REVIEW OF THE PRATT TRUSS NOW IN PROCESS OF ERECTION ACROSS THE SANTA CRUZ.

GRADUATION THESIS.

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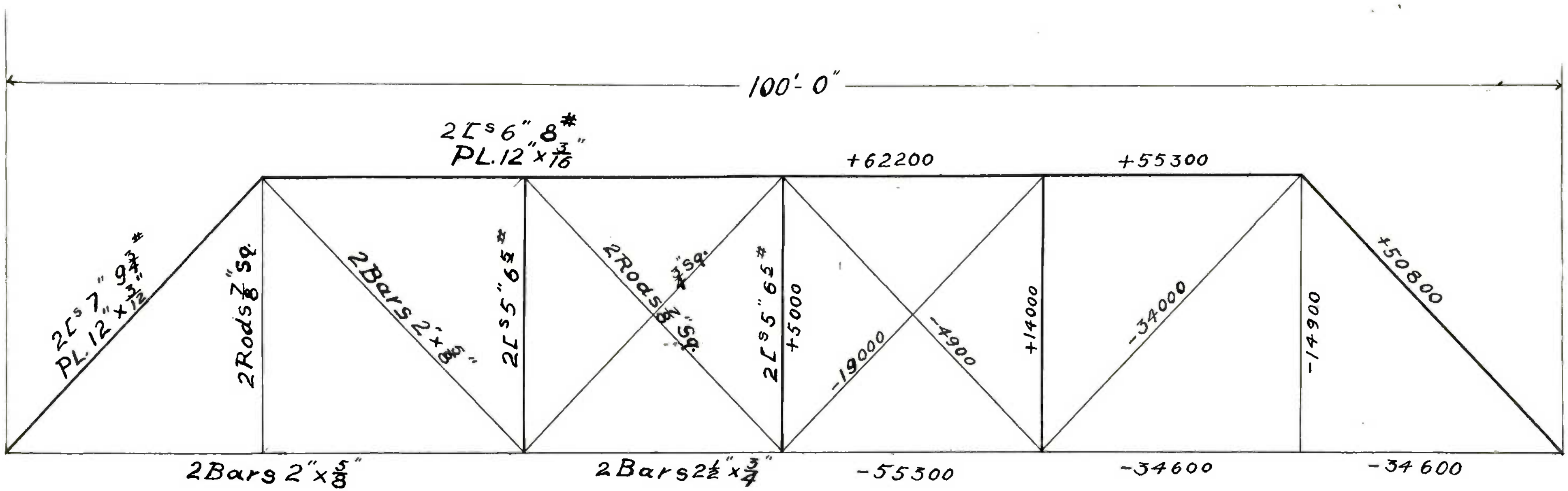
PLATE I.



Design.

Stresses.

A STUDY OF THE PRATT TRUSS NOW IN PROCESS OF ERECTION ACROSS THE SANTA CRUZ.

With what wonder would the man of a century ago, gaze upon the seemingly fragile parts of our bridge as they lie previous to erection especially were he told that these parts when placed in their proper position and connected together are calculated to hold safely 90 tons. The marvelous evolution in structural steel and the perfection of bridge designing have made this possible. The truss in question is at the present writing being built across the Santa Cruz and is situated at the foot of Congress street Tucson, Arizona.

In I844 the Pratt truss, which is the type of our span, was first introduced. It was then a "Combination Truss" built partly of wood and partly of iron. As a "Combination Truss", however, it was not a success, and soon after it began to be constructed entirely of iron, in which form it has been more extensively used than any other form of truss. Its main features are horizontal chords, vertical posts, inclined ties and for "Through Bridges" inclined end posts. An economy of material is effected first, by subjecting the verticals which are necessarily shorter than the diagonals to compression; second, by means of the inclined end posts, which may be considered simply as a continuation of the top chord and which are the means of eliminating end posts and making tension members of the next posts called the "Hip Verticals".

In design the Tucson span belongs to the lightest of the three classes into which highway bridges are divided; viz., Class C, which includes those subjected to ordinary light traffic, such as country bridges. The next class in order of strength is Class B, intended for occasional heavy loads; lastly, Class A, which includes those subjected to the continual application of heavy loads, as in city bridges. The roadway being carried on the bottom chord, our span is considered a "Through bridge" in distinction to the "Deck bridge", which supports its roadway on the top chord. Further, the joints are pin connected, a style not so generally used now as riveted joints, but formerly preferred for spans exceeding 100 feet in length. The pin connnected truss owes its popularity chiefly to the fact of its almost absolute freedom from secondary stresses, such as result from the neutral axes of different members not intersecting in a point, also when pin connected the different members have a tendency to adjust themselves to any strain which may be suddenly applied.

