ kg/m² on windward half

Panel load = 2.5 @ 21 × 8.34 = 440 kg. = w

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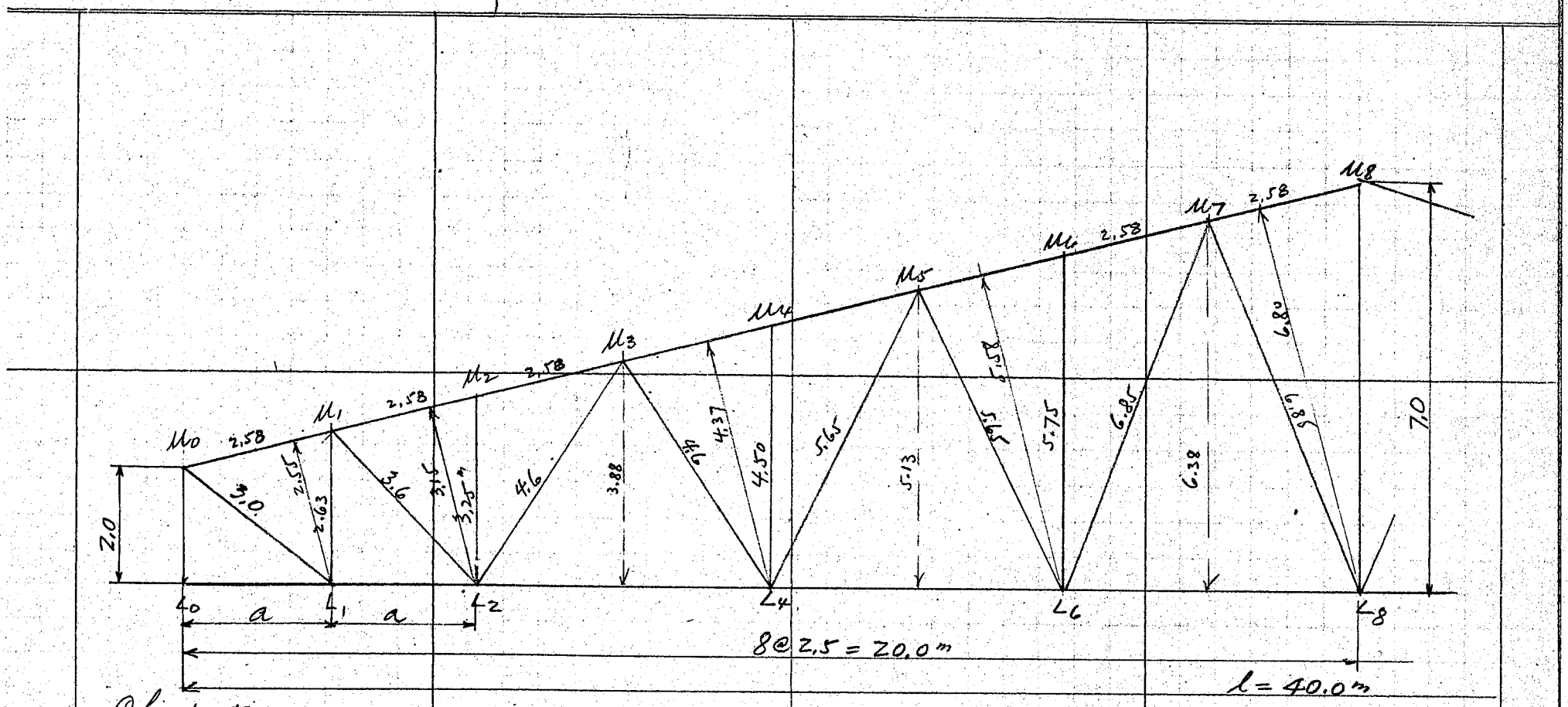
CALCULATIONS FOR

中央東急建設所格納庫



Chord stresses

Stresses of Chord members at normal state. (Dead load with snow load. = P)

Reaction R = 7.5P P = 2600 kg, a = 2.5m

members	R.d	- ΣPd'			a=m	Stress		Vertical Component 25/100
U_0-U_1	7.5Pa		= 7.5Pa =	48,800	÷ 2.55m =	19,200	C	4,800
L_1-L_2	7.5Pa		= 7.5Pa	48,800	2.63	18,600	T	
U_1-U_3	15Pa	- Pa	= 14Pa	91,000	3.15	28,900	C	7,200
L_2-L_4	22.5Pa	- 3Pa	= 19.5Pa	126,700	3.88	32,600	T	
U_3-U_5	30Pa	- 6Pa	= 24Pa	156,000	4.37	35,700	C	8,900
L_4-L_6	37.5Pa	- 10Pa	= 27.5Pa	178,700	5.13	34,800	T	
U_5-U_7	45Pa	- 15Pa	= 30Pa	195,000	5.58	35,000	C	8,800
L_6-L_8	52.5Pa	- 21Pa	= 31.5Pa	204,800	6.38	32,100	T	
U_7-U_8	60Pa	- 28Pa	= 32Pa	208,000	6.80	30,600	C	7,700

Chord stress due to vertical component of wind W = 440 kg per panel.

Reaction $R = W\left(\frac{15+14+13+12+11+10+9+4}{16}\right) = 5.5W$

	Rd.	- ΣWd'			a=m	Stress		vert. Comp.
U_0-U_1	5.5Wa		= 5.5Wa =	6,100	÷ 2.55 =	2,400	C	600
L_1-L_2	5.5Wa		= 5.5Wa	6,100	2.63	2,300	T	
U_1-U_3	11Wa	- Wa	= 10Wa	11,000	3.15	3,500	C	900
L_2-L_4	16.5Wa	- 3Wa	= 13.5Wa	14,900	3.88	3,800	T	
U_3-U_5	22Wa	- 6Wa	= 16Wa	17,600	4.37	4,000	C	1,000
L_4-L_6	27.5Wa	- 10Wa	= 17.5Wa	19,300	5.13	3,800	T	
U_5-U_7	33Wa	- 15Wa	= 18Wa	19,800	5.58	3,500	C	900
L_6-L_8	38.5Wa	- 21Wa	= 17.5Wa	19,300	6.38	3,000	T	
U_7-U_8	44Wa	- 28Wa	= 16Wa	17,600	6.80	2,600	C	700

Chord stress due to Crane load C = 3,000 kg, moving between L_4 to $L_{4'}$

	Load on pt.	R	Rd			Stress		vert. comp.
U_0-U_1	4	0.75C × a =	0.75Ca =	5600	÷ 2.55 =	2200	C	600
L_1-L_2	4	0.75C × a =	0.75Ca	5600	2.63	2100	T	
U_1-U_3	4	0.75C × 2a =	1.5Ca	11,300	3.15	3600	C	900
L_2-L_4	4	0.75C × 3a =	2.25Ca	16,900	3.88	4400	T	
U_3-U_5	4	0.75C × 4a =	3.0Ca	22,500	4.37	5200	C	1300
L_4-L_6	5	0.69C × 5a =	3.45Ca	25900	5.13	5100	T	
U_5-U_7	6	0.63C × 6a =	3.78Ca	28,400	5.58	5100	C	1300
L_6-L_8	7	0.56C × 7a =	3.92Ca	29400	6.38	4600	T	
U_7-U_8	8	0.50C × 8a =	4.00Ca	30,000	6.80	4400	C	1100

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CALCULATIONS FOR
中央来賓養成所格納庫

Stresses of web members
Stresses due to Dead load and Snow load.

