Thesis.

An Investigation of the Strength of Yellow Pine Ties for Railroad Bridges.

May 1910

An Investigation of the Strength of Yellow Pine Ties for Railroad Bridges.

The object of this thesis was a study of the use of yellow pine ties on railroad bridges as determined by modern railroad practice. The investigation was divided into three parts:-

"A". The determination of the dimensions of ties in relation to the varying types of bridge construction.

"B". A study of the methods of preserving ties and the influence of the preservative processes on the strength of the timber.

"C". A series of tests to determine the resistance to longitudinal shear due to the conditions of loading as found in actual practice.

"A."

The following tables were prepared from data obtained from the latest bridge specifications of the principal railroads of the United States, Canada and Great Britain. The railroads included in these tables represent only a part of those whose specifications we examined. In many cases, especially the railroads of Great Britain, no details were given concerning the size and quality of the ties to be used on bridges.

Table I gives the dimensions of the ties, the spacing in clear, and the kind of timber used by many of the American railroads at the present time.

Table II gives the distances center to center of plate girders on deck bridges for varying spans, and for through bridges the distance center to center of stringers.

Table I.

Railroad	Dimensions of Ties	Spacing in clear	Kind of Timber
At. Top. & Santa Fé	8"x8"x12'	4" to 6"	L.L.Y.P.
Balt. & Ohio	8"x8"x9'	6"	L.L.Y.P.
Chicago & Alton	8"x8" —	4"	—
L.S. & M.S.	8"x8"x10'	4"	—
Lehigh Valley	—	6"	Y.P.
N.Y., N.H., & Hart.	— 10'	$\leqq$ 6"	Chestnut
Penn.	—	$\leqq$ 6"	White Oak
Queen & Crescent	8"x8"x9'-6"	$\leqq$ 6"	L.L.Y.P.
C.R.R. of N.J.	8"x10"x12'	4"	—
Seaboard Air Line	(8"-12")x(8"-12")	6"	L.L.Y.P.
Atlantic Coast Line	8"x9"x10'	5"	L.L.Y.P.
Oregon Short Line	Width $\geqq$ 8"	4"	—
Mo. Pacific	7"x8"x9'	6"	—
Nor. & Western	8"x10"x($\geqq$12')	—	White Oak
Can. Pacific	Width = 8"	4"	Pine
Texas & Pacific	8"x10"	6"	—
Wabash	8"x8"x10'	6"	—
Del. & Hudson	9"x9"x9'	—	—
Phil & Reading	—	—	L.L.Y.P.
Kan. City, Mex. & Orient	—	—	South. Y. Pine
Pencoid Iron Works	8"x10"	6"	—

Table II

Railroad	Plate Girder Br. ℄-℄ of G	Span	Through Br. ℄-℄ Stringers	Remarks
Penn.	6'-6"		6'-6"	
Ph. & Rd.	6'-6"		6'-6"	
Q & Crescent	$\geq$ 6'-0"		$\geq$ 6'-0"	Thicknes & length of ties proportional to spacing girders Web of girder 1" in cross ties.
St.L & San.	7'-0" 8'-0"	$\leq$ 60' 60-100'	7'-0"	
C.RR. of N.J	6'-6"		6'-6"	
Atl. Coast Line	6'-6"		6'-6"-7'0"	
M.S.P. & S.St.M	8'-0"		8'-0"	
D. & R.G.	7'-0" (7'-0")+$\frac{1}{12}$Span	$\leq$ 80' $>$ 80'	6'-6"	
Oregon Short L.	7'-0" 8'-0" 9'-0"	$\leq$ 60' 80' 100'	7'-0"	Depth of ties $\geq$ 7½" @ Center $\geq$ 9½" " End
N & West.	6'-6"		6'-6"	
K.C. M & Orient	7'-0" 8'-0"	$\leq$ 80' $>$ 80'	7'-0"	
Wabash	6'-6"		6'-6"	
C.C.C. & St.L.	6'-6"		6'-6"	
C. & E. Ill.	7'-0" 8'-0"	$\leq$ 60' $\leq$ 110'		
Pencoid Iron	6'-6"		8'-0"	

Table II Continued

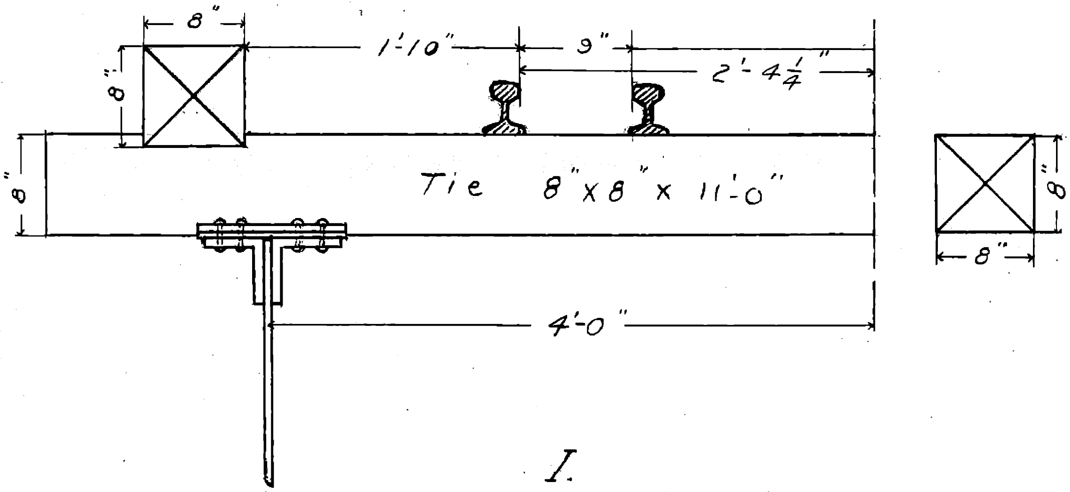
Railroad	Plate Girder Br.		Through Br.	Remarks
	℄-℄ of Girder	Span	℄-℄ Stringers	
Seaboard Air Line			6'-6" 7'-0" 7'-6" 8'-0" 9'-0" 10'-0"	
B & O	6'-6" 8'-0"	$\leq$ 80' $\geq$ 80'	6'-6"	
C & Alton	7'-0" 8'-0" 9'-0"	$\leq$ 60' 60-80' 80-100'	7'-0"	Depth ties $\geq$ 7½" @ Center $\geq$ 9½" " end. Width $\geq$ 8"
D. & H.	7'-0" 8'-0" 1/12 Span	$\leq$ 60' 60-100' > 100'		
D.L. & W.	7'-0" 8'-0" 1/12 Span	$\leq$ 60' 60-100' > 100'	6'-6"	
Erie	7'-0"		5'-0"	
I. Central	7'-0" 8'-0" 9'-0"	$\leq$ 60' 60-80' 80-100'	4 Stringers 2'-6" Sps.	
L.S. & M.S.	6'-6" (6'-6")-(8'-0")	$\leq$ 70' 70-100	6'-6"	Depth of tie increased 1" for each 6" above 6'-6" Spacing

In the following discussion we have dealt with steel railroad bridges for single track and of open floor construction, as these give the conditions of loading for which the tests of Part "C" have been made.

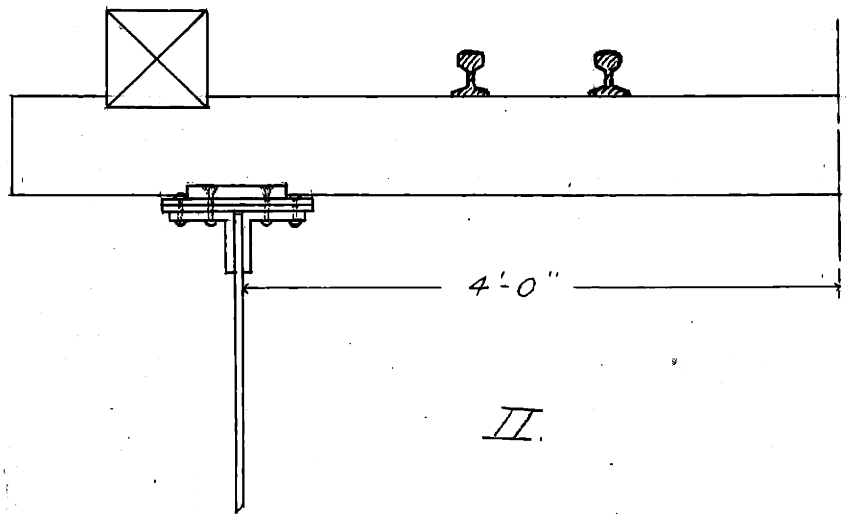
Open floor steel railroad bridges are of two general types, 1st deck plate girder bridges in which the the ties rest directly on the top flanges of the girders; and 2nd through bridges in which the ties are supported by the stringers, and the load taken to the main girders or trusses by means of the floor beams.

Of course there are various other types of construction in use at the present time, but the two types just mentioned are used so commonly that we have confined our attention to them entirely.

The following sketches represent some of the methods used at the present time to secure the ties to the girders or stringers as the case may be.



I.



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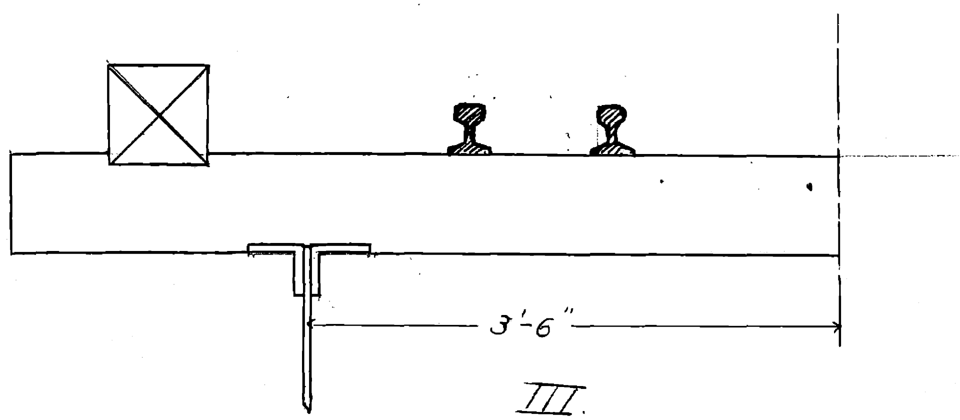

