THE DESIGN OF A 3-HINGED STEEL ROOF TRUSS

Thesis for the Degree of B. S.

Julius L. Kleinfeld

1937

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STEEL ROOF TRUSS

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A THESIS

Submitted to the faculty of the
Michigan State College

By
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Bachellor of Science

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DESIGN OF THREE HINGED STEEL ROOF TRUSS FOR PROPOSED INDOOR SKATING ARENA

THE urgent need for a stadium of large proportions to be used as a combination building adaptable to ice skating, indoor pageants, conventions, and tournaments has existed now for the past several years. Since it is the opinion of several prominent citizens that this stadium might be built in the near future, I have therefore made this initial step toward a complete design, hoping to encourage a spirited movement toward the fulfillment of this idea.

The part played by the Michigan State College and it's faculty cannot be overpraised. In their conscious and continuous efforts to improve the life and living of the citizens of Michigan and of the United States. Always alert to be of use to the public, it has introduced a real spirit of progress into the consciousness of it's students. Much more can be said in praise of this great institution, but the gist of all such comments would be that of praise- praise for the administration, the faculty, and, egotistically, for the students. I can only add that it is the duty of the public and state administration to realize its' dutytowards this school to its'fullest extent and give it all the real cooperation necessary.

There is a great deal more that I would like to say relative to this theme of administrative and public cooperation, and perhaps an opportunity will be afforded me to do so in the near future, but at the present time the matter of recording my design is more urgent.

This thesis is written as part of the require- for the degree of Bachelor of Science at the Michigan State College, and I wish to take this opportunity now, to express my sincerest appreciation to Professor C.M. Allen, Mr. J. E. Meyer, and others of the department of engineering, for their suggestions and comments in aiding me to prepare this paper.

Julius L. Kleinfeld
Michigan State College
E.Lansing, Michigan
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PRELIMINARY CONSIDERATIONS

The primary purpose of this design is to provide an enclosed stadium containing an unobstructed area of at least two hundred by three hundred feet on each side. This will be sufficient to accomodate an ice surface one hundred by two hundred feet on each side, allow for an unobstructed view to the spectators seat- on each of the surrounding three sides and permit the construction of a mammoth stage located on the fourth side of the rectangular area.

It will be necessary to provide locker -rooms for members of the hockey teams, rest rooms and toilets for both men and women, sufficient accomodations for the public to facilitate donning of ice skating shoes in place of street shoes and vice versa, dressing rooms and storage rooms in connection with the stage, and miscellaneous such as refreshment booths, ticket offices and offices of the building.

A preliminary investigation of a Pratt-type truss resulted in the conclusion that while it is relatively easy to construct a long span truss of this type, it's extraordinary weight would be uneconomical for this structure. The use of that type of a truss in the recently constructed Kansas City Auditorium was successful economically because of the unusually gigantic size of the completed structure. The use of three hinged trusses in buildings of this type, that I am proposing, have been successfully used in the University of Illinois DrillHall, and in the St. Louis City Auditorium, built in 1913 and 1917 respectively. There are several features incorporated in my design which are a direct result of the improvement of the quality of steel and of greater knowledge as to the limitations of the structural materials. Two of these are (1) greater spacing between trusses, 50 feet, (2)and lighter members furnishing the required rigidity and strength.

DESIGN DATA

The type of truss to be used will be a three-hinged open web steel arch. Center to center of abuttment pins will be two-hundred - six feet and the height from floor surface to the axis of the center pin will be sixty-two and one-half feet.

The loadings will be taken as follows: Snow load as twenty-five pounds per square foot of roof surface; Unit wind pressure will be taken as thirty pounds per square foot of vertical surface. These figures are in conformity with present day knowledge and were obtained from Hool and Kinne," Steel and Timber Structures".

The general requirements governing the design of the steel work will conform to the standard practice for this type of structure. Working stresses used in the design of the structure will be those Standard Specifications for Structural Steel Buildings as Adopted by the American Institute of Steel Construction.

ALLOWABLE STRESSES (maximum pounds per sq. inch)

Tension	18000
Compression	$\dfrac{18000}{1+\dfrac{L^2}{18000\,r^2}}$
with a maximum of --------	15000

in which L is the unsupported length of the column and r is the least radius of gyration of the section, both in inches. $\frac{l}{r}$ less than 120 (main compression members)

Shearing on pins	13500
Bearing on pins, single shear	24000

Rivets will be taken as three-fourths inch in diameter and rivet holes punched one-sixteenth inch larger than the rivet diameter. In calculating net areas of tensoin the diameter of rivet holes will be taken one-eighth in. larger than the rivet or seven-eighths inch.