The object of this thesis is to review only the truss itself: to ascertain by actual computation whether every part will stand safely the tension or compression under the loading assumed for this class of bridge, and whether the greatest possible economy of material has been effected in the design of its members. The entire length of the bridge including approaches, is 160 feet. The main dimensions of the span itself are: centre to centre of end pins 100 feet: centre to centre of trusses, 15 feet: six panels 16 2/3 feet each: height, centre to centre of pins 18 feet.

The width in the clear is 14 feet, allowing two teams to pass each other, though not at full speed. Considering the average width of a team to be 5 feet it leaves only two feet between the teams when each team is one foot from the guard rail. The most economic height of a bridge is a factor of no little importance in its design. It is determined often by the clear head room desired, also that depth is sought which renders the quantity of material a minimum. This depth has been found to be from I/5 to I/6 ofthe length of the span. Our truss in depth is I6 feet or between I/5 and I/6 of its length and would therefore be considered quite economical in this respect. The span is fixed at one end and rests on roller bearings at the other to allow for the expansion and contrac-tion, due to change of temperature.

The loads to be considered are, first; the dead load which consists of the weight of the entire structure, including the floor system, railings and fastenings. This will be assumed as 40,000 pounds though the computation of the entire xxxxxx weight is a little less. Second; the live load, which will be assumed as 70 pounds per sq. ft., a weight which would be caused, it is estimated, by a crowd of people densely packed over the whole length of the bridge. To this will be added an allowance for impact. Third; wind loads, which are considered under two heads, direct and indirect or transferred wind loads. The wind pressure will be assumed as 200 pounds per lineal foot of bridge on bottom chord and I00 pounds per lineal foot on top chord. To this must be added an impact allowance as it is customary to consider the wind load as alive or moving load.

The entire dead load being 40,000 pounds, I/2 of 40,000 pounds or 20,000 pounds will be carried by each girder. Also I/2 of 20,00o or I0,000 pounds will be the abutment reaction due to dead load. The distribution of the dead load is shown in Fig. I.

In order to find the stresses in a-B & a-b pass a plane through any where between a & b and consider as free the part to the left, indicating all the forces which hold it in equilibruim as shown in Fig. 2.

Where the process is the same for finding the stresses in the different members, one process only will be shown and the results given

Fig. 1.

Dead Load 20000#.

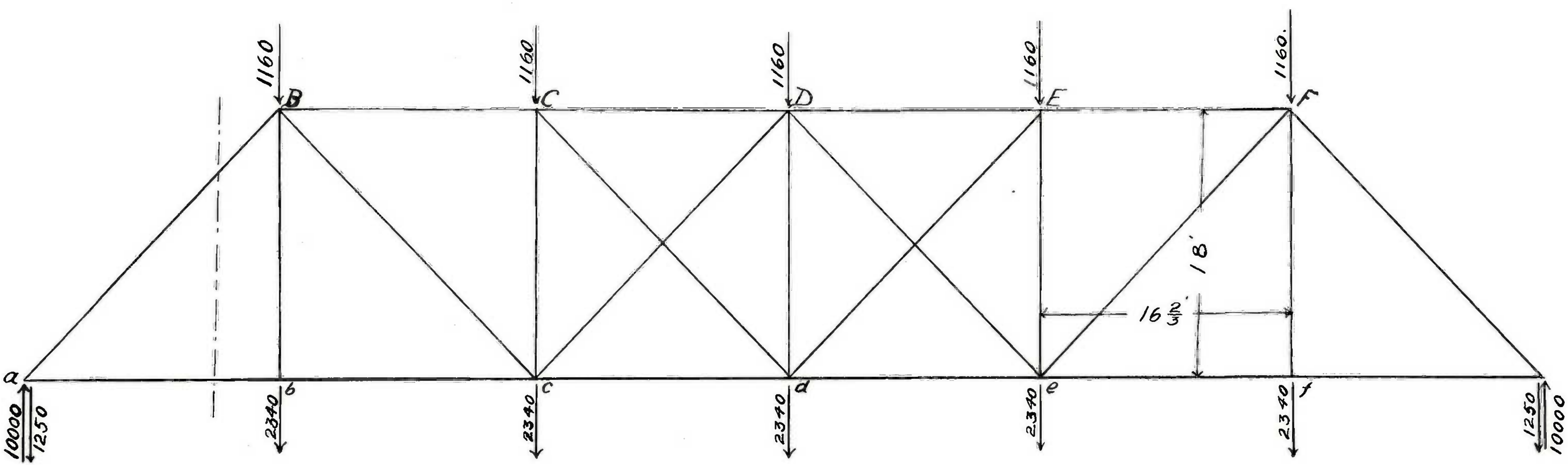
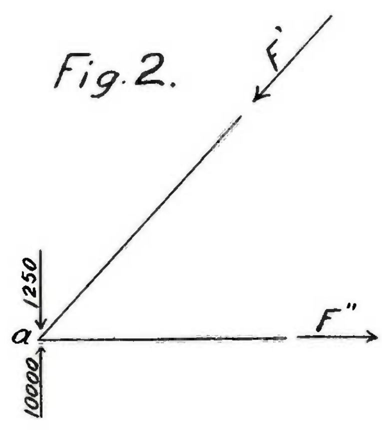