Members		Vert. shear.	chord vert. comp.	Net shear	Coef.	Stress
U_0-L_0	7.5P =	19,500 −	0 =	19,500	× 1.00 =	19,500 C
U_0-L_1	7.5P	19,500 −	4,800	14,700	× 1.50 =	22,100 T
U_1-L_1	7.5P	19,500 −	4,800	14,700	× 1.00 =	14,700 C
U_1-L_2	6.5P	16,900 −	7,200	9,700	× 1.37 =	13,300 T
L_2-U_3	5.5P	14,300 −	7,200	7,100	× 1.19 =	8,500 C
U_3-L_4	4.5P	11,700 −	8,900	2,800	× 1.19 =	3,300 T
L_4-U_5	3.5P	9,100 −	8,900	200	× 1.10 =	200 C
U_5-L_6	2.5P	6,500 −	8,800	−2,300	× 1.10 =	2,500 C
L_6-U_7	1.5P	3,900 −	8,800	−4,900	× 1.07 =	5,300 T
U_7-L_8	0.5P	1,300 −	7,700	−6,400	× 1.07 =	6,900 C
U_8-L_8		2@	7,700		=	15,400 T
U_2-L_2, U_4-L_4, U_6-L_6						2,600 C

Stresses due to vertical component of wind pressure.

Members		vert. shear	chord vert. comp.	net shear	Coef.	Stress
U_0-L_0	5.5W =	2,400 −	0 =	2,400	× 1.00 =	2,400 C
U_0-L_1	5.5W	2,400 −	600	1,800	× 1.50 =	2,700 T
U_1-L_1	5.5W	2,400 −	600	1,800	× 1.00 =	1,800 C
U_1-L_2	4.5W	2,000 −	900	1,100	× 1.37 =	1,500 T
L_2-U_3	3.5W	1,500 −	900	600	× 1.19 =	700 C
U_3-L_4	2.5W	1,100 −	1,000	100	× 1.19 =	100 T
L_4-U_5	1.5W	700 −	1,000	−300	× 1.10 =	300 T
U_5-L_6	0.5W	200 −	900	−700	× 1.10 =	800 C
L_6-U_7	−0.5W	−200 −	900	−1,100	× 1.07 =	1,200 T
U_7-L_8	−1.5W	−700 −	700	−1,400	× 1.07 =	1,500 C
U_8-L_8		2@	700		=	1,400 T
U_2-L_2, U_4-L_4, U_6-L_6						400 C

Stresses due to Crane load.

Members		vert. shear	chord vert. comp	net shear	Coef.	Stress
U_0-L_0	0.75C =	2,300 −	0 =	2,300 ×	1.00 =	2,300 C
U_0-L_1	0.75C	2,300 −	600	1,700 ×	1.50 =	2,600 T
U_1-L_1	0.75C	2,300 −	600	1,700 ×	1.00 =	1,700 C
U_1-L_2	0.75C	2,300 −	900	1,400 ×	1.37 =	1,900 T
L_2-U_3	0.75C	2,300 −	900	1,400 ×	1.19 =	1,700 C
U_3-L_4	0.75C	2,300 −	1,300	1,000 ×	1.19 =	1,200 T
L_4-U_5	0.69C	2,100 −	1,300	800 ×	1.10 =	900 C
U_5-L_6	0.63C	1,900 −	1,300	600 ×	1.10 =	700 T
L_6-U_7	0.56C	1,700 −	1,300	400 ×	1.07 =	400 C
U_7-L_8	0.50C	1,500 −	1,100	400 ×	1.07 =	400 T
U_8-L_8		2@	1100		=	2,200 T
U_2-L_2, U_4-L_4, U_6-L_6						0

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CALCULATIONS FOR
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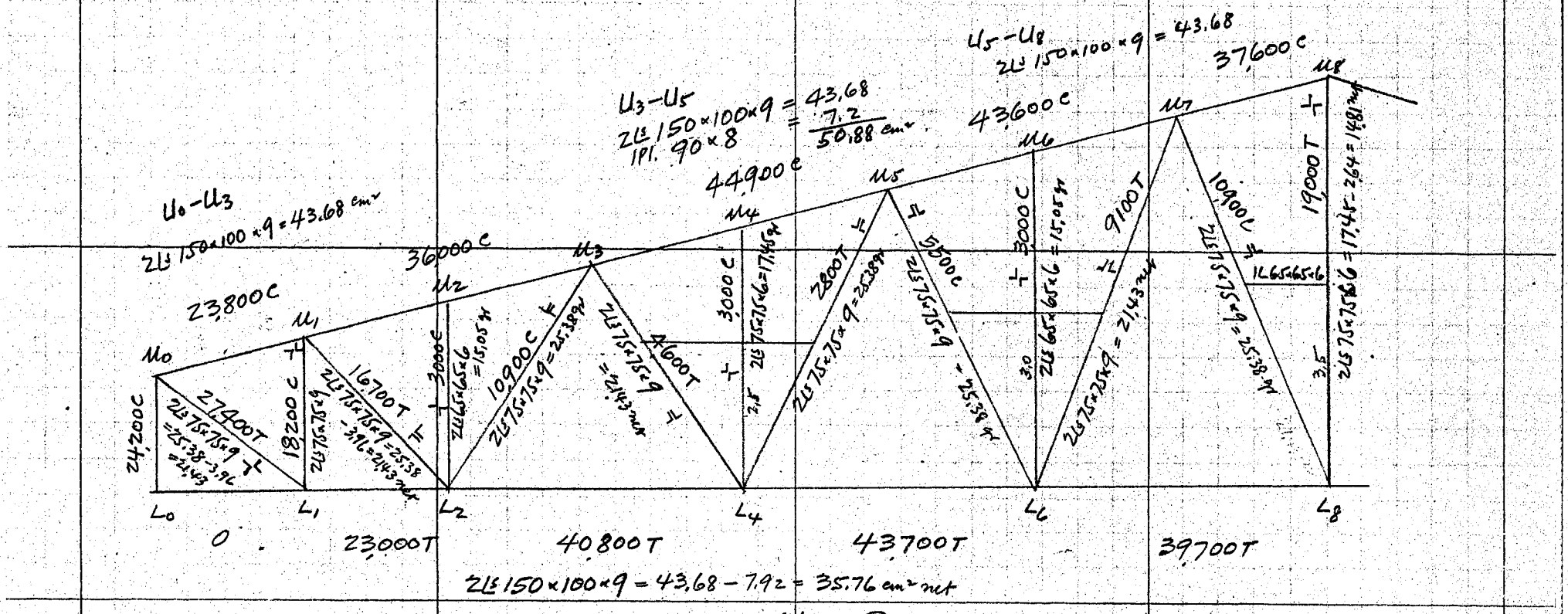
Summary of Stresses.