WIND LOADS

Wind pressure will be taken as equal to thirty pounds per square inch of vertical roof projection. Since the friction of air is light against comparatively smooth surfaces, it was assumed that only the normal effect of this pressure need be considered. The amount of normal pressure for varying slopes is given by Duchemin in the following formula.

$$Pn = Ph\left(\frac{2 \sin A}{1 + \sin A}\right)$$

in which Ph = the horizontal pressure per sq. ft. on vertical surface,

Pn = the normal pressure per sq. ft. of sloping surface,

A = the angle of inclination of the sloping surface.

For a horizontal pressure of 30 lb. per sq. ft., the values of the total Pn per panel of curved roof surface are calculated on the following table.

Panel	Angle	Lbs per sq. ft.	Total Lbs pn
1	90	30	9000
2	90	30	9000
3	90	30	18000
4	32-10	25	18250
5	26-40	22	18500
6	25	22	18320
7	16	15	14900
8	12-30	12	10450
9	9 -20	10	8370
10	8- 25	10	7150

Calculations:

1. $30 \times 50 \times 6 = 9000$ lbs.per sq.ft.

2. $30 \times 30 \times 6 = 9000$ " " " "

3. $30 \times 30 \times 12 = 18000$ " " " "

4. $(30 \times 6 \times 30)\ (25 \times 7.4 \times 50) = 18250$ lb.per sq.ft.

5. $(25 \times 7.4 \times 50)\ (22 \times 8.4 \times 50) = 18500$

6. $(22 \times 8.4 \times 50)\ (22 \times 8.25 \times 50) = 18320$

7. $(22 \times 8.25 \times 50)\ (15 \times 7.8 \times 50) = 14900$

8. $(15 \times 7.8 \times 50)\ (12 \times 7.7 \times 50) = 10450$

9. $(12 \times 7.7 \times 50)\ (10 \times 7.5 \times 50) = 8370$

10. $(10 \times 7.5 \times 50)\ (10 \times 6.8 \times 50) = 7150$

DESIGN OF RAFTERS

The composition of the roof covering will be a 5-ply felt and asphalt material, constructed by expert workman, and will be supported by one inch no. 2 yellow pine roofing planks, which will in turn rest on wooden joists, spaced at two foot intervals on centers.

RAFTER LOAD

(1)

Roofing and sheating $10\frac{1}{2}$ lb.× 2'×16.80	= 353 lbs.
Rafters 2"×10/12×16.80× $4\frac{1}{2}$	= 126
Snow load 16.80×2'× 25 lb.	= 840
	1319 "

(2)

Roofing and sheating	= 353
Rafters	= 126
$\frac{1}{2}$ Snow load	= 336
Normal wind 20lb.× 2'×16.80	= 739
	1554 "

M=1/10 wl = 1/10 1554 16.8 2 = 52,300 "#/

Max.shear S =1554 lb.

Select 2"×10" yellow pine joists

Allowable stress:	bending	1650 lb./sq."
	shear//grain	240
	shear ⊥ grain	150
	bearing // "	1800
	bearing "	350

$$Fm = \frac{Mc}{I} = \frac{52,300 \times 5}{2 \times 1000/12} = 1570 \text{ lb./sq.in. (all.1650)}$$

Req'd bearing area $= \frac{1554}{330} = 4.72''$ use $6''$ ($3''$ lap at each end).

Design of Purlin:

The design of the purlin will be subject to the following considerations of it. Firstly, purlins will be placed at panel points only. There will be therefore seven purlins ,spaced as shown, for each half of the truss. Exceptions to the general design of the purlin I have indicated will be necessary at the panel next to the center pin, where a purlin of only four foot depth will be required, and at the end panel where end architectural details might require some variations.

The purlin designed will be 50'(-) from truss to truss, 5' in depth, and consist of 10 panels each 5 ' in width. The lower end chord of each purlin will be cambered at angle so as to be supported at the upper and lower extremity of the vertical members of the truss.

In the design of the purlin as well as that of the truss the maximum stressed members ,only will be designed and repeating sections used as much as permissible.