Fig. 2.



for the others. Since also the truss is symmetrical about its center, one-half only of the different members need be considered, the stresses in symmetrical pieces being the same.

COMPUTATION OF STRESSES.

STRESS in aB.

(24'.5 length of the diagonal aB, 18' depth.)

From the sum of vertical components equal to 0 (Fig. 2).

$$\frac{18}{24.5}\cdot F' + 1250 - 10000 = 0, \quad \frac{18}{24.5}\cdot F' = 8750.$$

$$F' = 11900.$$

By this method the stresses in the web members may be all computed.

STRESS in ab.

From the sum of the moments about B equal to 0 (Fig. 2).

$$18F''-8750\cdot 16\tfrac{2}{3} = 0.$$

$$18F'' = 14580.$$

$$F'' = 8100.$$

The stresses in the members composing the top and lower chords are all computed by the last method.

Table of Stresses Due to Dead Load

Member	Stress
ab	-8100
bc	-8100
cd	-12950
Bc	-7150
Cd	-2400
Bb	-2350
Cc	+1750
Dd	+1150
aB	+11900
BC	+12950
CD	+14600

STRESSES DUE TO UNIFORM LIVE LOAD.

The live load, as specified in a preceding paragraph 70 pounds per square foot, will first be distributed uniformally over the bridge and the resulting stresses obtained, by the methods already shown. The batter posts and top and bottom chords are strained to their maximum by the above loading, therefore the stresses in these members alone will be given. The web members are under maximum stress when the bridge is unsymmetrically loaded, a condition which will be considered later.

Table of Uniform Live Load Stresses.

Members	Stress	Impact Allowance	Total Stress
aB	27787	IIII5	+38900
BC	30244	I2098	+42350
CD	34025	I3609	+47650
ab	I8903	756I	-26450
bc	I8903	756I	-26450
cd	30244	I2098	-42350

The allowance for impact has been figured from Waddell's formula for highway bridges: viz.,

$$I = 10000/(L + 150)$$

in which I is the percentage for impact to be added to the live load and L the lenght in feet of the span or portion of span that is covered by the said load. L in the above equals I00