Members		Loads. Dead & Snow	Wind	Crane	Max. Stress. (in kg)
Top chords.	U_0-U_1	19,200 C	2,400 C	2,200 C	23800 C
〃	U_1-U_3	28,900 C	3,500 C	3,600 C	36,000 C
〃	U_3-U_5	35,700 C	4,000 C	5,200 C	44,900 C
〃	U_5-U_7	35,000 C	3,500 C	5100 C	43,600 C
〃	U_7-U_8	30,600 C	2,600 C	4400 C	37,600 C
Bott. chords.	L_0-L_1	0	0	0	0
〃	L_1-L_2	18,600 T	2,300 T	2,100 T	23,000 T
〃	L_2-L_4	32,600 T	3,800 T	4400 T	40,800 T
〃	L_4-L_6	34,800 T	3,800 T	5,100 T	43,700 T
〃	L_6-L_8	32,100 T	3,000 T	4600 T	39,700 T
Diagonals	U_0-L_1	22,100 T	2,700 T	2600 T	27,400 T
〃	U_1-L_2	13,300 T	1,500 T	1900 T	16,700 T
〃	L_2-U_3	8,500 C	700 C	1700 C	10,900 C
〃	U_3-L_4	3,300 T	100 T	1200 T	4,600 T
〃	L_4-U_5	200 C	300 T	(900 C) 2300 T	2,800 T
〃	U_5-L_6	2,500 C	800 C	(700 T) 2200 C	5,500 C
〃	L_6-U_7	5300 T	1200 T	(400 C) 2600 T	9100 T
〃	U_7-L_8	6900 C	1500 C	(400 T) 2500 C	10,900 C
Verticals.	U_0-L_0	19,500 C	2,400 C	2,300 C	24200 C
〃	U_1-L_1	14,700 C	1,800 C	1,700 C	18,200 C
〃	U_8-L_8	15,400 T	1,400 T	2200 T	19,000 T
〃	U_2-L_2	2600 C	400 C	0	3000 C
〃	U_4-L_4				
〃	U_6-L_6				



Stress Diagram

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CALCULATIONS FOR
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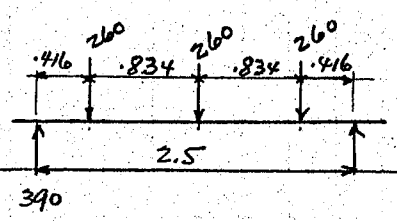
Section of each member.

Top chord U_3-U_5. stress 44900 kg C, Purlin concentration = 62×4.17 = 260 kg



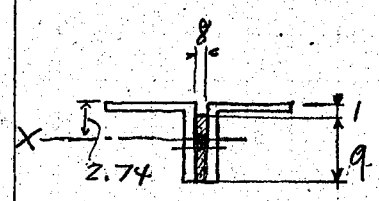
Moment on chord due to purlin concentration

390×1.25 = 488
−260×.834 = − 217
271

For continuity of chord, moment will be taken as 2/3 of above value.

$M = 271 \times 2/3 = 180$ kgm.

Assumed section



2Ls 150×100×9 = 43.68
1Pl. 150 ×8 = 12.00
55.68 cm² gr

Center of gravity of the section

2Ls	43.68 × 2.29	=	100.0
1Pl.	7.20 × 5.5	=	39.6
	50.88		139.6

$\frac{139.6}{50.88}$ = 2.74 cm below back of Ls

2Ls, $43.68 \times 0.45^2 + 2@175$ = 359
1Pl. $7.2 \times 2.76^2 + \frac{0.8 \times 9^3}{12}$ = 104
I_x = 463 cm⁴

radius of gyration $r_x = \sqrt{\frac{463}{50.88}} = 3.02$ cm

$\ell/r_x = \frac{250}{3.02} = 83$

Allowable unit stress = 1400×(1−0.03×83) = 1052 kg/cm² C

Max. unit compression in top chord.

Bending stress = $\frac{180 \times 100 \times 7.26}{463}$ = 282

Direct stress = $\frac{44900}{50.88}$ = 980 --- < 1052

1262 kg/cm² C < f_c = 1400

assumed section is ample.
Top chord U_5-U_7, see page 7.

Bottom chord

max stress = 43700 kg T

Section required = $\frac{43700}{1400}$ = 31.2 cm² net

Use 2Ls 150×100×9 = 43.68 − 7.92 = 35.76 cm² net

Diagonal members.

U_0-L_1

Stress = 27400 kg T S.R. = 19.56 cm² net
2Ls 75×75×9 = 25.38 − 3.96 = 21.43 cm² net

U_1-L_2

2Ls 75×75×9

L_2-U_3

Stress = 10900 C
2Ls 75×75×9 = 25.38
$\ell_r = \frac{407}{2.25} = 181$
$f = 1400(1-0.03 \times 181) = 640$ kg/cm²
$S = 10900 \div 25.38 = 430$

Unsupported length, taken as a distance between centers of rivet connections.

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CALCULATIONS FOR
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U_7-L_8 Stress = 10,900 c
2Ls 75×75×9 = 25.38 cm²-gr.

$l_y/r = \frac{345}{2.25} = 154$

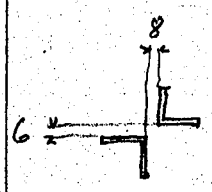
$l'_y/r' = \frac{685}{3.42} = 200$ $f = 1400(1-0.03\times200) = 560$ kg/cm²

Stress $= \frac{10900}{25.38} = 430$ kg/cm²

Vertical members

U_8-L_8 Stress = 19000 kg T SR = 13.58 cm² net
2Ls 75×75×6 = 17.45−2.64 = 14.81 " "

U_1-L_1 Stress = 18200 kg C



$l_y/r = \frac{263}{2.84} = 93.$ $f = 1400(1-0.03\times93) = 1010$ kg/cm² C

Stress $= \frac{18200}{25.38} = 718$ kg/cm² C.

use 2Ls 75×75×9 = 25.38 cm² gr. in X-section

Other verticals use 2Ls 65×65×6 = 15.05 cm² gr in X-section
Stresses, very small.

Top chord U_5-U_7
Stress 43600 kg C , M = 180 kgm.