DESIGN OF PURLINS

Purlin load 43154 lbs.

left reaction 21577 lb.

Shear panel (1) 21577 - (1554+55×21 /2+(1+3)/5 (1554)

=18416

Bending panel (5) = 21577×25 - (1554)(25+23+21+19

+17+15+13+11+9+7+5+3+1)

+ (45×25×25/2)

=262350 ft.lb.

=3148200 in lb.

Upper chord: compression member

shearing stress =18446

area req'd = 18446/12000=1.54 sq.in.

bending stress =3148200/60 =52500 lb.

area required = 52500/15000 =3.50 lb./sq.in.

Try 2 angles 3"×3"×3/8"

A = 2.11 S =.88

W = 7.2 lb. r =.91

I =1.8 x =.89

area furnished = 4.22 sq. in.

r = 2(1.8/4.22) = .92

l/r = 60/.92 = 65 ok.

18000/1+l² /18000×r² = 14, 400 lb.

area required is 52,500/14400 =3.62 sq.in.

area furnished is in excess of this and 4.22 sq.in.

PURLINS

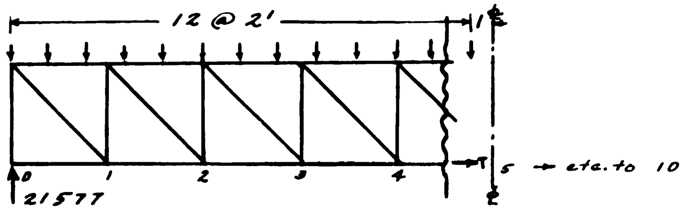
5 ply roofing	7.00 lb.per sq.ft.
1" yell. pine sheating	3.50 " " " "
ceiling	7.00 " " " "
Total	17.50 " " " "

Rafters: 26-2"×10"×17' @ 7.5 lb.per ft. = 127.5#/rafter

Purlin load:

(1(	roofing	17.5 × 50 × 16.8	14690 lb.
	purlins	45 × 50	2250
	snow	25 × 50 × 15	18750
	rafters	26 × 127.5	3320
	Total		39010 lb.
	1/3 wind load 15/3 15 ft. 50	=	3750
	Total		42760 lb.
(2)	roofing		14690
	purlins		2250
	wind load(max)	25×50×17	21220
	rafters		3320
	Total		41480 lb.
(3)	roofing		14690
	purlins		2250
	1/2 snow		9375
	wind load		15000
	Total		41315 lb.

Lower chord: tension member



5×12"×T = (21577×20) − (1554)(20+18+16+14+12+10

+ 8+6+4+2)+(45×20×10)

T = 2971920 "lb. /60 = 49,532 lb.

req'd area = 49,532/18000 = 2.755 sq.in.

area of 2 rivet holes- 7/8" dia. — 2(.601) = 1.202

total area req'd = 3.957 sq.in.

2 angles ∟- 3"×3"×3/8"

total area furnished is 4.22 sq.in.

Diagonal:

All the diagonals of the purlin are tension members and as in the design of the previous members, a design will be made for the diagonal carrying the most stress, and this design repeated for the other diagonals. The diagonal in panel(1) carries the greatest stress.

$Tv = 21880 - 1554 + (3+1/5)1554 + 45 \times 5 \times 5/2$

$Tv = 18020$

$T = 18020/.707 = 25500$ lb.

allowing for 2-7/8" rivet holes, area required is $25500/18000 + 2(.551) = 2.6$ sq."

try 2 angles 3"× 2-1/2"× 3/8"

area furnished= 2(1.62)=3.24 sq."

Vertical:

These are all compression members. As the end verticals will be subjected to the greatest stress, the design will be made accordingly.

Stress = 15446 lb.

area req'd = $15446/10000 = 1.54$ sq."

try 2 angles 3"× $2\frac{1}{2}$"× 3/8"

$A = 1.62$	$S = .81$
$W = 6.6$	$r = \sqrt{I/A} = .94$
$I = 1.7$	$x = .96$

$l/r = 60/.94 = 64$

$s = 18000/\ 1 + l^2/18000\ r^2 = 14690$ ok.

Weight of purlin:

200'- 3"×3"×3/8" angles at 7.2 lb./' =1440 lb.

198.4'-3"×3/8"×2½" " " 6.6 " " =1245

rivets " = 50

Total =2735 lb.