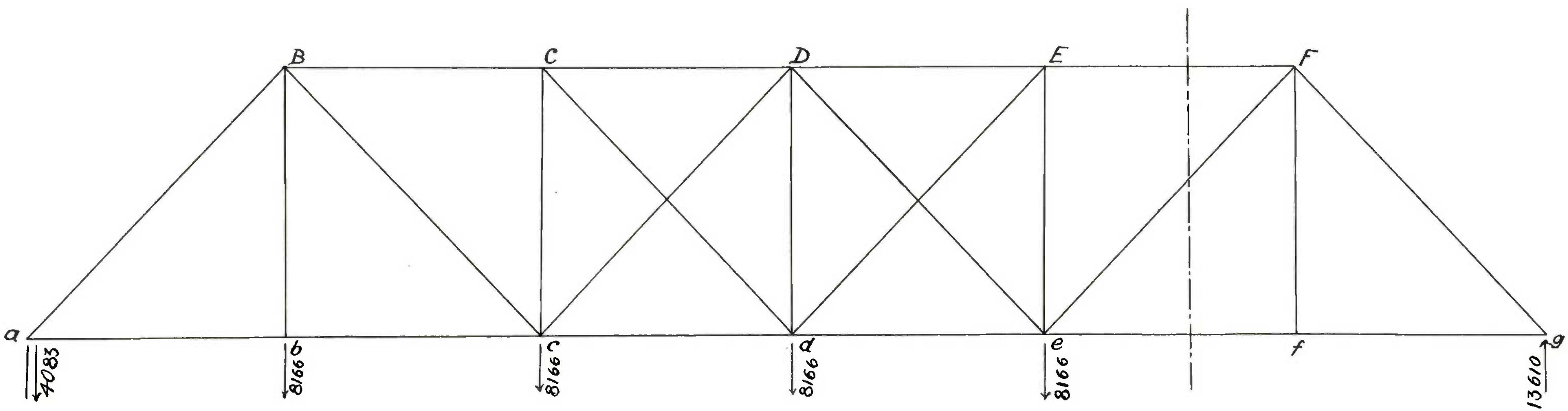
Therefore, $I = 10000/(100 + 150) = 40\%$.

Stresses due to unsymmetrical live load:--

The same live load 70 pounds per square foot or 8I66 pounds panel live load, will now be considered to extend all the way along the bridge to e, as shown on figure 3, and the stresses computed in the diagonal eF,

Fig. 3.

Loaded to e.



by passing a plane through anywhere between e and f and considering as free the part to the right of the cutting plane. The load will then be moved back successively to d,c and b and the stresses computed in the diagonal immediately to the right of the loading. A new abutment reaction will have to be obtained for each successive change of loading, and the right hand portion considered free as before.

When the live load is at c or extends from a to c then the counterbrace cD is brought into action, likewise when the load is at e the counterbrace eD is strained. Under the computation for the deadload and a uniform live load the counterbraces were not considered, since they are not brought into use.

Unsymmetrical Live Load Stresses.

Member.	Stress.	Position of load for max. stress.	Per-cent impact.	Impact.	Total Stress.
eF - Bc	-I8500	Loaded to e.	45	-8350	-26850
dE - Cd	-III00	.. .. d.	50	-5550	-I6650
cD - De	-3I50	.. .. c.	54	-I725	-4900
Bb - Ff	-8I50	.. .. c.	54	-4400	-I2550
Ee - Cc	+8I50	... .. d.	50	+4000	+I2250
Dd	+2333	.. .. c.	54	+I250	+3600

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WIND LOAD STRESSES.

Direct Wind Load.

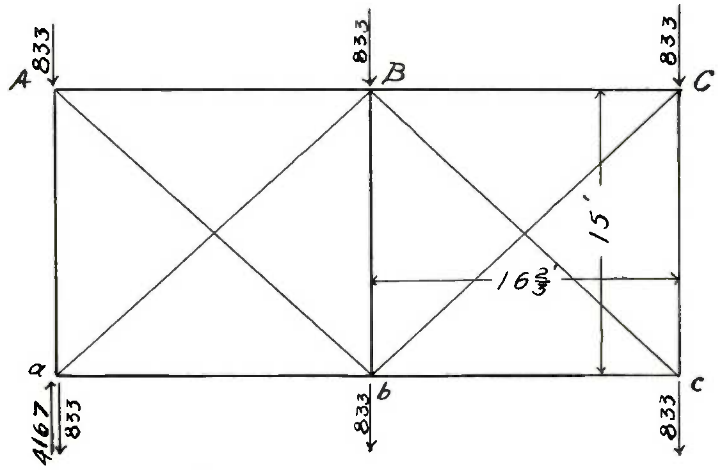
Two systems of lateral bracing,upper and lower are designed to withstand the pressure due to wind. We will first consider the top chord laterals. As already specified the wind pressure upon the top chord will be assumed as 100 lbs. per lineal foot of bridge giving a panel load of 1666 lbs.; also this will be regarded as a live load and the stresses computed for both uniform and unsymmetrical loading to obtain the maximum stresses in the different members. The distribution of the loading and principal dimensions are shown in Fig. 4.

(The length of the diagonal between panel points is 22.4 ft.)

Wind Load Stresses Upper Chord Laterals.