2Ls 150×100×9 = 43.68 cm² Section modulus S.m = 45.46 cm³
r = 2.84 cm $l_y/r = \frac{2.50}{2.84} = 88$

allowable unit comp.
$f = 1400\times(1-0.03\times88) = 1031$ kg/cm² C

Unit stress.
Direct stress = $\frac{43600}{43.68} = 998 < 1031$

Bending stress = $\frac{180\times100}{45.46} = \frac{396}{1394}$ kg/cm² $< f_c = 1400$

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CALCULATIONS FOR
中央卸売食糧庁 格納庫

Dead load Deflection of main truss at center.

General equation of deflection $\Delta = \Sigma \frac{SL}{EA} T$ or $\frac{1}{E} \Sigma \frac{SL}{A} T$

where Δ = Deflection at any point in centimeter
S = Stress of each member
T = stress of each member due to a unit load on the panel pt, at which deflection is desired in the direction of the deflection.
L = length of each member in centimeter
A = sectional area of each member in cm^2 gross section
E = modulus of elasticity in kg/cm^2 = 2,100,000

Stress in each member due to a unit load at panel point L8. R = 0.500

Chord stress.

member.			Stress	vert. comp.
U_0-U_1	0.500 × 2.5 ÷	2.55	= − 0.490	− 0.123
L_1-L_2	〃 〃 ÷	2.63	0.475	
U_1-U_3	0.500 × 5.0 ÷	3.15	− 0.793	−0.198
L_2-L_4	0.500 × 7.5 ÷	3.88	0.967	
U_3-U_5	0.500 × 10.0 ÷	4.37	− 1.143	−0.286
L_4-L_6	0.500 × 12.5 ÷	5.13	1.218	
U_5-U_7	0.500 × 15.0 ÷	5.58	− 1.343	− 0.336
L_6-L_8	0.500 × 17.5 ÷	6.38	1.372	
U_7-U_8	0.500 × 20.0 ÷	6.80	− 1.471	− 0.368

Web stress

U_0-L_1	0.500 − 0.123 =	0.377 × 1.50	= 0.565	
U_1-L_1	〃 〃 =	〃 × 1.00	−0.377	
U_1-L_2	〃 − 0.198 =	0.302 × 1.37	0.414	
L_2-U_3	〃 〃 =	〃 × 1.19	− 0.359	
U_3-L_4	〃 − 0.286 =	0.214 × 1.19	0.255	
L_4-U_5	〃 〃 =	〃 × 1.10	− 0.236	
U_5-L_6	〃 − 0.336 =	0.164 × 1.10	0.181	
L_6-U_7	〃 〃 =	〃 × 1.07	− 0.176	
U_7-L_8	〃 − 0.368 =	0.132 × 1.07	0.141	
U_8-L_8	0.368 × 2		= 0.736	

Dead load stress 95/125 or 76% of Dead & snow load stress.

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CALCULATIONS FOR
中央乘員養成所格納庫

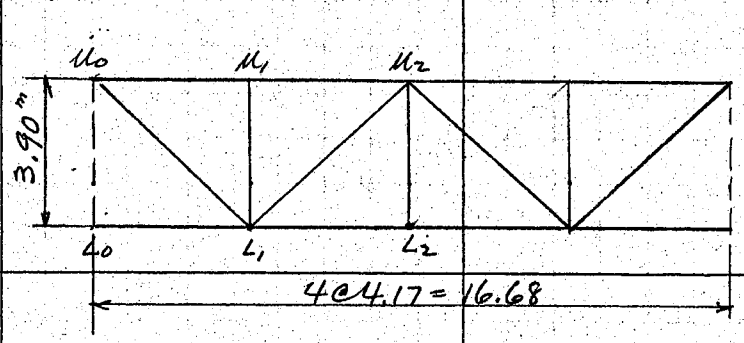
Deflection at center of span.

members	Length L	Area A	Unity T	LT/A	D.L. S.	SLT/A
M_0-M_1	258 cm	43.68 cm²	− 0.490	− 2.890	− 14,600	42,200
M_1-M_3	516	〃	− 0.793	− 9.370	− 22,000	206,000
M_3-M_5	516	50.88	− 1.143	− 11,590	− 27,200	315,500
M_5-M_7	516	43.68	− 1.343	− 15,880	− 26,600	422,000
M_7-M_8	258	〃	− 1.471	− 8,700	− 23,300	202,800
L_1-L_2	250	43.68	0.475	2.720	14200	38600
L_2-L_4	500	〃	0.967	11.070	24800	274200
L_4-L_6	500	〃	1.218	13.950	26500	369,500
L_6-L_8	500	〃	1.372	15.700	24400	383.000
M_0-L_1	300	25.38	0.565	6.680	16800	112.300
M_1-L_1	263	〃	−0.377	− 3,910	− 11,200	43,800
M_1-L_2	360	〃	0.414	5.870	10,100	59,400
L_2-M_3	460	〃	−0.359	− 6.510	− 6,500	42,300
M_3-L_4	460	〃	0.255	4.620	2,500	11,600
L_4-M_5	565	〃	−0.236	− 5,250	− 150	800
M_5-L_6	565	〃	0.181	4.030	− 1,900	7,700
L_6-M_7	685	〃	−0.176	− 4.750	4000	19,000
M_7-L_8	685	〃	0.141	3,810	− 5,200	19,800
M_8-L_8	700	17.45 × 1/2	0.736	14.750	11.700	172,500

Summary for one half span 2,743,000
〃 the entire span 5,486,000

Dead load Deflection at center = $\frac{5486000}{2,100,000}$ = 2.62 cm.

Longitudinal Truss between center columns supporting main truss.
Span length 16.68 meters, Depth of truss 3.90 meters about



Weight of beam assumed as 160 kg/m
panel load 160 × 4.17 = 670 kg.

truss concentration
Dead + snow load 7.5 × 2600 × 2 = 36000
wind 5.5 × 440 × 2 = 4,800
crane 0.75 × 3000 × 2 = 4,500
45,300 kg

Diagonal length = $\sqrt{3.9^2+4.17^2}$ = 5.71
Coef. = 5.71 ÷ 3.90 = 1.465

Reaction 670 × 1.5 = 1,000
45300 ÷ 2 = 22700
23,700 kg

L_1-L_2
23,700 × 8.34 = 197500
−670 × 4.17 = − 2,800
194,700 ÷ 3.9 = 49,900 kg T SR = $\frac{49900}{1400}$ = 35.70 cm² net

M_0-M_2
23700 × 4.17 ÷ 3.9 = 25,400 kg C

M_0-L_1
23700 × 1.465 = 34700 kg T SR. = $\frac{34700}{1400}$ = 24.80 〃

L_1-M_2 23700 − 700 = 23000 × 1.645 = 33700 kg C.