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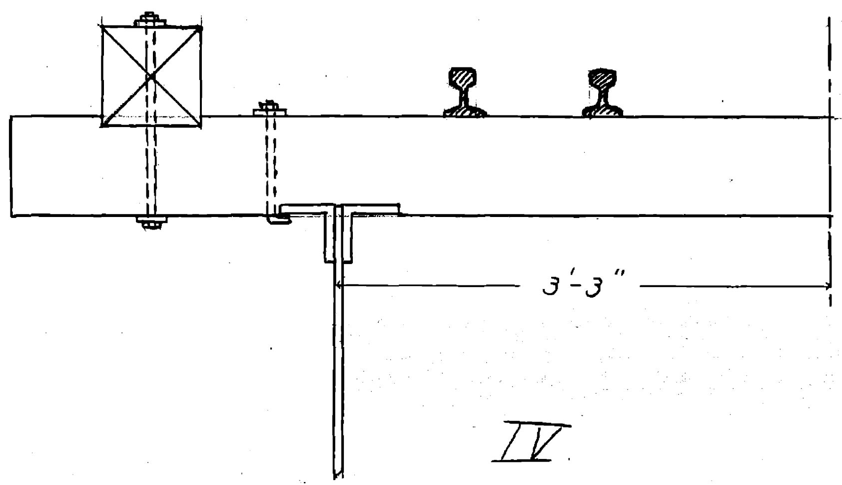

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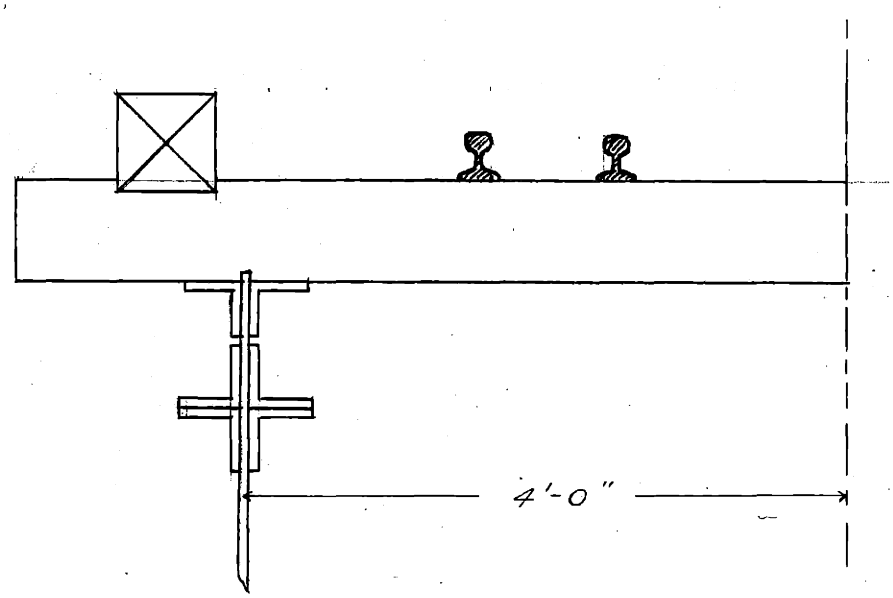


V.

For purposes of comparison the ties have all been taken 8"x8"x11'-0", the guard timber 8"x8", and the spacing of guard timber and guard rail as shown. The supporting girders or stringers are shown in different forms, at varying distances apart, and with several methods of securing the tie.

In all of these cases the guard timber has been notched 1" over each tie as this appears to be the recognized means of securing the ties against "bunching" in case of derailment. In a few cases however the notch may vary in depth from $\frac{3}{4}$" to 2".

Case I shows the tie notched 1" over the top flange of the girder. This condition would necessitate additional grooves being made in the tie to fit over the rivet heads to insure the tie coming to an even bearing on the girder itself. In some cases the rivets are countersunk throughout the top flange to obviate the rivet grooves in the tie.

Case II resembles the first case except that an additional plate about 8" wide has been added throughout the length of the girder and the rivets through the plate are countersunk. This top plate is kept perfectly level by means of fillers which are added wherever the cover plates are cut.

Case III represents the ordinary condition on through bridges in which the tie is simply notched 1" over the top flange of the stringer.

Case IV shows the use of the common "hook" bolt used in many cases as an additional security against the "bunching" of the ties in case of an unusual longitudinal thrust such as may be caused by the derailment of a truck or a whole car. This "hook" bolt when used is generally placed in every fourth tie, although in some cases it has been placed in every third tie. The guard timber is generally bolted as shown at every fourth tie. A modification of this form of fastening, not shown here, may be adapted where the guard timber is more nearly over the stringer or girder. In this case a long "hook" bolt is used through the guard timber and tie, and is directly fastened under the flange of the supporting stringer or girder.

Case V is a rather recent development of the foregoing cases. It is used regularly on the Chicago, Milwaukee and St. Paul R.R. at the present time. In this case the web of the girder projects $\frac{1}{2}$" to 1" above the angles forming the top flange. The tie is then notched over the web so that it may come to an even bearing on the flange. With such an arrangement the shape of the top flange is necessarily different from the general types in order that enough flange area may be procured without the use of horizontal cover plates. A simple form of flange is shown in the sketch, but in actual practice it may be effectively strengthened by the use of vertical side plates and additional angles.

The preceding figures are not intended to cover all cases arising under the ordinary conditions but are merely illustrative of the general types used by modern railroads.

"B."

The question of tie renewal is too often considered an unimportant item in the expense account of the maintenance of way department, and as a consequence the expenses are continually increasing. A good deal of the general expense may be charged to the deterioration of track due to the softening of ties from incipient decay. The renewal of ties may be considerably reduced by the use of preservative materials.

The information obtained on this subject was taken in part from a circular published by the U.S. Department of Agriculture under the Forest Service entitled "Experiments on the Strength of Treated Timber" by W. Kendrick Hatt, Ph.D., Civil Engineer.

In outlining the plan for these tests, two divisions were made, dealing respectively with the effect on the strength of timber of the preliminary processes of steaming, superheating and vacuum, commonly employed in the preservation of wood; and the effect of the preserving materials themselves. The effect of these processes was determined on both green and on seasoned timber, the preservative fluids including only creosote and zinc chlorid.

The series of experiments relating to the effect of steaming were carried on with small specimens of both green and seasoned loblolly pine. With the green timber, which was tested immediately after treatment with steam, results showed that the strength of the wood was decreased as the temperature and pressure were increased.

Wood steamed and then treated with zinc chlorid was weaker under this test than natural timber, but was not as weak as the wood that was merely steamed. However, under impact loading, there was a noticeable deficiency of strength in this Burnettized wood.

Results from the tests on the creosoted pieces showed that the creosote did not directly weaken the wood. The steamed and creosoted wood was weaker than the natural wood, but was not as weak as the steamed wood. A microscopic examination of the treated wood showed that the creosote probably does not enter the substance of the cell walls, but merely fills the openings of the cells and paints the surfaces. It therefore should not directly weaken the wood.

Creosoted wood, however, dries out more slowly than untreated wood, because the cells and passage-ways are full of creosote. Thus, creosoted timber will season very slowly; and if two pieces of wood having essentially the same structure - one green and natural and the other green and creosoted - were allowed to season, the untreated specimen would develop increased strength due to the seasoning, while the creosoted piece would not gain in strength nearly as rapidly. This result was shown in the experiments carried on by Mr. Hatt for the Forest Service. As large timber rapidly seasoned often develops weaknesses due to checks and shakes, the creosoting process could be used to great advantage in preventing such defects in the timber.

The tests on the seasoned timber were carried on after the natural, the steamed and the Burnettized wood has reached the same (about 13) per cent of moisture; while the creosated wood had surely not reached that degree of dryness. Results from these experiments showed that the natural and the steamed wood reached the same strength after having been dried. However, results showed that there was a permanent weakening of the full sized ties that had been steamed and then allowed to season. Under static tests the specimens that had been steamed and creasoted and then air dried did not reach the strength of pieces that had been steamed only and then air dried. This was probably due to the fact that the cell walls had not completely dried out.

Under impact tests the wood showed greater shock resistances at rupture.

Pieces that had been steamed and treated with zinc chlorid and then air dried were not weaker under static loading than the pieces which had been merely steamed and then air dried. Under impact tests, however, the Burnettized wood showed a deficiency in strength very nearly proportional to the amount of zinc chlorid that had been forced in.

In addition to the tests on small pieces, the strength of full sized ties was obtained in bending and in com-pression, both parallel and at right angles to the grain. The entire tie was treated and afterwards tested full size. In the bending tests under a static load, the tie was 8' long and supported on a span of 80" and loaded at the third points of the span. In the tests of compression at right angles to the grain the width of the tool equaled that of the base of an 80-pound rail of the American Society of Civil Engineers standard, that is 5".

The ties tested were average bridge ties of large inherent variability. The New Mexico pine was of straighter grain and contained fewer knots than the Arizona pine, which was of coarse growth and crooked grained and knotty. The results of these tests are summarized in Table III.

Table III

Effect of preservative treatments on the strength of western yellow pine.
[Tested full size; air dried before testing]

Treatment	Ties From	Cylinder Conditions: Steaming: Period	Cylinder Conditions: Steaming: Pressure	Cylinder Conditions: Steaming: Temperature	Absorption of preservative	Strength: Static: Bending modulus of rupture	Strength: Static: Compression parallel to grain	Strength: Impact: Bending Ht. of drop causing failure
		Hours	Lbs. per sq. in.	°F	Lbs. per cu. ft.	Per cent	Per cent	Per cent.
						Untreated wood = 100 per cent.		
Zinc chlorid	Arizona	3	20	258	16.05	138.3	104.3	107.1
					14.37	120.9	124.2	214.2
	New Mexico	3	20	258	13.84	79.5	126.2	156.0
					19.77	61.8	94.6	104.0
Creosote	Arizona				11.34	134.0	104.9	164.3
		3	20	257	17.54	130.6	81.9	100.0
				220	12.27	142.9	93.5	107.1
				275	12.78	133.9	97.4	135.8
	New Mexico				10.34	71.5	64.9	104.0
		3	20	257	10.61	71.7	69.3	80.0
				220	10.21	85.0	83.5	108.0
				275	10.66	59.0	87.9	92.8

Arerage Strengths of the Untreated Wood.