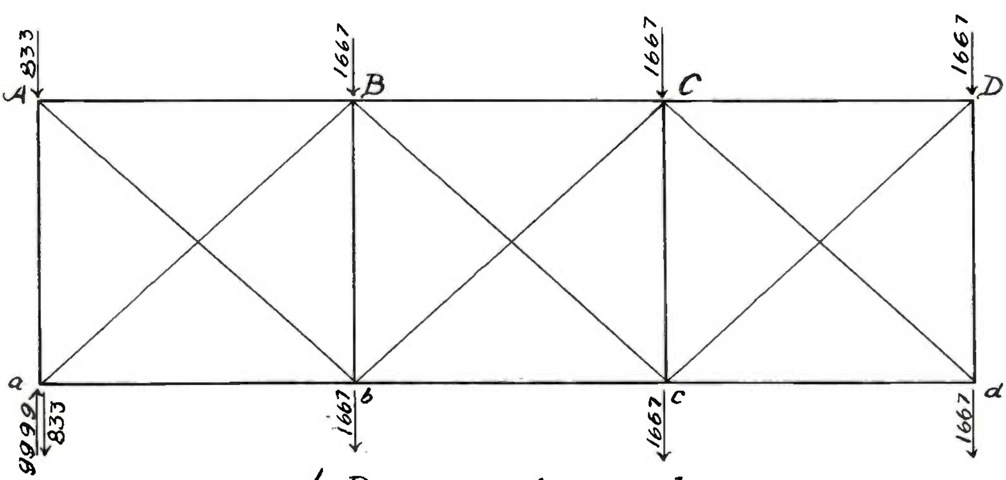
Member.	Position of load for max. stress.	Maximum stress.
Aa	Loaded throughout.	+3350.
Bb	" "	+2500.
Cc	" "	+1650.
AB	" "	+2800.
BC	" "	+3700.
Ab	" from A to D	-3050.
BE	" " A to C	-1850.

Fig. 4.



½ Top Laterals.

Fig. 5.



½ Bottom Laterals.

Wind Load Stresses, Bottom Chord Laterals.

Member.	Position of load for max. stress.	Maximum Stress.
aA	Loaded throughout.	+10000.
bB	" "	+8350.
cC	" "	+5000.
dD	" "	+3350.
bc	" "	-9250.
cd	" "	-14800.
Ab	" "	+12450.
Bc	" from g to c.	-8300.
Cd	" " g to d.	-6650.

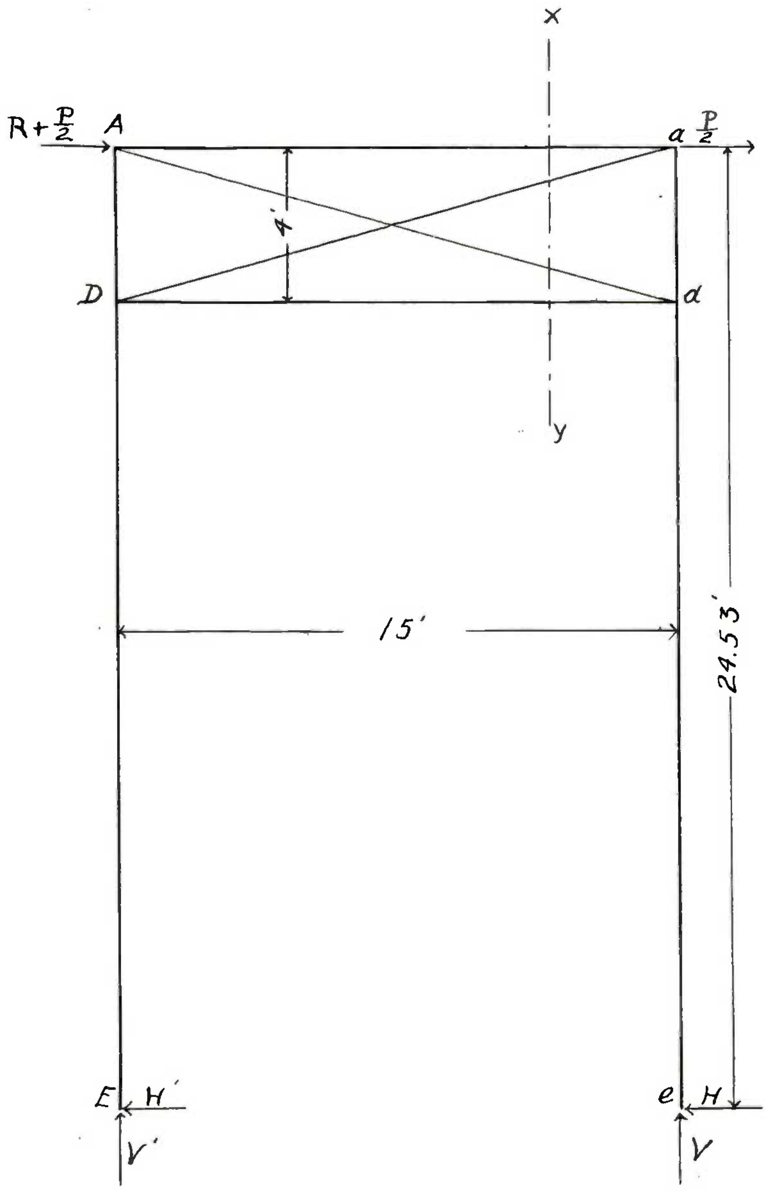
The wind load will not be considered as acting simultaneously with the live load, therefore the resulting chord stresses as given in the above table will be combined with those due to the dead load.