2735/50 = 54.7 lb./ ft. (assumed 55 lb./ ft.)

Purlin load:

purlin and cieling load 2300 lb.

26 rafters at 1554 lb. 40400 lb.

Total 42700 lb.

Truss load per panel:

Roofing, sheating and ceiling 14800 lb.

Rafters 3510

Purlins 2000

Total 20310 lb.

panel load = 20310/2 =10150 lb.

purlin: 10150/50×15 =13.5#/sq.ft.hor.proj.

truss: =15.0 " " " "

truss load per panel 28.5 " " " "

A loading of 10,000 lb. per panel will be used and the maximum stress will be proportionately increased in the ratio of 28.5/13.5 .

DESIGN OF TRUSS MEMBERS

Top Chord:

Because of the variation of stress in the top chord, some of the members having maximum stresses while in compression while others have maximum stresses when in tension, a member will be designed which will satisfy the greatest requirement in either case. This variation of stress in the top chord of three-hinged trusses of this type is not unusual and it is better to have the top chord uniform and artistic in appearance than to allow economical and diverse sections to be used.

U_3 - U_4 (tension member). 3/4"rivets . A=.442 sq.in.

S = 23[illegible]450 lb.

A = 2[illegible]/16000 + 4(.442) = 16.015 sq.in. necessary

2-15 lb. [s 9"×2½" = 8.78 " "

1- C.P. 16"×7/16" = 7.00

15.78 " " furnished

U_5 - U_6 (compression member).

S = 149000 lb.

A - req'd = 149000/13500 = 11.1 sq.in.

2-15.4 lb [s 9" 2½" = 8.78 " "

1- C.P. 16 7/16" = 7.00

15.78 " " furnished

$$\frac{16000}{1+\frac{l^2}{18000\,r^2}} = 15000 \text{ ok. (used 13500)}$$

L/r = 39.4 ok. (less than 125)

$U_8 - L_{10}$ (tension)

S =105200

Area req'd.=105200/ 18000

= [illegible]25 sq.in. + 2(.412) = 7.01[illegible] sq.in.

Use 15"- 30 lb. I . Area = 8.7[illegible] " "

Members $U_2 - L_4$, $U_3 - L_5$., $U_4 - L_6$, $U_5 - L_7$, $U_7 - L_9$, $U_8 - L_{10}$, ($U_6 - L_8$), will all be made of the same size member.

BOTTOM CHORD

Maximum stress occurs in bar $L_3 - L_4$ and equals 5[illegible]5000 lb. compression.

$L_4 - L_{10}$ has maximum stress of 497 500 lb.comp.

$L_0 - L_3$ has maximum stress of 406 [illegible]00 lb.comp.

member $L_0 - L_2$:

Area req'd =40[illegible]00/15000 = 27.05 sq"

2- 35 lb.[s	10"× 3 1/8"	=20.54	"	furnished
1 - C.P.	16"× 7/ 16"	= 7. 00	"	"
	Total	27.54	"	"

member $L_2 - L_5$:

Area req'd = 5[illegible]500/15000 = 37.5 "

2- 35 lb.[s	10"× 3 1/8"	= 20.54	"	"
2- C.P.	16"7/ 16"	= 14.	"	"
1- C.P.	16" 3/8"	= 6	"	"
	Total	40.54	"	"

member $L_5 - L_{10}$;

Area req'd.=497500/15000 = 33.18 " "

2- 35 lb [s =10" 3 1/8 = 20.54 " "

2- C.P.	16" by 7/16"		14.00 Sq."
		Total	34.[illegible]4 " "

Diagonals - $U_2 - L_5$, $U_3 - L_6$, $U_4 - L_7$, $U_5 - L_8$, $U_6 - L_9$,
$U_7 - L_{10}$.

maximum stress = 105260 lb. tension

area req'd = 105260 / 18000 = 5.85 sq.in.

= 5.85 + 2(.601) = 7.05 " "

2-Ls 2½"×2½"×5/16" = 3.5 " " furn.

(less 10% outstanding leg)

Member A - L_1

stress = 71090 lb. tension

area req'd. = 71090 / 18000 = 3.95 sq.in.