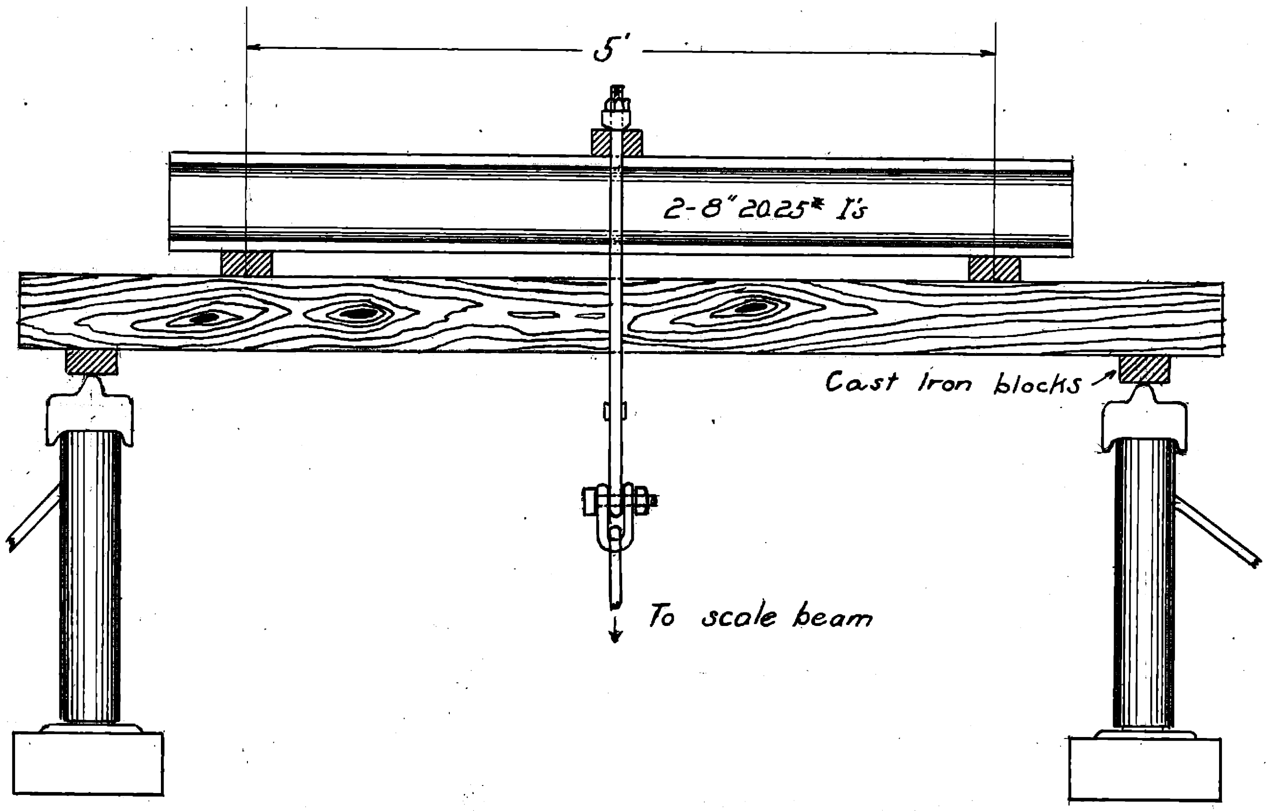
Static Loading		Static Loading: Arizona	Static Loading: N. Mexico	Impact Loading: Arizona	Impact Loading: N. Mexico
Modulus of elasticity	lbs. per sq in	818 000	1,170000	807 000	813 000
Bending strength at elastic limit	"	2 280	3438	4296	5 255
Bending strength at rupture	"	3 020	5,595		
Compress. strength parallel to grain	"	3 720	4 343		

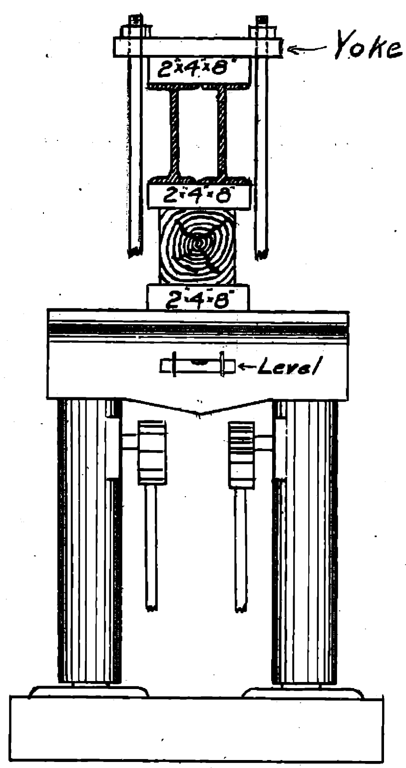
"C."

The third and principal part of this thesis was a series of tests to determine the resistance of the tie to longitudinal shear under the conditions of loading found in actual practice.

The resistance to longitudinal shear was found to be increased due to two principal causes. We found that the over-hang of the tie beyond the support caused a considerable additional resistance to shear. We also were able to show a very strong reinforcing action due to that part of the beam between the loads i.e. where the shear was equal to 0.

The following sketch illustrates our method of testing full sized specimens, and with its accompanying description will be readily understood.


5'
2-8" 20.25# I's
Cast Iron blocks
To scale beam


Yoke
2"x4"x8"
2"·4"·8"
2"·4"·8"
Level

The apparatus used for testing the full sized specimens was the 100,000 lb. Beam Machine located in the Massachusetts Institute of Technology, in the lower Applied Mechanics Laboratory. From an inspection of Table I., it is evident that present practice would indicate a tie, having a cross-section of 8" X 8". Owing however to the fact that the jacks on the machine were not up to standard, we were obliged to confine our tests to timber of 6" X 6" cross-section, in order not to use the jacks beyond their actual efficiency, which was probably not over seventy per cent.

Since our idea was to subject the tie to, as nearly as possible, the same loading as would be brought upon it on a bridge, we applied our loads symmetrically and at a distance of five feet center to center, which is approximately the distance apart of the rails. We supported the tie on small blocks over the jacks at span lengths from 6'-6" to 9'-0", representing the distance apart of stringers or girders. The blocks which we used between the I beams and the tie gave a bearing area on the tie of 24 square inches for each block. This, although somewhat less than the bearing area under a rail, gives a very fair approximation. We used the same size blocks above the jacks in order to keep the same crushing intensity over the supports as we had under the loads. Owing to the fact that the blocks were of cast iron, we wired them to the jacks in such a manner that in case of an accident no part of the block could fly out and do any damage. We also took the precaution of lashing the jacks firmly to the vertical standards of the machine. In order to load the tie as symmetrically as possible, we suspended a plumb line from the ceiling over the exact centre of the apparatus and used this plumb line as a guide throughout our experiments. The power was applied by hand levers to the jacks, and the total load measured at the scale beam which was attached to the central yoke. On account of the necessity of applying the load by hand, and also owing to the more or less complicated method of placing each tie exactly centered and level in the machine, more time was required than might ordinarily have been thought necessary at the outset.

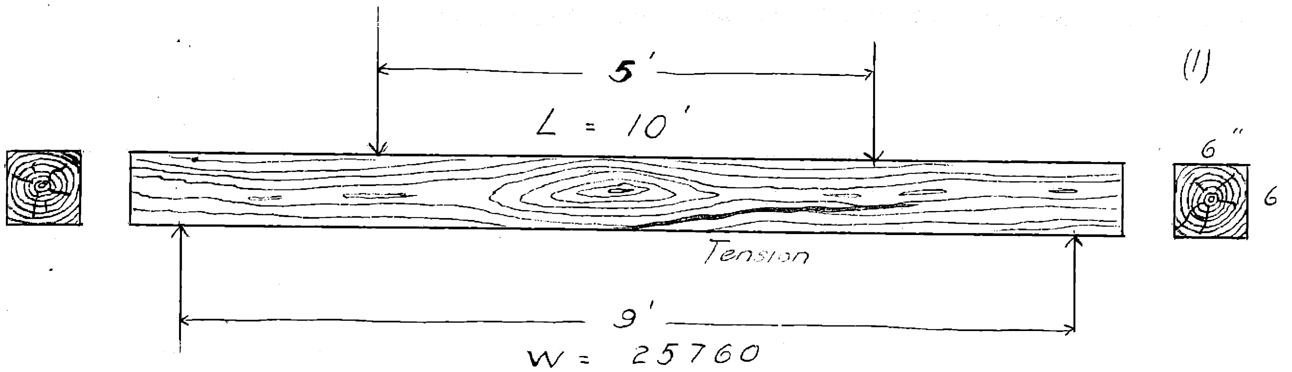
At the outset we attempted to determine at the same time both the resistances to longitudinal shear caused first by the overhang, and second by the reinforcing action of the portion of the beam with zero shear. After several preliminary tests we found however that it would be impossible to determine both these effects independently on full sized specimens. Our original idea was to test sets of beams without an appreciable overhang, then with the same span and conditions, test other beams with varying overhangs, thus finding, if possible, the point at which the increased resistance to shear, due to the overhang, was great enough to cause the beam to fail in bending rather than by shear.

With ideal conditions of lumber, the above scheme would no doubt have given fairly definite results. The timber, which we used, was about one thousand feet of 6"x6" long leaf yellow pine of inherent variability. The lumber as it came from the wharf was for the most part very wet and in many cases cross grained and full of knots. Thus after making several different tests, we were convinced that we could not secure the definite results in regard to the two kinds of reinforcement at the same time. The principal fact that deterred us from continuing the majority of our tests in this direction was our frequent failures by tension which made it almost impossible for us to get actual relations between the beams with, and those without the overhang.

We therefore tested the majority of our full sized beams with no appreciable overhang, endeavoring merely to get the resistance to longitudinal shear due to the reinforcing action of that part of the beam between the applied loads, since this seemed to be the more important factor of the resistance to shear.

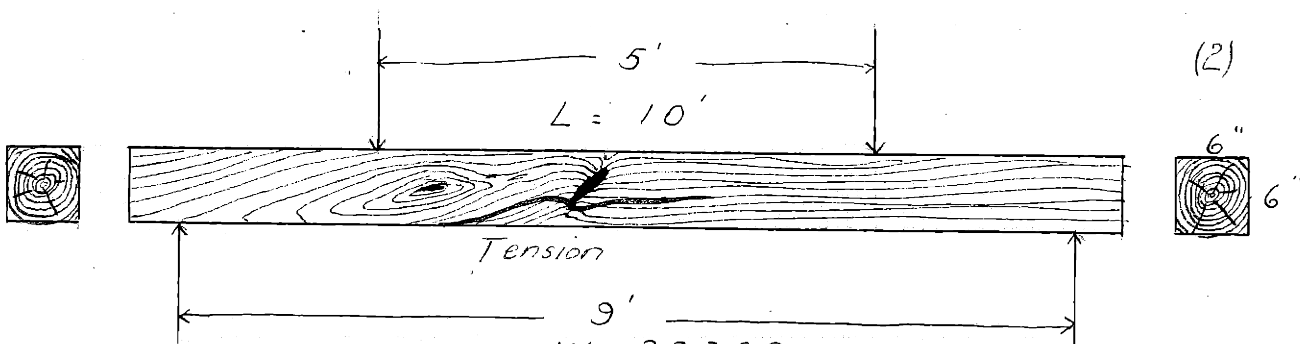
We also made special tests on short specimens in the Olsen Machine to determine the resistance due to the overhang of the beam beyond the supports.

The following sketches show the failures of our full sized specimens. In these thirty five drawings we have attempted to show the physical characteristics of the wood to a reasonable degree of accuracy, as well as the exact methods of failure.