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CALCULATIONS FOR
中央来客養成所格納庫

Chord section M_0-M_2 Stress = 25400 c

⊥L 2L 130×130×9 = 45.48 cm²

$l_r = \frac{834}{5.6} = 149$

$l'_r = \frac{417}{3.96} = 105$

$f = 1400(1-0.03\times149) = 774$ kg/cm² c.

Stress = $\frac{25400}{45.48} = 559$ kg/cm² c

L_1-L_2 Stress = 49900 kg T Section required = $\frac{49900}{1400} = 35.7$ cm² net

⊥L 2L 130×130×9 = 45.48 − 7.92 = 37.56 cm² net

Diagonal section

M_0-L_1 Stress = 34700 kg T $S.R. = \frac{34700}{1400} = 24.8$ cm²

⊥L 2L 100×100×10 = 38.0 − 8.8 = 29.2 cm² net

L_1-M_2 Stress = 33,700 kg C

2L 130×130×9 = 45.48 cm² $l_r = \frac{5.71}{3.96} = 144$

$f = 1400(1-0.03\times144) = 795$ kg/cm² c

⊥L unit stress = $\frac{33700}{45.48} = 741$ kg/cm² c.

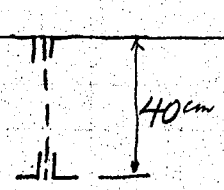
Roof Girder (下弦) span length = 5.0 m. spacing = 4.17 m c to c

load on girder

Cement slate	18
purlin	25
roof girder	15
Snow 30 cm.	30
misc fixtures etc say	12
	100 kg/m²

load on girder = 4.17 @ 100 = 417 call this 420 kg/m of span

moment = $\frac{1}{8}\times420\times5.0^2 = 1,315$ kgm



Effective depth say 40.0 − 2@2.05 = 35.9 cm

flange stress = $\frac{1315\times100}{35.9} = 3,660$ kg T or C

flg. area req'd = 3660 ÷ 1400 = 2.62 cm² net

use 2L 75×75×6 = 17.45 − 2.64 = 14.81 cm² net

Column C1 (下弦)

load on column $\frac{1}{2}\times5.0\times4.17$ @ 100 = 1040

Column say	3.5@40	= 140
awall "	2.0×3.5@30	= 210
misc.		110
		1500 kg

Wind moment 90 × 2.0 = 180 kg/m

$m = \frac{1}{12}\times180\times3.5^2 = 185$ kgm.

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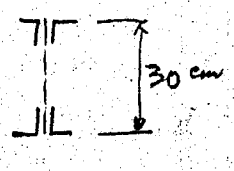
CALCULATIONS FOR
中央乘务员宿舍所拾納庫

Assumed section 4Ls 75×75×6 = 34.90 cm²



Moment stress = $\frac{185\times100}{25.9\times17.45}$ = 41

direct stress = $\frac{1500}{34.9}$ = $\frac{43}{84}$ kg/cm² c

Main Columns (本家)

load on columns.

Side columns.

		常時	地震時
Dead and snow load	7.5@2600 =	18,000	18000×110/125 = 15,800
wind	5.5@440 =	2400	—
Crane	0.75×3000 =	2300	—
		22700	
wall	8×8.34@30 =	2,000	2000
weight of column	12.0@100 =	1,200	1200
		100	100
		26,000 kg	19,100 kg

Center column.

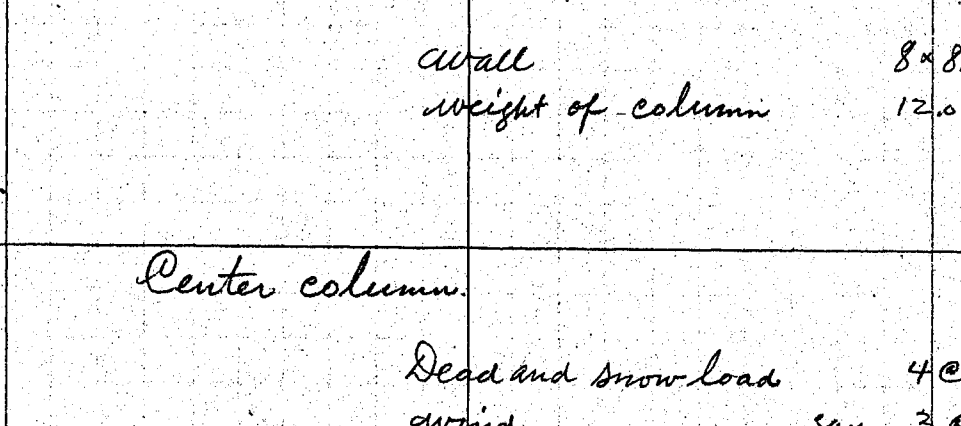
Dead and snow load	4@18,000 =	72,000		72000×110/125 = 63,400
wind	say 3@2400 =	7,200		—
Crane	2×½×2300 =	2300		—
		81500		
Center longitudinal beam	16.68@160 =	2,700		2,700
weight of column say	12.0@300 =	3600		3,600
		200		200
		88,000 kg		69900 kg

Wind pressure on vertical plane. (南北ノ方向ニ於ケル風圧ナリ、東西ノ方向ニ対シテハ14条参照)

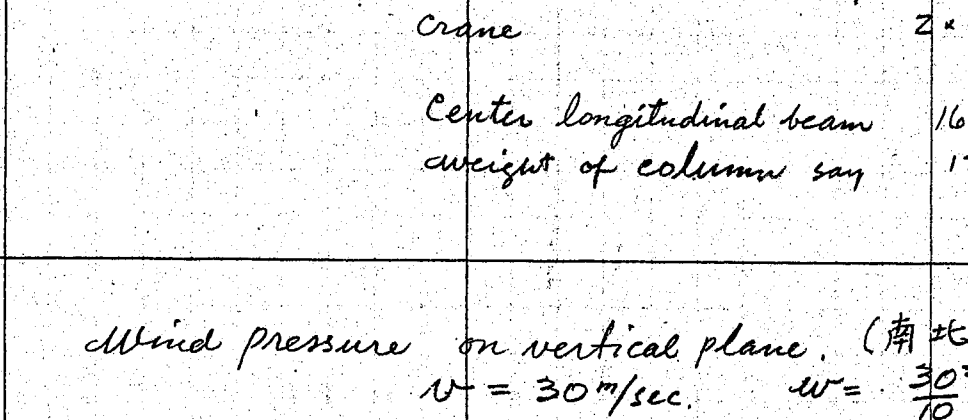
v = 30 m/sec. $W = \frac{30^2}{10}$ = 90 kg/m².

Wind load manner assumed as shown below.