= 3.95 + 4(.442) = 4.718 " "

2-Ls 2½"× 2½"×3/8" = 3.46 " "

1- plate 10"×3/8" = 3.75 " "

Total 7.21 " "

(lug angles used at connections)

rivets: 71090/2(5,960) = 6 rivets

Members B - L_1

stress = 269400 lb. tension

area req'd. = 269400 /18000 = 14.97 sq.in

= 14.97+4(.442) = 16.74 " "

4- Ls 3½"×3½"×½" = 13.00 " " prov.

1- plate 10"×7/16" = 4.3 " " "

Total 17.3 " " "

rivets: 209400 / 9000 = [illegible] use 30, 15 in each leg.

member B - L_2

stress = 1[illegible]0 lb. compression

area req'd. = 1[illegible] / 1[illegible] = 1[illegible].41 sq.in.

4- ∠s 3"×3"×5/8" = 8.46 sq.in [illegible]rn.

1- plate 10"×7/16" = 4.5 " " "

Total 1[illegible].74 " " "

I = [illegible] + .[illegible]

A = [illegible]+4.37 = [illegible] sq.in.

$r = \sqrt{I/A}$ = .[illegible]

l/r = 81.6"/.[illegible] = 1[illegible] ok.

$$\text{stress} = \frac{1[illegible]00}{1 + \frac{L^2}{1[illegible]000\, r^2}} = 16{,}550 \text{ lb. ok.}$$

member C - L_2

stress = 110,[illegible]00 lb. tension

area req'd. = 110,[illegible] / 1[illegible]000 = 6.17 sq.in.

= [illegible] 6.17 4(.[illegible]42) = 7.9[illegible]" "

1-[s 30 lb. - 10" [illegible] = 8.8[illegible]0 " "

member C - L_3

stress = 94,100 lb. comp.

area req'd. = 94,100 / 15000 = 6.[illegible] sq.in.

1-[s 10" [illegible] 7/8" - [illegible]lb. = 7.35 sq.in.

least r = 0.6[illegible]

L = 99"

99/0.[illegible] = 14[illegible] (only allowed 120)

try 2- $\angle$s 2½"×2½"×½" = 4.50 sq.in.

1- plate 10"×3/8" = 3.75 " "

Total 8.25 " "

99/ 1.28 = 77 ok

allowable stress $= \dfrac{18000}{1+\dfrac{L^2}{18000\,r^2}}$ = 16550 lb. ok

member D - L_3 , D - L_4 the same as C - L_3

member U_1 - L_4

stress is 1[illegible],000 lb. compression

area req'd is 1[illegible],000/ 15000 = 13.0[illegible] sq.in.

4-$\angle$s 2½"×2½"×½" = 9.00 " "

1- plate 10"×½" = 5.00 " "

Total 14.00 " "

L/r = 210/[illegible] = [illegible]1.6 ok

allowable stress $\dfrac{18000}{1+\dfrac{L^2}{18000\,r^2}}$ = 18000/1.[illegible] = 15180

try 4-$\angle$s 3"×3"×5/8" if the following doesn't succeed.

4- $\angle$s 3"×3"×½" for a=11.00 sq.in.

1- plate 10"×½" " = 5.00 " "

Total " 16.00 " "

from Carnegie handbook,

$$r = 2.[illegible] + 0.[illegible]2 = 2.9[illegible]$$

$$l/r = 210/2.9[illegible] = 7[illegible] \quad \text{ok.}$$

allowable stress,

$$\frac{18{,}000}{1 + \frac{L^2}{18000\, r^2}} = 14{,}1000 \text{ lb.}$$

check area,

196,000 / 14,100 = 13.9 sq." req'd

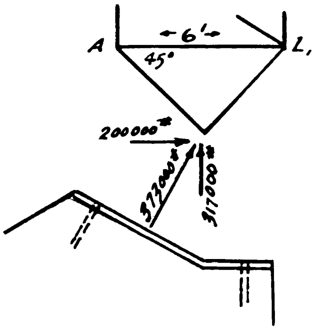
= 16.0 " provided ok.

rivets are in single shear,

196,000 / 2([illegible]60) = 16.44 , use 18 rivets

9 in each leg.

Design of shoe pin and connections:



7/8" rivets will be used-

allowable stress: shear 12000

bearing 24000

$\tan^{-1} \cancel{\theta} = 200,000 / 317000 = 32° 15'$

$R = 100 \sqrt{20^2 + 31.7^2} = 373,000$ lb.

maximum vertical reaction is 317,000 lb.