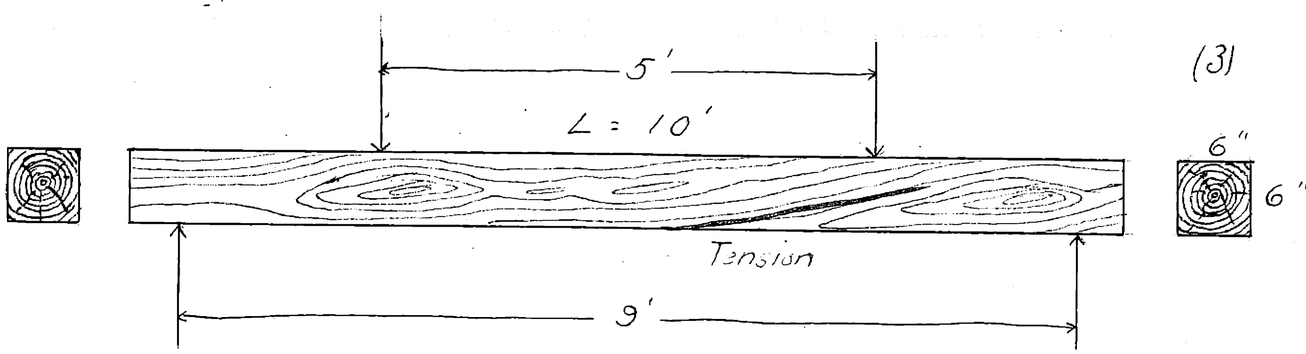
W = 25760

f = 8590 s = 537



W = 29300

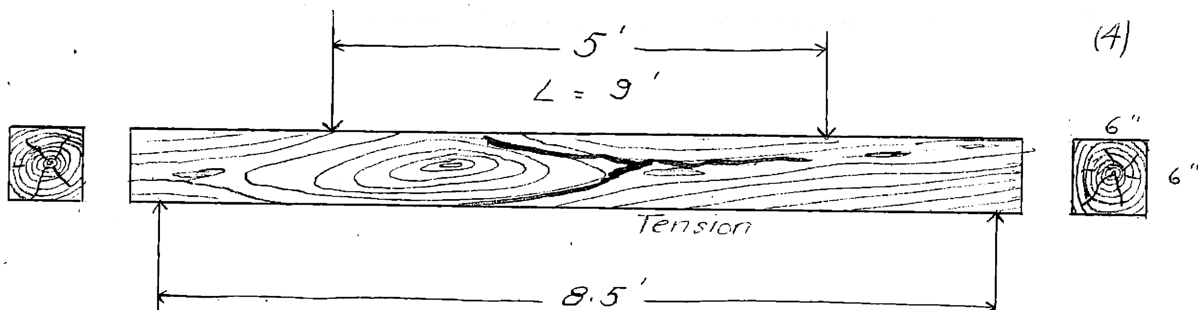
f = 9770 s = 610



W = 30,000

f = 10,000 s = 627

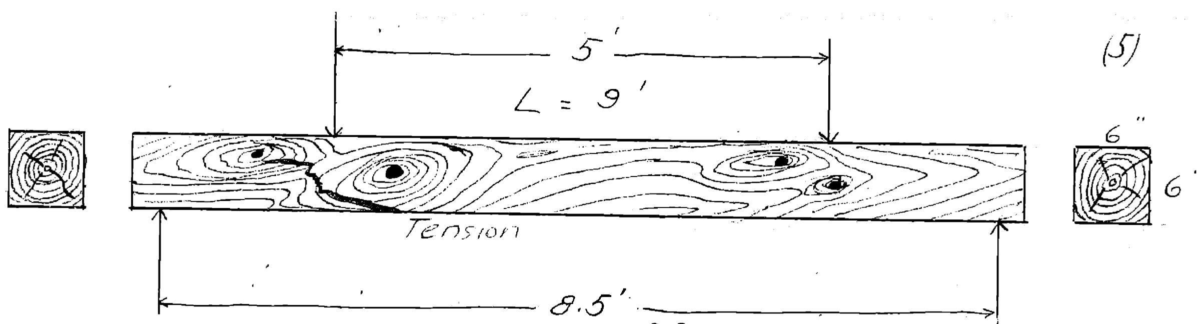
(4)



W = 23,950

f = 7000 s = 503

(5)



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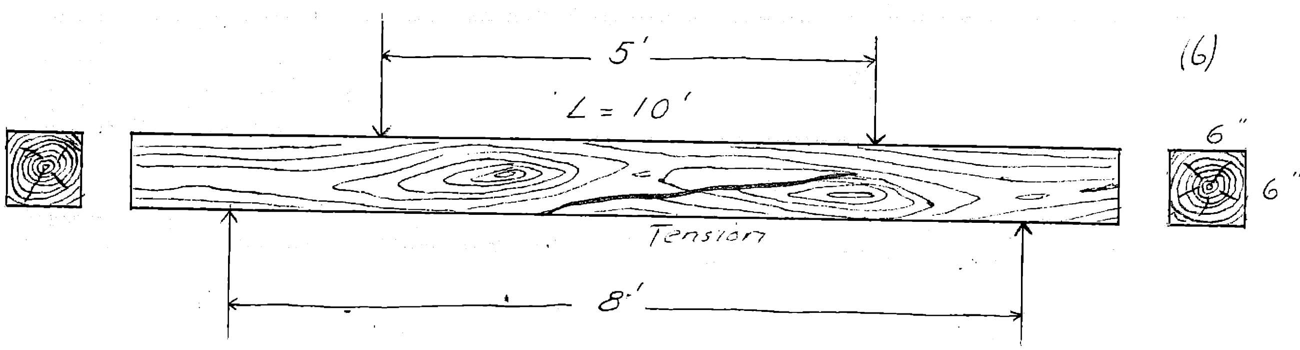
f = 7400 s = 535

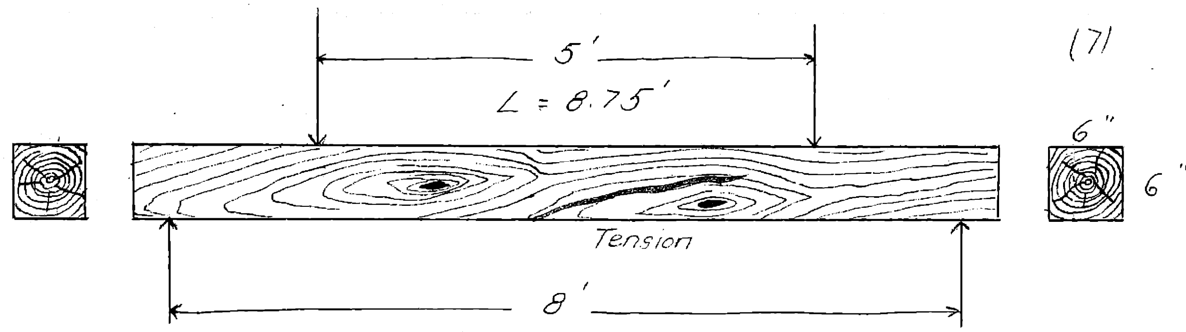
(6)



W = 39,130

f = 9780 s = 815

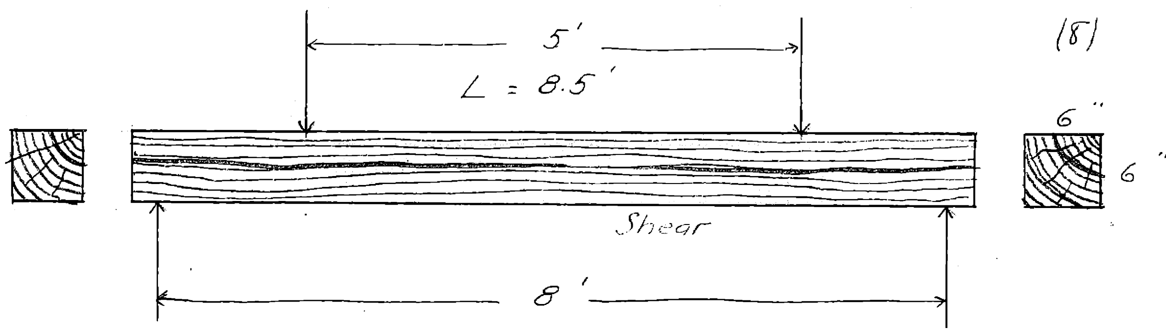
(7)



W = 39,200

f = 9800 s = 817

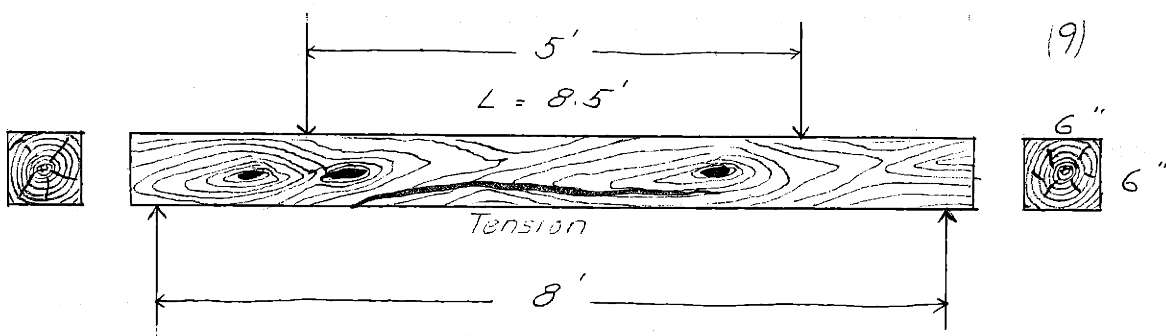
(8)



W = 33,100

f = 8270 s = 690

(9)



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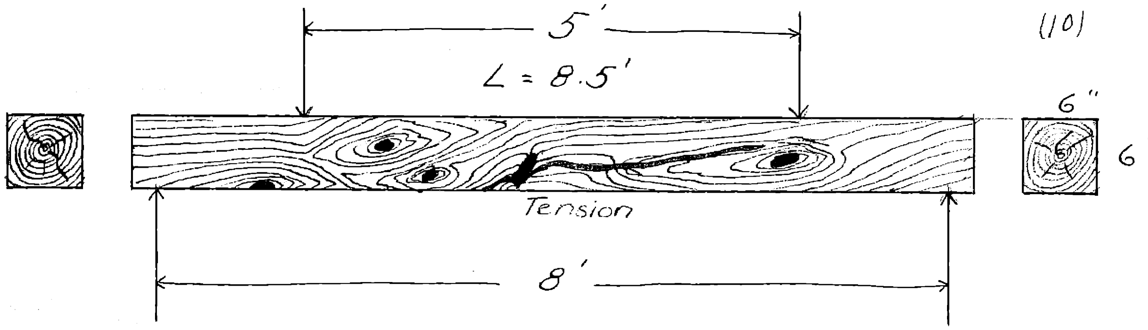
f = 8100 s = 675