$W_1 = 0.8\ W = 0.8\times90 = 72$ kg/m²
$W_2 = -0.4\ W = 0.4\times90 = -36$ "
$W_3 = (1.2\sin\delta - 0.4)W = -0.1W = -9$ "
$W_4 = -0.4W = -36$ "



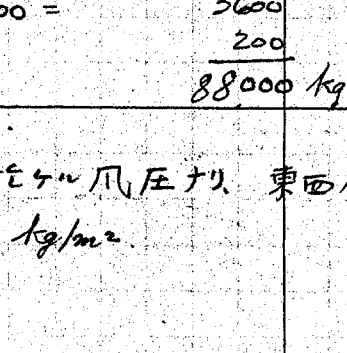
$W_1 = 72\times12.5 = 900$ kg $V_1 = 0$ $H_1 = 900$ kg
$W_2 = 36\times12.5 = 450$ $V_2 = 0$ $H_2 = 450$
$W_3 = 9\times20.6 = 18.5$ $V_3 = \frac{185}{1.03} = 180$, $H_3 = \frac{185\times1.25}{1.03} = 45$
$W_4 = 36\times20.6 = 740$ $V_4 = \frac{740}{1.03} = 720$, $H_4 = \frac{740\times1.25}{1.03} = 180$



採集風圧

270 kg
785
565

Total moment on column per meter.
H = 270 + 785 = 1055 kg



$1350\times\frac{7.25}{12.5} = 785$
$1350\times\frac{5.25}{12.5} = 565$

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CALCULATIONS FOR
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Moment on columns.
Top of column.
1055 × 3.25 = 3430 for 3 columns 3430 ÷ 3 = 1140 kgm for one col.
Bottom of column
1055 × 5.25 = 5540 " 5540 ÷ 3 = 1850 " "
Moment on side column.
Top of col.
1140 × 8.34 = 9500
Ecc. of load 18000 × 0.25 = 4500
14000 kgm
Vertical load say
−315 × 8.34 = −2600
26000
23,400 kg

Bottom of col.
1850 × 8.34 = 15400
ecc. of load say 4500 ÷ 2 = 2300
17,700 kgm

Moment on center column.
Top of col. 1140 × 16.68 = 19,000 kgm
Bottom " 1850 × 16.68 = 30800 "
Vertical load. say
−900 × 16.68 = −15,000
88000
73,000 kg

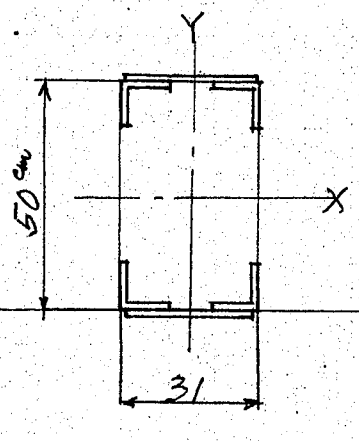
Moment during Earthquake
Acceleration assumed as 1000 mm/sec² or Coef. k = 0.100
Hor. thrust on side column = 19,100 × 0.1 = 1900 kg.
" " " Center column = 69900 × 0.1 = 6700 "

Moment on side column.
Top of column
1900 × 3.25 = 6200
Ecc. of load 15800 × 0.25 = 4000
10,200 kgm
Vertical load.
say 19,100 kg

Bottom of column.
1900 × 5.25 = 10,000
Ecc. of load say 4000 × ½ = 2000
12,000 kgm
19,100

Moment on center column.
Top of column.
6700 × 3.25 = 21,800 kgm
Bottom of column
6700 × 5.25 = 35,200 kgm
say 69,900 kg
69,900

Assumed cross section of Side column



Moment of inertia abt X-axis
4L 100×100×7 = 54.48 × 22.3² + 124×4 = 27600
2 cov. pls 300 × 8 = 48.00 × 25.4² = 31000
102.48 I_x = 58600 cm⁴

radius of gyration $r_x = \sqrt{\frac{58600}{102.48}} = 23.92$ cm.

Unsupported length l = 1050 − 200 = 850 cm $l/r = \frac{850}{23.92} = 35.5$

allowable unit comp. $f_c = 1400(1 - 0.03 \times 35.5) = 1250$ kg/cm² C.

Direct comp. = $\frac{23400}{102.48} = 229 < 1250$

Bending stress = $\frac{17700 \times 100 \times 25.8}{58600} = 779$

1,008 kg/cm² C < 1400

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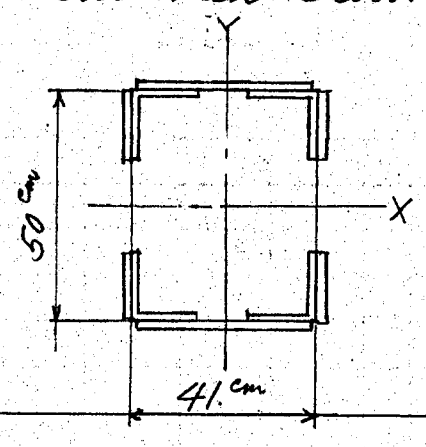
CALCULATIONS FOR
中央栄負養成所柱納庫

Assumed cross section of Center column.



Moment of inertia abt X-axis

4Ls 150×150×11 =	128.00	× 20.93² + 633×4 =	58600
2 Cov. Pls 400 × 12 =	96.00	× 25.6² =	62900
4 " Pls 150 × 16 =	96.00	× 17.5² + $\frac{1.6\times15^3}{16}\times4$ =	29700
	320.00 cm²	Ix =	151,200 cm⁴

128.00 × 16.43² + 633×4 =		37,100
1.2×40³ ÷ 12 =		6400
96 × 21.3² =		43600
	Iy =	87,100 cm⁴

$r_y = \sqrt{\frac{87100}{320}} = 16.5$ cm $\quad l_y = \frac{850}{16.5} = 52$

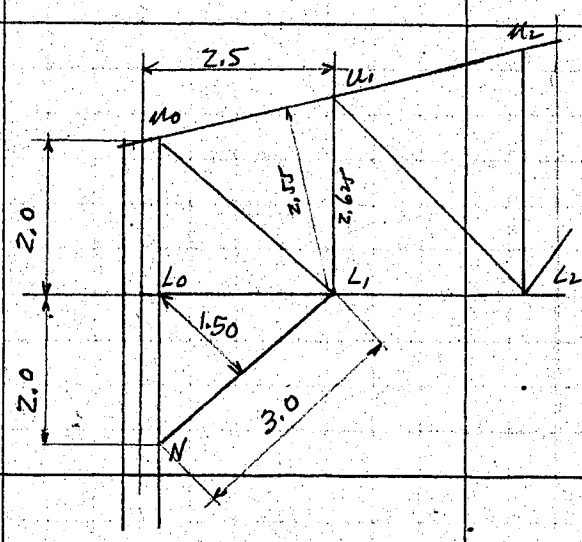
$f_c = 1400(1-0.03\times52) = 1,182$ kg/cm²c.