If the allowable bearing ona concrete foundation is taken as 400 lb. per sq.in. the area of the base of the shoe must be

373000/400 = 932.5 sq.in., at an angle of 32° 15'.

The bearing area between the members and the pin must be sufficient to keep the bearing pressures within the allowable limits, which will be taken as 24000 lb. per sq. in. and, the extreme fiber stress due to bending, considering the pin as a simple beam will be taken within the allowable limits as 25,000 pounds per sq. inch.

A 6- inch pin will be assumed. The load brought down by the pin to the shoe is equal to the reaction, 373,000 lb.

The width of bearing required on the web of the shoe is

$$373,000/6(24000)(2) = 1.11 \text{ in. for each web.}$$

assuming that a cast-steel shoe is used, the webs will be made 2" thick.

The load brought down by the arch to the pin is equal to the resultant of the horizontal and ver- components of the maximum reaction, which is 373,000 lb. The width of bearing required at the lower end of the arch truss is $373,000/(2)(6)(24000) = 1.1$ inches.

The main gusset plate at the joint will be 5/8" thick.

The thickness of the angles is 13/16". Add 2-5/8" plates on each side ,

total = 2 7/16"

this is the thickness on each leg of the channel iron, and though in excess of the area required ,it was desired that a rigid detail be provided rather than use a lesser number of plates. A clear space of 1/4" is provided between the several members.

The allowable stress for 7/8" rivets,

double shear = 14,430

bearing 7/8"×5/8"×24000 = 13125 lb.

The rivets connecting the 10"[s to the plates are in double shear, when both [s are assumed to act together. For the allowable shearing value given, the double shear value of a rivet is 14430 lb. The number of rivets required is 86400/14430 = 6 rivets. Assuming that the 3/8" filler plates and the channel and gusset plates act together, the total load to be carried is (86400 + 78500) = 164900 lb. As shown in the detail, the connecting rivets are in bearing on the 5/8" gusset plates, and hence the number of rivets required is 164900/13125 = 13 rivets.

The bending moment of the pin is determined by calculating the moment of the resultant.

B.M. 378000/2 + 2-31/32" = 564000 in.lb.

From the table (Carnegie Handbook) the allowable stress for 6" pin with a fiber stress of 25000 lb. at the outermost surface is 530,100 in.lb. It is necessary to use the next larger size of pin and that is a 6-1/4" pin.

The pin plates added to the gusset plate, in order to increase the width of bearing on the pin must be fastened to the [illegible] plate so that all the plates will act as a unit. Assuming that the load carried by each plate is proportional to its' thickness, the load carried by each [illegible]" filler plate is [illegible] ×([illegible]) = [illegible] lb., 11/16"-[s is [illegible] $\frac{11}{16}$" / [illegible] = [illegible],400 lb. The figure also shows the details of the shoe designed to carry the vertical and horizontal components of the reactions. The slope of the base of the shoe is determined by the condition that it should be perpendicular to the resultant of the maximum conditions of the reactions.

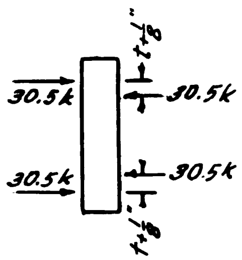
The base area that is required on the line x-x, must provide for the maximum reaction of 370000 lb. It is usual to provide a short horizontal base area , shown x- y .

Design of the Center - pin:

The design of the center pin is made in the same manner as for the footing pin. 7/8" rivets will be used. The allowable stresses

double shear	14400 lb.
bearing	[illegible] lb.

$\frac{14430 \quad 12000}{2} \times \sin 15°15' = 30500$ lb.



Assume a 3" pin.

Allow. bearing is 24000 lb.

Allowable B.M. is 63000 lb.

Plate thickness required

$$t = \frac{30500}{3 \quad 24000} = .424\text{"}$$

To resist shearing the required size of pin assuming a 3" pin is used, area of 7.069 sq. in.,

allowable stress = 12000 lb.

$$t \times 12000 \times 7.069 = 84800t = 30,500 \text{ lb.}$$

$$t = .36\text{"}$$

and a 3" is ok.

Actual bending moment 61,000 lb. = 30,500(t 1/8")

$t = 1.96$" required, use

$t = 2.00$"

Allow 1/8" clearance .