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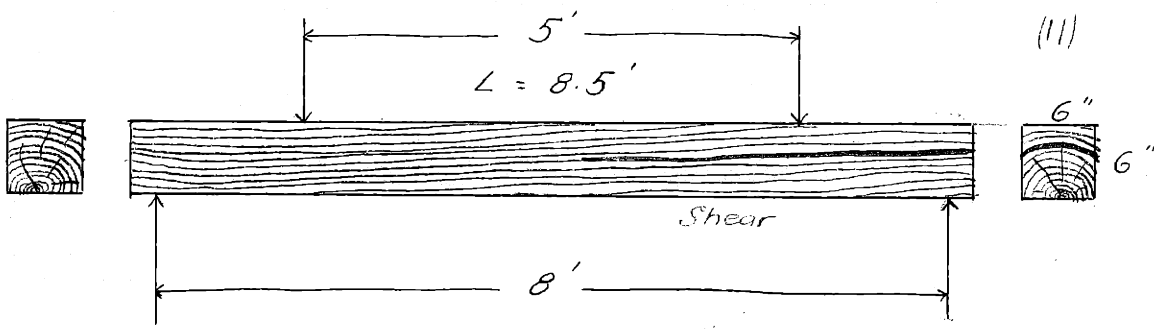
f = 6000   s = 500



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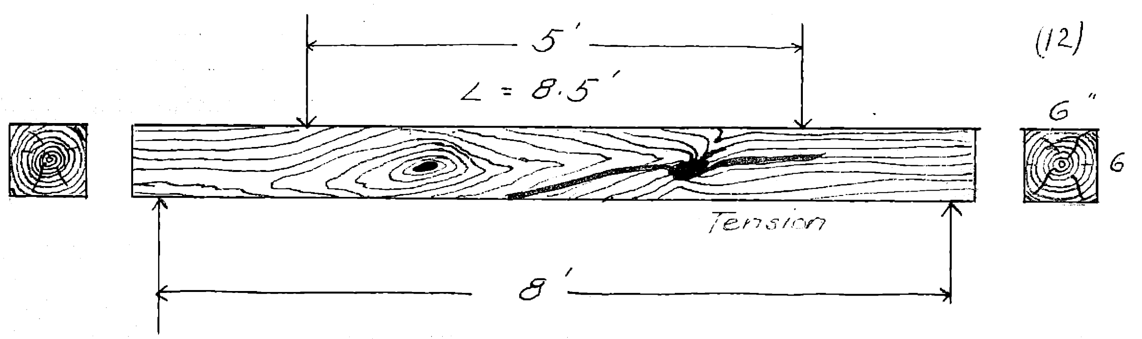

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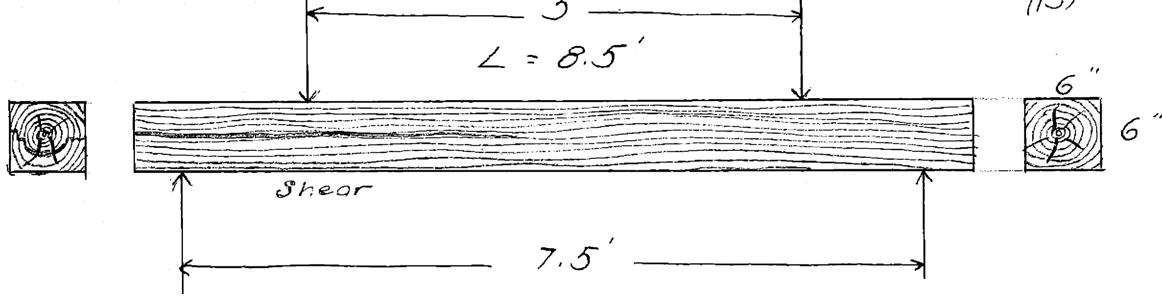
f = 6280   s = 523

(13)



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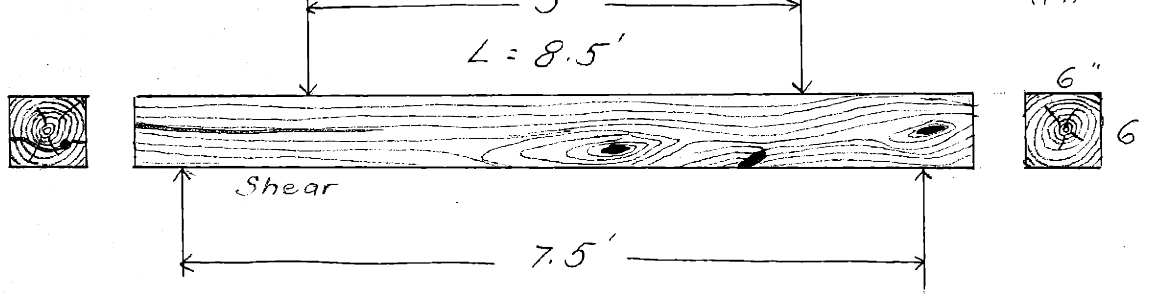
f = 6,140   s = 614

(14)



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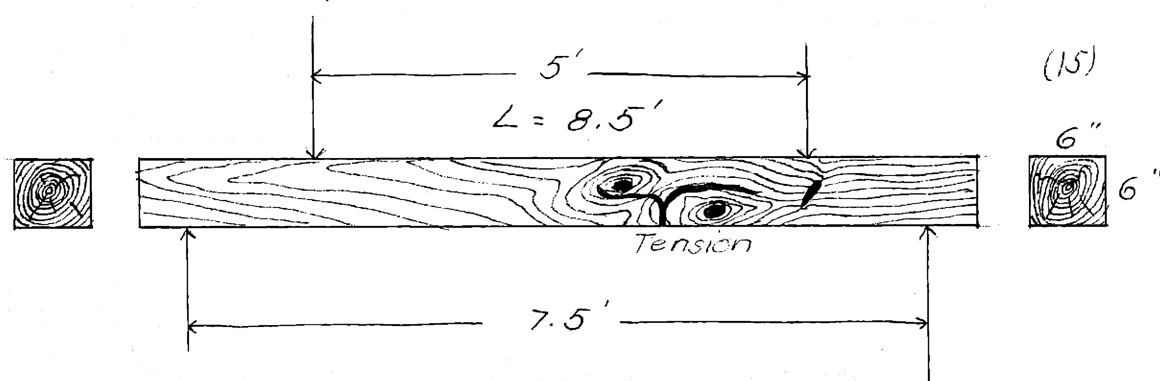
f = 7530   s = 753

(15)



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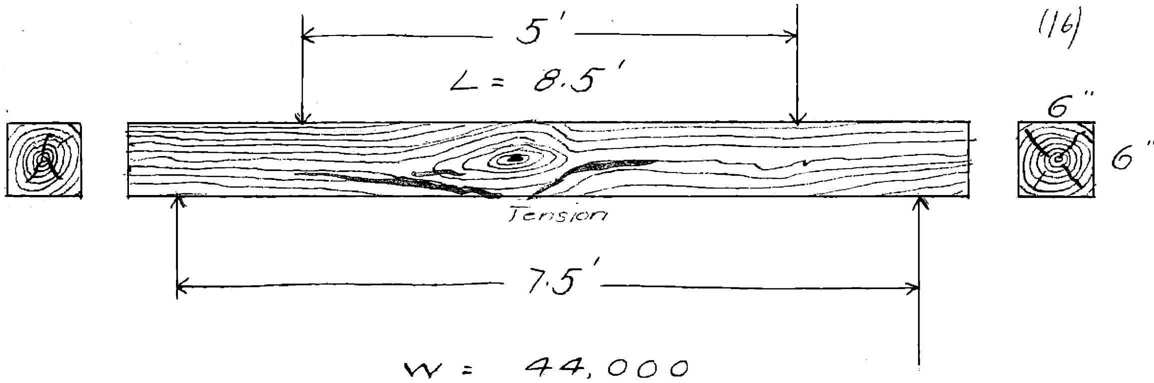

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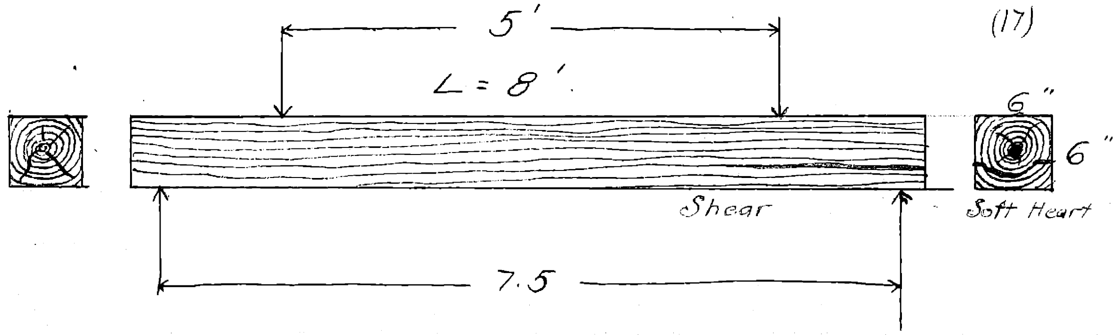

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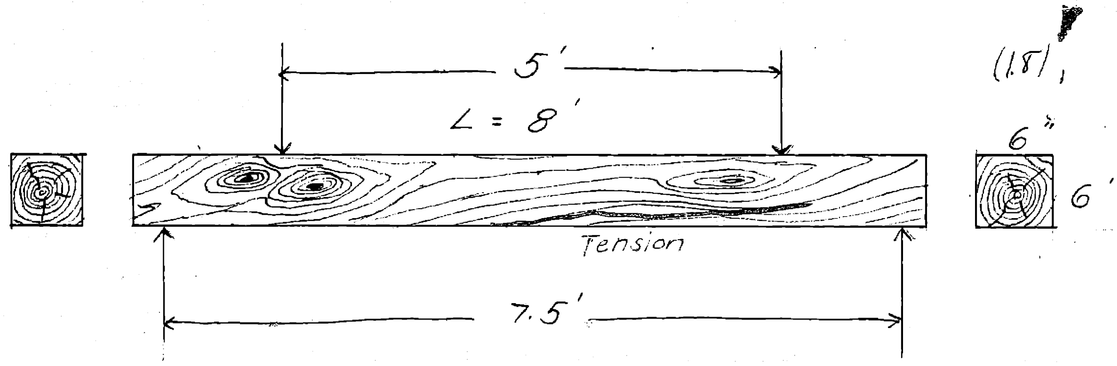
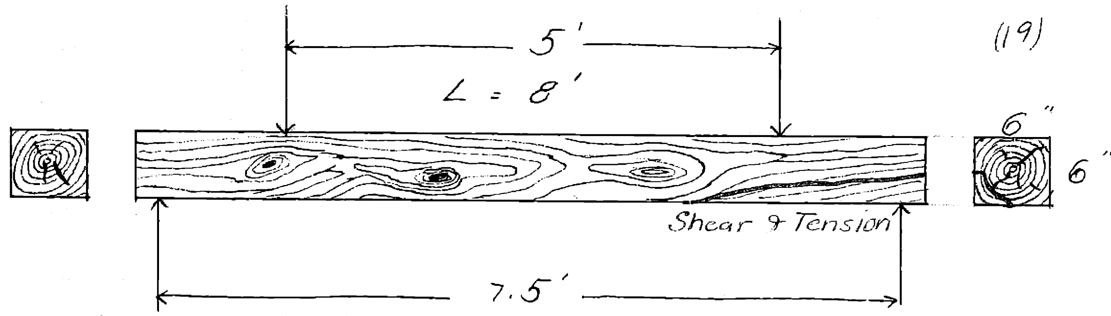