Indirect Wind Load.

There is also a uniform compressive stress in the windward lower chord induced by the action of the portal bracing which when combined with the two above stresses, deadload and direct windload, often results in a reversal of stress. This will now be considered.

The form of the portal bracing and the forces acting upon it are shown in Fig. 6. One half the entire load upon the intermediate panels is transferred to the point A, call this load R. In addition one half a full panel load is applied at A and one half at a, call each $\frac{P}{2}$. These forces must be held in equilibrium by external forces applied at E-e, whose horizontal and vertical components H, H' and V, V' respectively are shown.

Fig 6.



Portal Bracing.

These forces form two couples (R+ P) and 2H, with an arm of 24.53 , and V and V' whose arm is I5, which are in equilibrium and therefore have equal moments.

$$R = 2500, \quad R + \frac{P}{2} = 3333. \quad R+P = 4I66.$$

From the sum of Horizontal Components = 0.

$$R+P = H+H' \text{ or } 4I66 = 2H.$$

Moment of Horizontal Couple = (R+P)24.5

" " Vertical " = (V)I5.

$$V = \frac{4I66 \times 24.53}{I5} = 6800.$$

Passing the section XY, taking the center of moments about D:- (The piece Ad is not in action with the wind from the left.)

Compressive Stress in Aa:

$$\frac{-(R+\frac{P}{2})4 - (H')20.53}{5} = 11220.$$

From the sum of the Horizontal Components = 0:

Stress in diagonal Da :

$$V \times \frac{I5.8}{4} = 28I60.$$

From some of the moments about a = 0:-

Compressive stress in Dd:

$$\frac{H' \times 24.53 - V' \times I5}{4} = I2775.$$

From the sum of the Vertical Components = 0:-

$$68I3 \times \frac{I6}{2453} = 4629.$$

Therefore the forces in the bottom chords induced by the indirect wind load are 4629 compression on the windward side and an equal amount of tension in the leeward side.

Returning again to Fig.5 in order to find the compression induced in the different parts of the windward chord AD, by the usual moment equation, we obtain:-

Compression in AB ---------------- 9260.

" " BC ---------------- I4380.

" " CD ---------------- I2950.

Resulting Compressions Due to Direct

and Indirect Wind Loads.

Member.	AB	BC	CD
Indirect.	4630.	4630.	4630.
Direct.	9260.	I4800.	I2960.
Total Compression.	I3900.	I9450.	I7600.
Tension due to dead load.	8I00.	8I00.	I2960.
Reversal resulting.	5800.	II350.	4650.

As seen by the above table a reversal(compression) occurs in the lower chord members, which being made up of simply Eye-bars are not designed to withstand compression. The string - ers cannot be relied upon to withstand buckling therefore the lower chord should either be counterbraced or built itself capable of with- standing both tension and compression.

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DESIGN OF MEMBERS.

Floor Beams.

The length, center to center of the end supports of the floor beams, is 15', or the distance, center to center, of trusses. The specifications call for an I-Beam, the size of which must be arrived at by trial. We will assume a 15" I-Beam.

Weight of floor planking - - - - - - - -	13300#
" " stringers - - - - - - - -	7600#
" per 100' of bridge,	20900.
" per panel,	3480.
Trial weight of floor beam,	672.
Panel dead load,	4150.
Panel live load,	25150. (plus impact)
Total weight on floor beam,	29300.