STRESS DATA:

member	dead load	snow load max.	snow load min.	wind load left	wind load right	max. Total
A- 1	35400	31100	12450	64600	10250	100050
A- 2	25100	22050	8830	45700	7500	72800
B- 4	-78100	-68500	-27400	80800	-44000	-149500
C-6	-119200	-102650	-41800	96500	-62700	-223700
D- 8	-96400	-86400	-35800	94500	-60200	-190400
E- 9	-88700	-77800	-31100	78500	-45000	-166500
F-11	-11300	-99100	-39600	130000	-76500	-127400
G-13	-104500	-91600	-36700	169500	-97250	-238450
H-15	-70700	-62000	-24800	182300	-110800	-206300
I-17	-23200	-20400	-8160	172200	-103600	149000
J-19	27420	24050	9650	121500	-61900	148200
K-21	60200	52800	21100	47700	-5800	123000
Z-1	204500	179000	71800	12800	48000	124900
Z-3	248000	21700	87100	-26500	71000	383500
Z-5	299000	262000	104900	-43500	92000	406100
Z-7	300000	263000	105000	-45000	96700	561000
Z-10	265000	232500	93300	-69500	108000	563000
Z-12	265000	231500	93300	-109000	129500	497500
Z-14	238500	209500	84000	-138000	145000	496500
Z-16	196500	172000	68900	-145000	155500	467500
Z-18	144000	126000	50500	-131300	145200	339700

member	dead load	snow load max.	snow load min.	wind load left	wind load right	max. Total
Z-20	90500	79400	31800	-79500	103000	225300
Z-22	66500	58400	23300	- 6900	53000	142800
1- 2	-25290	-22200	-8900	- 45800	7500	-71090
2- 3	-143500	-125900	-50400	49000	-51000	-269400
3- 4	99250	87000	34800	-25800	35500	186250
4- 5	-59200	-51800	-20750	23000	-26500	-111000
5- 6	42200	51900	14888	-7600	19000	94100
6- 7	31650	27800	11100	-2700	5000	59450
7- 8	-20700	-18150	-7260	20000	-3100	-38850
8- 9	104500	91700	3670	-68000	52250	196200
9-10	16200	13310	5340	51500	-23000	67700
10-11	31650	27800	11100	-43000	32000	74750
11-12	25750	22400	9050	30000	-7500	55750
12-13	- 8450	-7420	-2960	-35500	23000	-43950
13-14	44400	38900	1555	8000	2000,	83300
14-15	-30600	-26850	-10750	-12000	16500	-57450
15-16	44800	39300	15700	-9000	10000	84100
16-17	-46500	-40700	-16300	8800	-5500	-87200
17-18	41250	36100	14450	-23000	23000	78700
18-19	50700	44500	17780	51000	-41000	101700
19-20	31700	27800	11100	-29000	26000	68800
20-21	-34850	-30600	-12210	77000	-58200	-105260
21-22	-10550	-9270	-3700	-27000	3000	-37550

member	dead load	snow load max.	snow load min.	wind load left	wind load right	max. Total
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4- 5	-59200	-51800	-20750	23000	-26500	-111000
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21-22	-10550	-9270	-3700	-27000	3000	-37550

Reactions:

Left. Wind load right.

Rn = 39000---

Rv = 26500

Dead load.

Rn = 119250

Rv = 169000

Max. snow load.

Rn = 102750

Rv = 148000

Wind load left.

Rn = 36750

Rv = 54500

Min. snow load.

Rn = 41800

Rv = 59400

Dead load plus max. snow load--

Rn = 119250 +102750 = 122000

Rv = 169000 +148000 = 317000 (max.)

Dead plus min. snow plus max.wind right

Rn = 119250 + 39000 +41800 = 200,000 (max.)

Rv = 169000 +59400 +26500 = 254900

Seating:

Allowing seven square feet per person, and skating area of $200' \times 100' =$ equal to 20,000 sq. ft.

$$\frac{20,000}{7} = 2,856 \text{ (max. seating on main floor).}$$

5000 seats to be provided in balcony.

Total seating capacity 7,856 persons.

Skating area $2(45' \times 250') + 100 \times 50 = 38,500$

$38,500 + 5000 = 43,500$ sq.ft.

Allow 8'walk around seating area, the balcony space provided $43,500 - [(2)(8 \times 250) + 100 \times 8] = 38,700$ sq.ft.