W = 32,500

f = 6780   s = 678

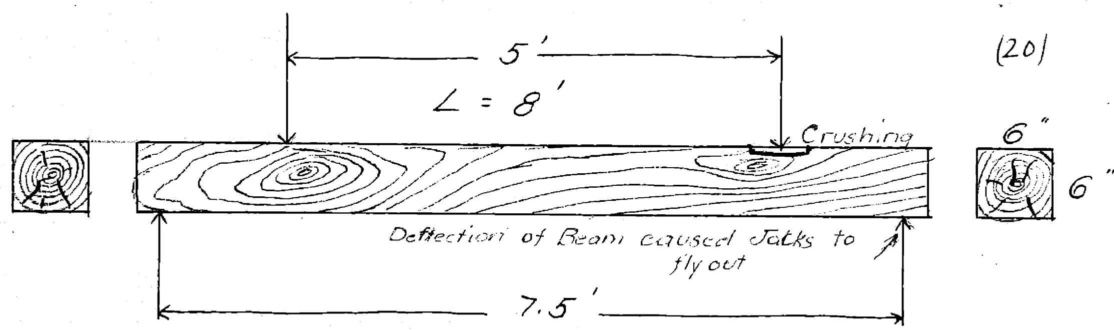
(19)



W = 29,700

f = 6230   s = 623

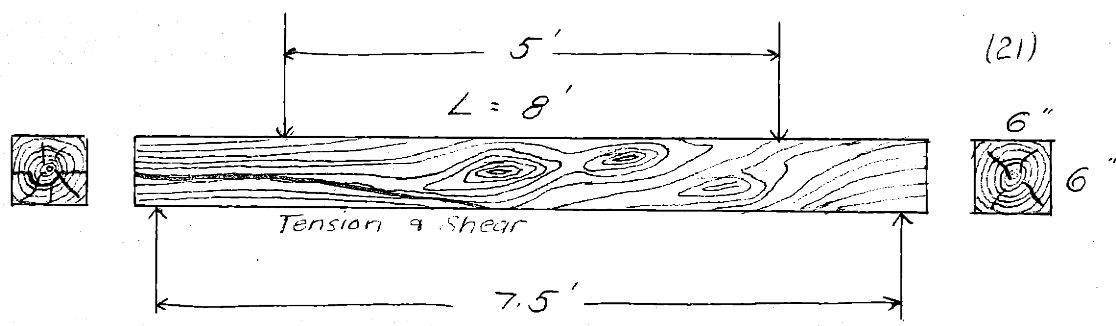
(20)



W = 29,400

f = 6190   s = 619

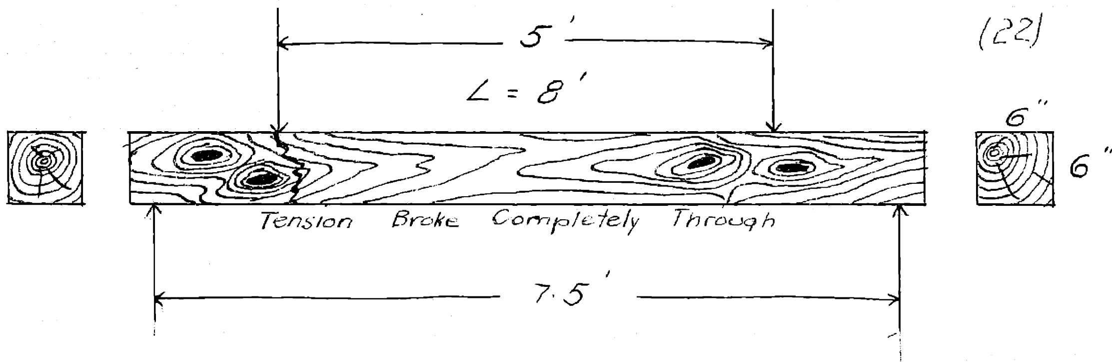
(21)



W = 35,500

f = 7,450   s = 745

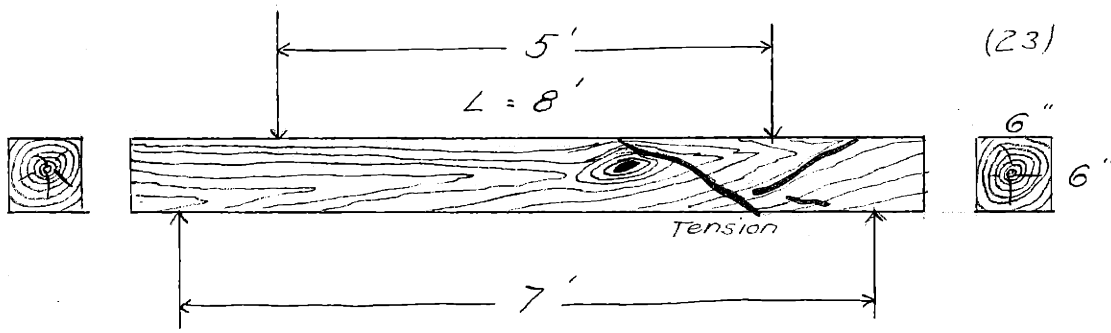
(22)



W = 29,100

f = 6120 s = 612

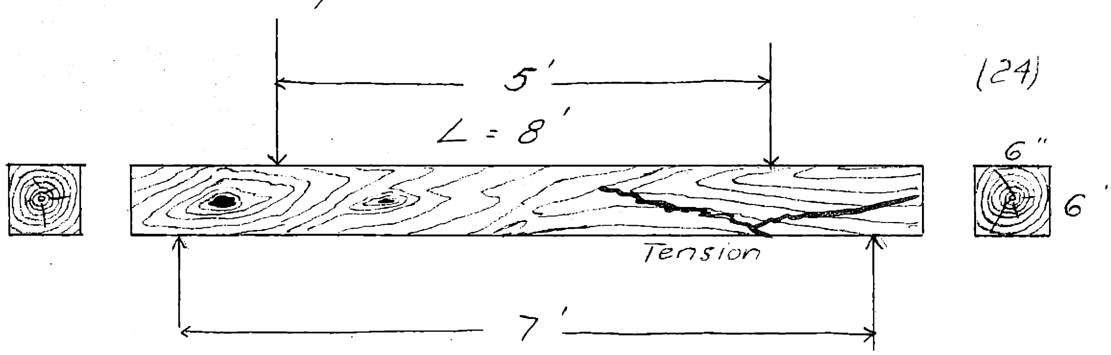
(23)



W = 35,450

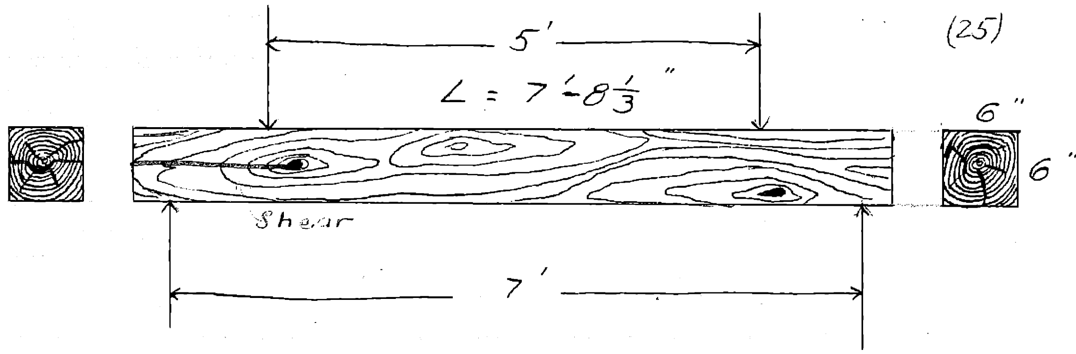
f = 5910 s = 738

(24)