From the formula $\frac{Wl}{8}$ = Max. bending moment we have, 659400.

From the formula $\frac{R'I}{e}$ M, in which R' is the safe stress in the outermost fibre, assumed for our computation 14000 lbs, per square inch, $\frac{I}{e}$ is the (Section Modulus.) and M is the maximum bending moment.

Substituting the values in the above formula:

$\frac{R'I}{e} = 14000 \times 58.9 = 824600$ (safe bending moment)

The maximum bending moment as figured previously is only 659400, which is much less than the allowable bending moment 824600. Therefor a 12" I-Beam which is the next size smaller was now tried but this gave as the safe bending moment 503533, which is too low. Hence a 15" I-Beam weighing 42 lbs. to the foot is the proper size to use.

Stringers.

Panel dead load - - - - - - - - - - 3480.

Panel live load plus impact. - - - - 25I50.

6)28635.

Total load on each joist, - - - - - 4770.

Maximum bending moment $= \frac{Wl}{8}$ - - - - - -II9300.

$\frac{R'I}{\frac{d}{2}}$ - ---- -- - - M - - - - - -II9300.

(Timber)

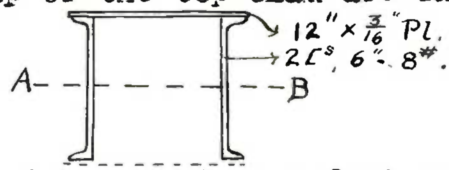
R' assumed as 2000lbs., per sq. in. and $\frac{I}{e} = \frac{bd}{6}$. Assume b as three inches and solve for d. d = II".

Therefore a 3" x I2" stringer (joist) will be used.

DESIGN OF TRUSS MEMBERS.

Top chord.

The dimensions and make up of the top chord are shown in the accompanying figure.



Two 6" channels weighing eight pounds per foot and one cover plate I2" x 3/I6".

For the allowable compression in the top chord the following formula as given by Waddell will be used:

$$18{,}000-(70\ l/r),$$

in which l is the unsupported length of the bridge and r the radius gyration.

To find the radius of gyration:-

$$r = \sqrt{I/A}\ .$$

(I equals the moment of inertia and A equals the sectional area)

Static Moments.

For P/late , 2 I/4 x 0 = 0.

For Channels, 2. 38 x 2 x 3 = I4. 28 .

Area = 2.25plus4.76 = 7.00 .

$\frac{14.28}{7}$ = 2.04 (Distance from back to neutral axis)

I for channels, I3.o x 2 = 26.0

4. 76 x $(.96)^2$, = 4.39

I plate , = 0.00

2/25 x $(2.04)^2$ = 9.36

Total I = 39.75

I/A = 39.75

$\sqrt{I/A}$ = 2.4 = Radius of gyration.

Substituting this value for the r in the formula:

$18{,}000 - \frac{70 \cdot 16\ 2/3}{2.4} = $ I2168 —— (allowable compression on top chord per sq. inch)

Greatcst compression in top chord - - 62200.

62200 ÷ 12168 = 5.1 sq. in. (Seccional area req.)

We have 7 sq. in. according to specifications.

BATTER POSTS.

As shown in Plate 1, the batter posts are made up of two 7 in. channels, weighing 9 3/4 lb. to the foot. In this computation, the formula used is;--

$$18000 - 80 \times \frac{L}{r}$$

The sectional area required is 5.9 sq. in., as against 7.95 sq. in. which the specifications call for.

TABULATION OF THE STRESSES AND SECTIONAL AREAS OF THE MEMBERS OF THE TRUSS.

Member	Stresses	Radius of gyration	Unit Stress	Area req'rd sq. in.	Area used
ab	-34600		18000	1.9	2.5
bc	-34600		18000	1.9	2.5
cd	-55300		18000	3.1	3.75
Bc	-34000		16000	2.1	2.5
Bd	-19000		13000	1.5	1.75
De	-4900		13000	.4	.75
Bb	-14900		13000	1.1	1.75
Cc	+14000	1.95	9350	1.5	3.9
Dd	+5000	1.95	9350	.38	3.9
aB	+50800	2.5	8580	5.9	7.95
BC	+55300	2.4	12150	5.1	7.0
CD	+62200	2.4	12150	4.5	7.0