Direct comp. $= \frac{69900}{320} = 219 < 1047$

Bending stress $= \frac{35200\times100\times26.2}{151200} = \frac{610}{829}$ kg/cm²c < 1400

上記ハ南北方向ノ风圧ニヨル応力ナリ。東西方向ノ风圧ニヨル応力ハ 14頁ニ記載セリ。

Knee Brace.



Compression on knee brace near side column.

Say $\frac{14000}{1.50}$ kgm = 9350 kg T or C.

Do. near center column.

Say $\frac{21800}{2\times1.5} = 7250$ kg T or C.

Use 2Ls 150×100×9 = 43.68

~~$l_y = \frac{300}{2.84} = 106 \quad f_c = 1400(1-0.03\times106) = 955$ kg/cm²c~~

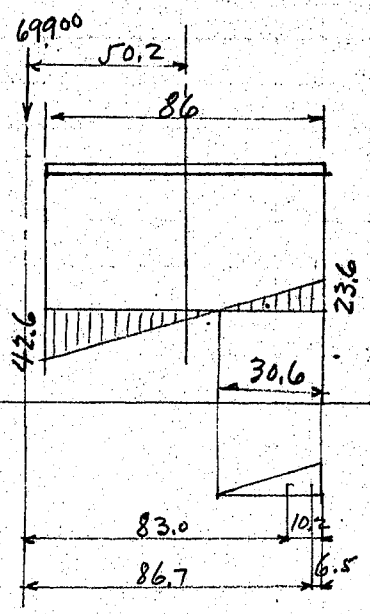
Stress $= \frac{9350}{43.68} = 214$ kg/cm² < 955.

Compression on truss member L_1-L_2

Say $\frac{14000}{2.625} = 5300$ kg

Column Base.

Center column.



Vertical load	69,900 kg
moment	35,200 kgm
Base area	86 cm × 86 cm = 7,400 cm²

$e_{cc.} = \frac{35200\times100}{69900} = 50.2$ cm

Bearing pressure $= \frac{69900}{86\times86}\left(1 \pm \frac{6\times50.2}{86}\right)$ (3.50) $= 42.6$ kg/cm²C

or 23.6 T

Use proper reinforcement under base pl. to take care of bearing pressure.

Tension on bolts

Total up lift $= \frac{86\times30.6\times23.6}{2} = 31,000$ kg T $\times \frac{83.0}{86.7} = 29,700$ kg T

2 anchor bolts 44φ net area = 2@11.25 = 22.5 cm² net

Stress on bolts $= \frac{29700}{22.5} = 1320$ kg/cm² < 1400

標準設計 第一輯 B4

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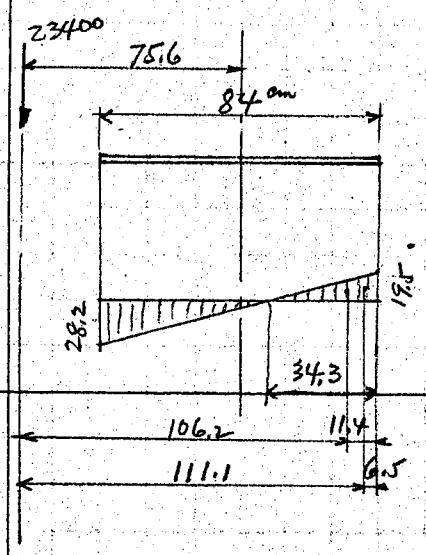
CALCULATIONS FOR
中央荣負養成所格納庫

Side column.



Vertical load 23400 kg
moment 17700 kgm
Base area = 64 cm × 84 cm = 5370 cm²

Ecc. = $\frac{17700 \times 100}{23400}$ = 75.6 cm

Bearing pressure = $\frac{23400}{64 \times 84}\left(1 \pm \frac{6 \times 75.6}{84}\right)$ (5.47) = 28.2 kg/cm² C
or 19.5 " T

Total up lift = $\frac{64 \times 34.3 \times 19.5}{2}$ = 21,400 kg T

Tension on bolts = $\frac{21400 \times 106.2}{111.1}$ = 20400 kg T

2 bolts 38φ net area = 2 @ 8.34 = 16.7 cm² net

Stress on bolts = $\frac{20400}{16.7}$ = 1222 kg/cm² T < 1400

Longitudinal wind pressure (東西方向ニ対スル風圧)

wind load on windward wall = 0.80 × 90 = 72 } = 108 kg/m²
" " " leeward " = 0.40 × 90 = 36 }

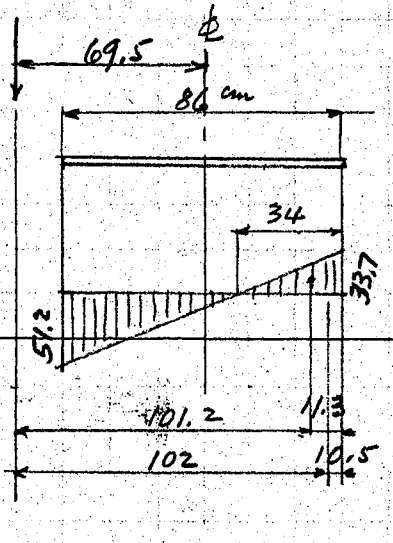
load on triangular portion ½ × 40" × 5.0" @ 108 = 10,800.
" " rectangular " 4.0 × 7.25 @ " = 31400
42,200 kg

Moment on center col. = $\frac{42200 \times 8.5}{2} \div 4$ = 44,900 kgm for one column.
vertical load on one col. = 88000 − 40 × 16.68 × 35 = 64,700 kg

Direct compression = 64700 ÷ 320 = 202 < 1182
Bending stress = $\frac{44900 \times 100 \times 22.1}{87100}$ = 1140
1342 kg/cm² < 1400

Bearing pressure on concrete.

Ecc. = $\frac{44900}{64700}$ = 69.5 cm



Bearing pressure = $\frac{64700}{86 \times 86}\left(1 \pm \frac{6 \times 69.5}{86}\right)$ = 51.2 kg/cm² C
or 33.7 " T

Total up lift = $\frac{86 \times 34 \times 33.7}{2}$ = 49200 kg × $\frac{101.2}{102}$ = 48800 kg