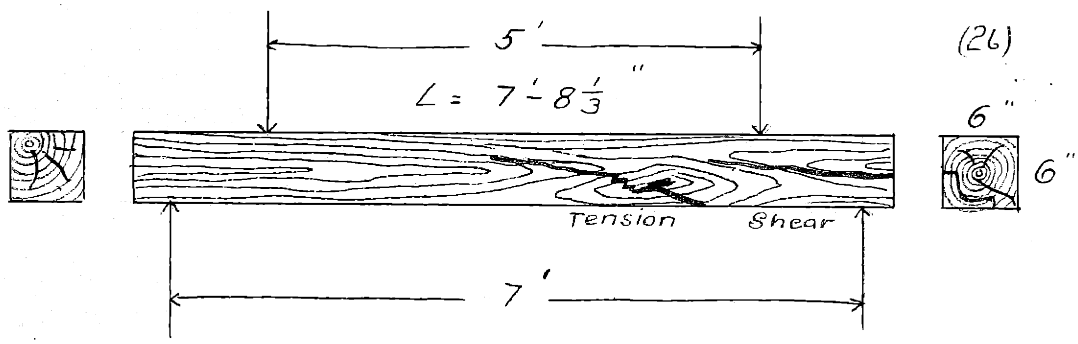
W = 24,330

f = 4050 s = 506



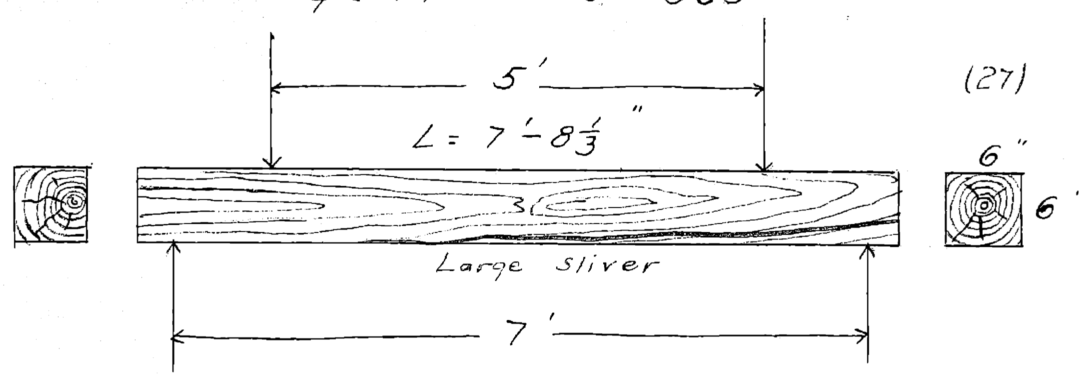
W = 42,700

f = 7120 s = 891



W = 46,230

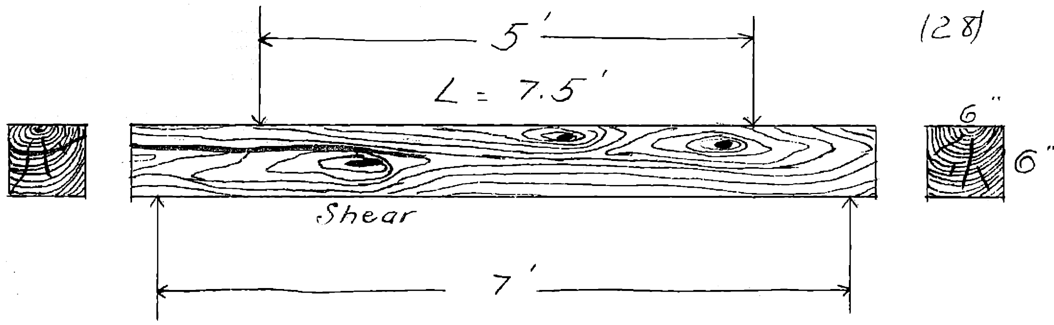
f = 7710 s = 963



W = 38,700

f = 6450 s = 806

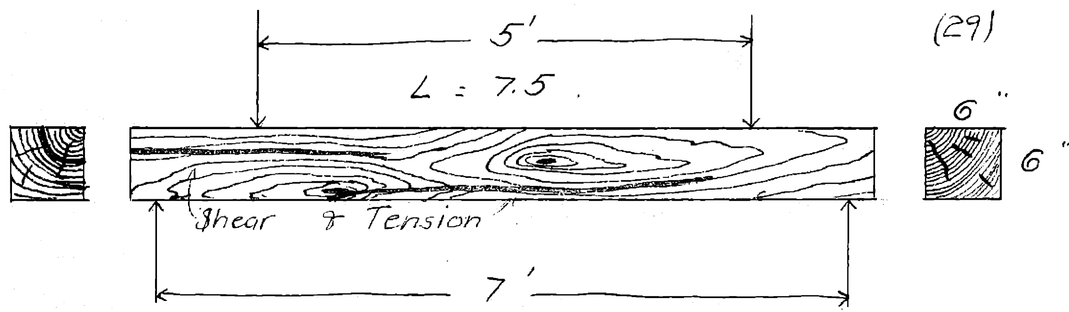
(28)



W = 36,200

f = 6330 s = 754

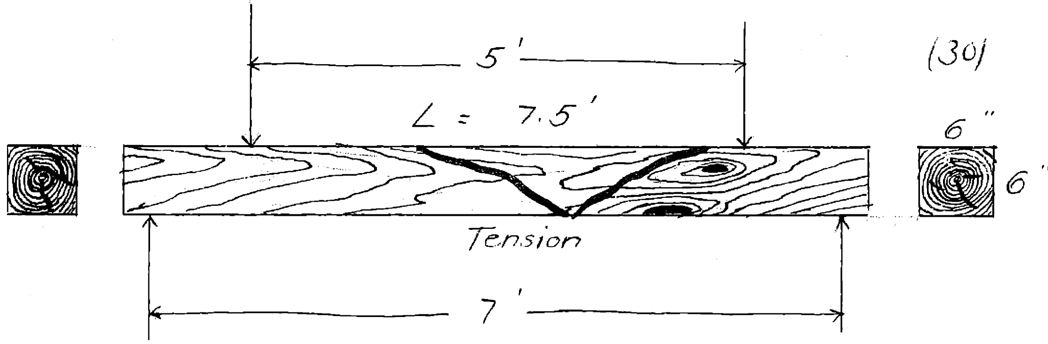
(29)



W = 36,400

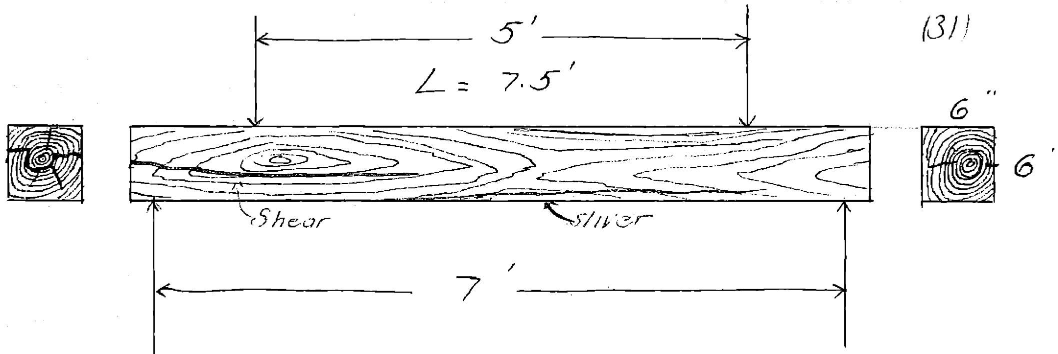
f = 6070 s = 759

(30)



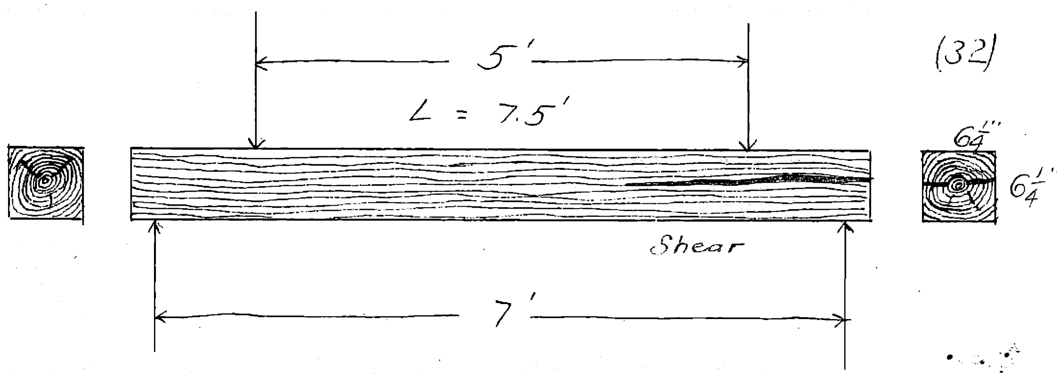
W = 31,400

f = 5230 s = 655



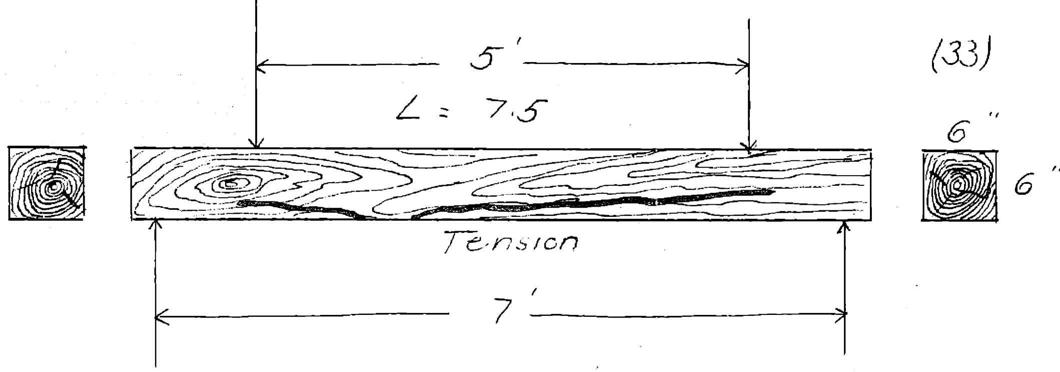
W = 41,900

f = 7000 s = 870



W = 60,000

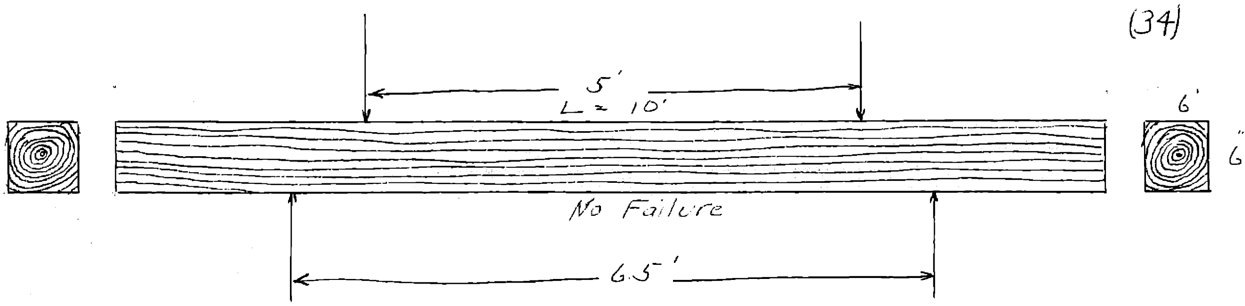
f = 10,000 s = 1150



W = 43,100

f = 7210 s = 897

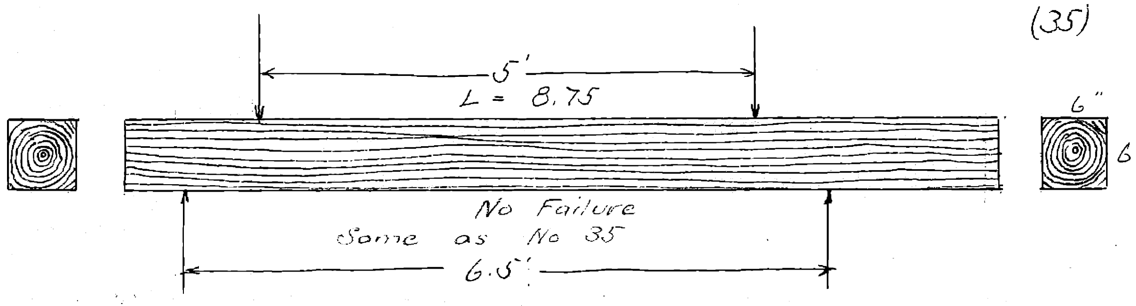
(34)



W = 60,000

f = 7500 S = 1250

(35)



W = 60000

f = 7500 S = 1250

Table IV which follows, gives a complete summary of the results of our tests on full sized specimens.

In order to figure our stresses more readily we constructed Diagram A, which gives the fibre stresses in bending and shear, for a 6"X6" cross-section, for loads from 10,000 to 60,000 lbs. on spans varying from 6'6" to 9'0". The method of using the diagram may readily be seen from the sketch in the lower right-hand corner of the sheet.

Table V. was made to show the relations between the average, maximum and minimum shearing and bending intensities for both those specimens failing by shear and for those not failing by shear.