4 anchor bolts 44φ net area = 4 @ 11.25 = 45.0 cm²

Stress in bolt = $\frac{48800}{45.0}$ = 1085 kg/cm² T < 1400

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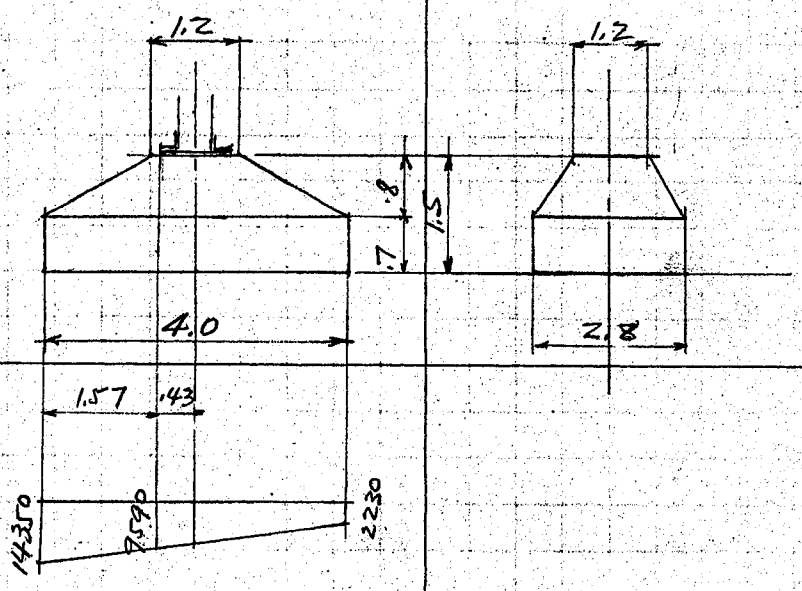
CALCULATIONS FOR
中央乗务員養成所橋

基礎応力計算書



Base for Center column.
Vertical load = 64,700 kg
moment = 44,900 kgm

Weight of Base.
4.0 × 2.8 × 0.7 = 7.85
2.6 × 2.0 × 0.8 = 4.16
12.01 m³ @ 2300 = 27600

Total vertical load = 64700 + 27600 = 92,300 kg

Eccentricity $e = \frac{44900}{92300} = 0.487$ m

Resultant force within middle third.

max. toe pressure $= \frac{92300}{4.0 \times 2.8}\left(1 \pm \frac{6 \times 0.487}{4.0}\right) = 14350$ kg/m² or 2230.

Bending moment on footing

			arm	
upward pressure = 2.8 $(\frac{14350 \times 9590}{2}) \times 1.57$ =		52,600 kg	× 0.84 =	44,200 kgm
Downward : base	2.8 × 1.57 × 0.7 @ 2300 =	−7,100	× 0.785 =	−5,600 "
" " "	2.0 × 0.4 × 1.57 @ " =	−2900	× 0.6 =	−1,700 "
" " earth	2.8 × 0.4 × 1.57 @ 1600 =	−2800	× 1.0 =	−2800 "
		34,100 kgm		

Effective depth required $= \sqrt{\frac{34100 \times 100}{280 \times 7.60}} = 40$ cm use d = 145 cm insulation 5 cm

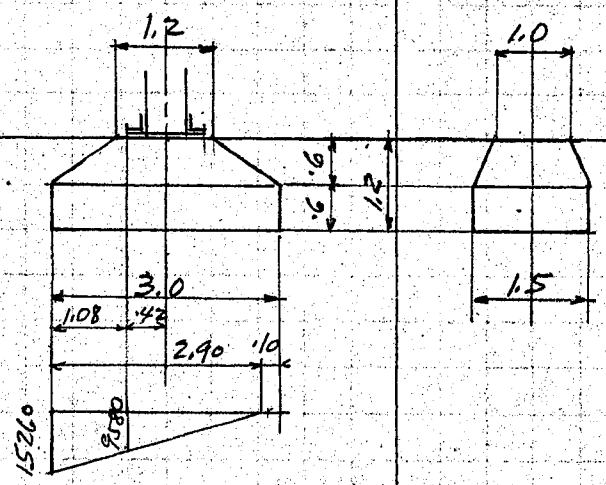
Steel area req'd. $= \frac{34,100 \times 100}{1400 \times 0.9 \times 145} = 18.70$ cm²

use 7 − 19φ bars = 19.85 cm² (6 spacings @ 45 cm = 2,700)

Bearing reinforcements.
8 − 19φ bars with hooping under base pl. of col.

備考 上記1直角1方向ニ於ケル基礎 Bearing pressure ハ 15020 / 1930 kg/m² ナリ.

Base under side columns.



Vertical load = 23400 kg
moment = 17,700 kgm

Weight of Base.
3.0 × 1.50 × 0.6 @ 2300 = 6200
2.1 × 1.25 × 0.6 @ " = 3600
9800 kg

Total vertical load = 23400 + 9800 = 33,200 kg

Eccentricity $= \frac{17700}{33200} = 0.534$ m

Resultant force out side of middle third, neglecting tension on heel,
pressure area (1.50 − 0.534) × 3 = 2.90 m
2.90 × 1.50 = 4.35 m²

max. Toe pressure $= \frac{33200 \times 2}{4.35} = 15,260$ kg/m²

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CALCULATIONS FOR
中央乘务养成所桥纳库

Bending moment on footing

Upward pressure $\frac{15260+9580}{2} \times 1.08 \times 1.5 =$ 20100 kg arm $\times 0.58 =$ 11,700

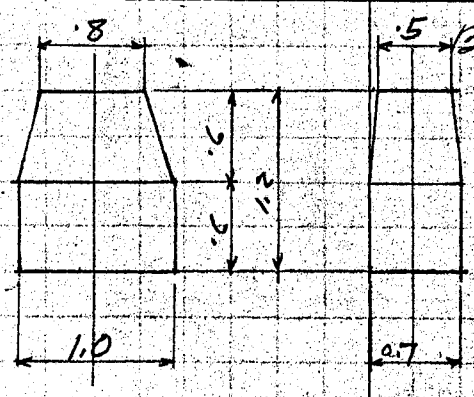
Downward " base $1.5 \times 0.6 \times 1.08 @ 2,300 =$ 2200 $\times 0.54 =$ 1200
" " " $1.25 \times 0.3 \times 1.08 @$ " $=$ 900 $\times 0.4 =$ 400
" " earth $1.5 \times 0.3 \times 1.08 @ 1600 =$ 800 $\times 0.7 =$ 600
9,500 kgm.

Effective depth req'd. $= \sqrt{\frac{9500 \times 100}{150 \times 7.13}} =$ 30 cm use d = 115 cm insulation 5 cm

Steel area required $= \frac{9500 \times 100}{1700 \times 0.9 \times 115} =$ 6.56 cm²

use 4-16φ bars = 8.04 cm² (3 spacings @ 45 cm)

下家柱 load 1500 kg
m = 180 kgm.



Base area 1.0 × 0.7

wt. of base = $1.0 \times 0.7 \times 0.6 = 0.42$
$0.9 \times 0.6 \times 0.6$ 0.32
$0.74 @ 2300 =$ 1700 kg

Total vert. load = 1500 + 1700 = 3200 kg

Ecc. $= \frac{180}{3200} = 0.056$

max. bearing pressure $= \frac{3200}{0.7 \times 1.0}\left(1 \pm \frac{6 \times 0.056}{1.0}\right) =$ 6120 kg/m² ≈ 3040