Table IV

No. Test	Span	Length	Over Hang	Breaking Load	Fibre Stress	Intensity of Shear	Method of Failure	Condition of Wood	Remarks
1	9'-0"	10'-0"	6"	25760	8580	537	Tension	No Knots	The span length was varied from 6'-6" to 9'0". The cross section was 6"x6" except for test no.32 that being $6\frac{1}{4}$"x$6\frac{1}{4}$". Loads were applied 5'0" apart this being approximately the distance center to center of rails.
2	"	"	"	29300	9770	610	"	Knots	
3	"	"	"	30000	10000	627	"	No Knots	
4	8'-6"	9'-0"	3"	23850	7000	500	"	" "	
5	"	"	"	25400	7400	535	"	Knots	
6	8'-0"	10'-0"	12"	39130	9780	815	"	No Knots	
7	"	8'-9"	$4\frac{1}{2}$"	39200	9800	817	Shear	No Knots	
8	"	8'-6"	3"	32000	8000	670	"	" "	
9	"	"	"	32400	8100	675	Tension	Knots	
10	"	"	"	24000	6000	500	"	"	
11	"	"	"	29200	7290	607	Shear	No Knots	
12	"	"	"	25100	6280	523	Tension	Knots	
13	7'-6"	8'-6"	6"	29200	6140	614	Shear	No Knots	
14	"	"	"	36000	7530	753	"	Knots	
15	"	"	"	32000	6710	671	Tension	"	
16	"	"	"	44000	9160	916	"	"	
17	"	8'-0"	3"	18850	3930	393	Shear	Soft heart	
18	"	"	"	32500	6780	678	Tension	Knots	
19	"	"	"	29700	6230	623	Sh. & Ts.	"	
20	"	"	"	29400	6190	619	Crushing	Wet & Knots	
21	"	"	"	35500	7450	745	Sh & Ts.	Knots	
22	"	"	"	29100	6120	612	Tension	Dry Rot	
23	7'-0"	8'-0"	6"	35500	5910	738	"	No Knots	
24	"	"	"	24300	4050	506	"	Knots	
25	"	7'-8"	4"	42700	7120	891	Shear	"	
26	"	"	"	46200	7710	963	Sh. & Ts.	"	
27	"	"	"	38700	6450	806	Large Sliver	No Knots	
28	"	7'-6"	3"	36200	6330	754	Shear	Few "	
29	"	"	"	36400	6070	759	Sh. & Ts.	"	
30	"	"	"	31400	5230	655	Tension	Dry Rot	
31	"	"	"	41900	7000	870	Shear	No Knots	
32	"	"	"	60000	8850	1150	"	"	
33	"	"	"	43100	7250	900	Tension	"	
34	6'-6"	10'	1'-9"	60000	7500	1250	No Failure	"	
35	"	8'-9"	$4\frac{1}{2}$"	60000	7500	1250	"	"	

Diagram A.

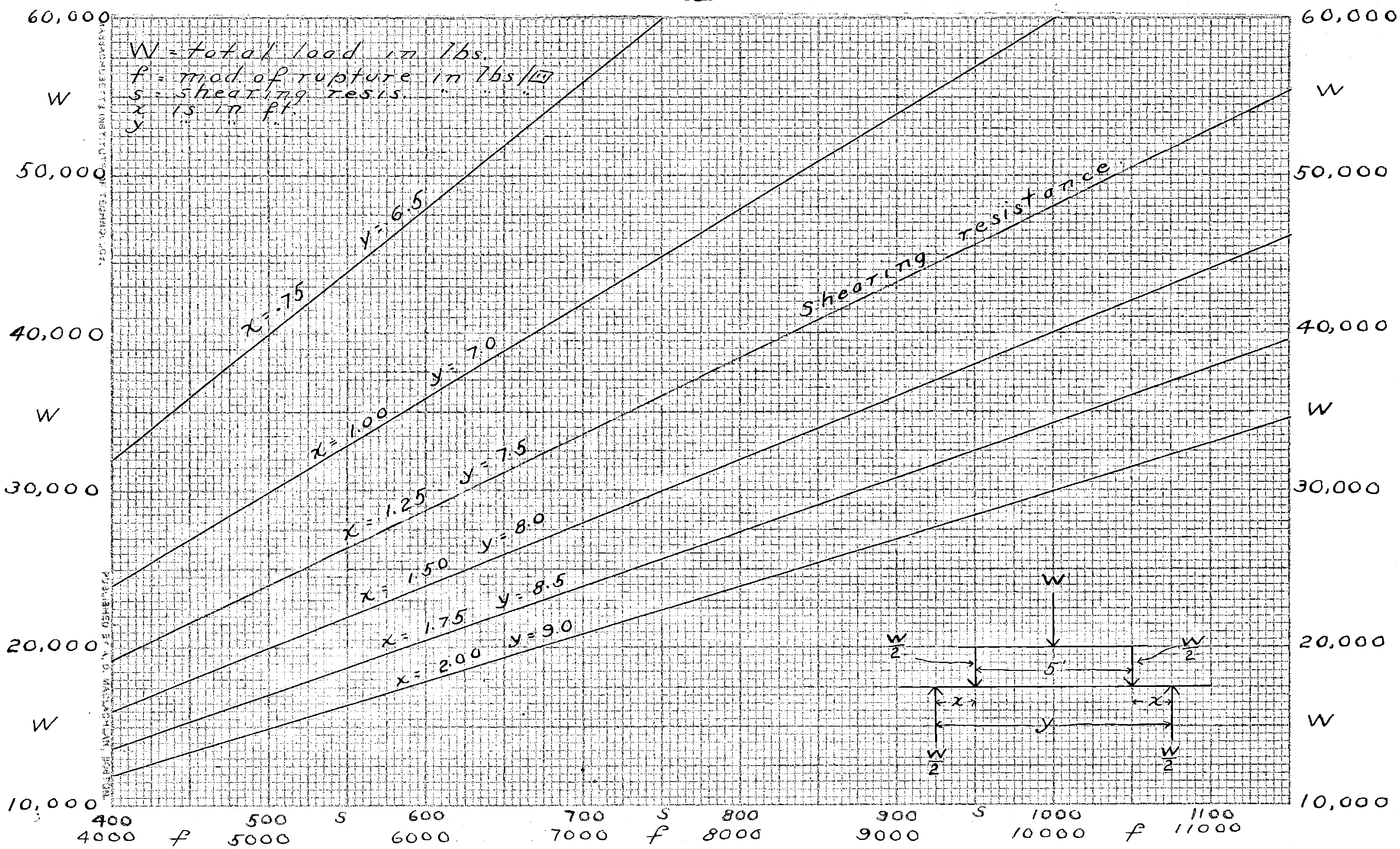

W = total load in lbs.
f = mod. of rupture in lbs/□
s = shearing resis. " "
x is in ft.
y " " "
x = .75 y = 6.5
x = 1.00 y = 7.0
x = 1.25 y = 7.5
x = 1.50 y = 8.0
x = 1.75 y = 8.5
x = 2.00 y = 9.0
shearing resistance
60,000
50,000
40,000
30,000
20,000
10,000
W
400 500 600 700 800 900 1000 1100
4000 5000 6000 7000 8000 9000 10000 11000
s
f
5'
x
y
W/2

Table V.

		Pieces failing in Shear.							Pieces not failing in Shear.						
		No. of	S at max load			f at max. load.			No. of	S at max. load			f at max. load.		
Dimensions	Span.	Tests.	Aver.	Max	Min.	Aver.	Max.	Min.	Tests.	Aver.	Max	Min.	Aver.	Max.	Min.
(6"x6") x 8'-9"-10'	6'-6"	—	—	—	—	—	—	—	2	1250	1250	1250	7500	7500	7500
(6"x6") x 7'-6"-8'-0"	7'-0"	6	898	1150	754	7180	8850	6070	5	721	900	506	5780	7250	4050
(6"x6") x 8'-0"-8'-6"	7'-6"	5	626	753	393	6260	7530	3930	5	699	916	612	6990	9160	6120
(6"x6") x 8'-6"-10'-0"	8'-0"	3	698	817	607	8360	9800	7290	4	628	815	500	8040	9780	6000
(6"x6") x 9'-0"	8'-6"	—	—	—	—	—	—	—	2	517	535	500	7200	7400	7000
(6"x6") x 10'-0"	9'-0"	—	—	—	—	—	—	—	3	591	627	537	9450	10,000	8580
Average		14	759	907	585	7100	8730	5760	21	710	841	651	7340	8520	

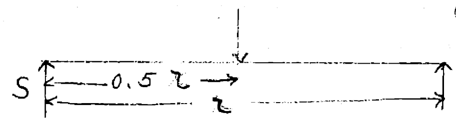
We have endeavored to get as many relations between our tests and those of other experimenters, as possible. In doing this we have tried to keep in mind the limitations with which we must deal, especially in the matter of relying too greatly on averages, obtained from a small number of tests.

Diagram B represents a study of our results in connection with the results of several other experimenters. We have at this point attempted to secure a somewhat definite ratio between the ultimate resistance to longitudinal shear and the proportion of the span which acts as a moment arm in producing the maximum bending moment on the beam. It will be noticed that we have neglected the actual span length in the following discussion. We were obliged to do this since the span length varied with different experimenters and in some cases was not given at all in the records of the tests. We realize that this factor would undoubtedly have some effect on the results, but from other conclusions which we have reached we feel confident that the following discussion will have some definite value.

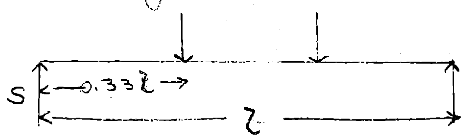
Diagram B is plotted with the ultimate intensities of longitudinal shear at failure as ordinates. The unit we have used is pounds per square inch. The abscissae represent ~~the total shear over the support mult-iplied by the per cent of the span which acts as a moment arm in pro-ducing the maximum bending moment, that is by the~~ proportionate distance between the support and the load, in either the case of central or of two-point loading.

Let M = maximum bending moment.
S = total shear over support
l = length of span

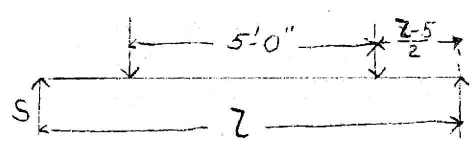
Then for central loading



$M = 0.50 S l$ for central loading



$M = 0.333 S l$ for third point loading



Similarly when l = 9'-0" $M = 0.222 S l$

l = 8'-6" $M = 0.206 S l$

l = 8'-0" $M = 0.188 S l$

l = 7'-6" $M = 0.167 S l$

l = 7'-0" $M = 0.143 S l$

l = 6'-6" $M = 0.115 S l$

Our values used for ultimate intensity of longitudinal shear for centrally loaded yellow pine beams were obtained from three sources.

(1) Tests quoted in Lanza's "Applied Mechanics" made in the applied laboratories at the Massachusetts Institute of Technology. Twenty four failures by longitudinal shear give an average intensity of 245 lbs. per square inch

(2) Tests made by students in the course of the regular work at the Mass. Inst. of Technology. Fifty failures by shear give an average intensity of 276 lbs. per square inch.

(3) Tests made by the U.S. government in 1907. Three failures give an average of 268 lbs. per sq. inch.

The mean average of the above 77 tests give an intensity of 266 lbs. per sq. inch which is the value used in Diagram B.

Our values for third point loading were obtained from the following two sources.

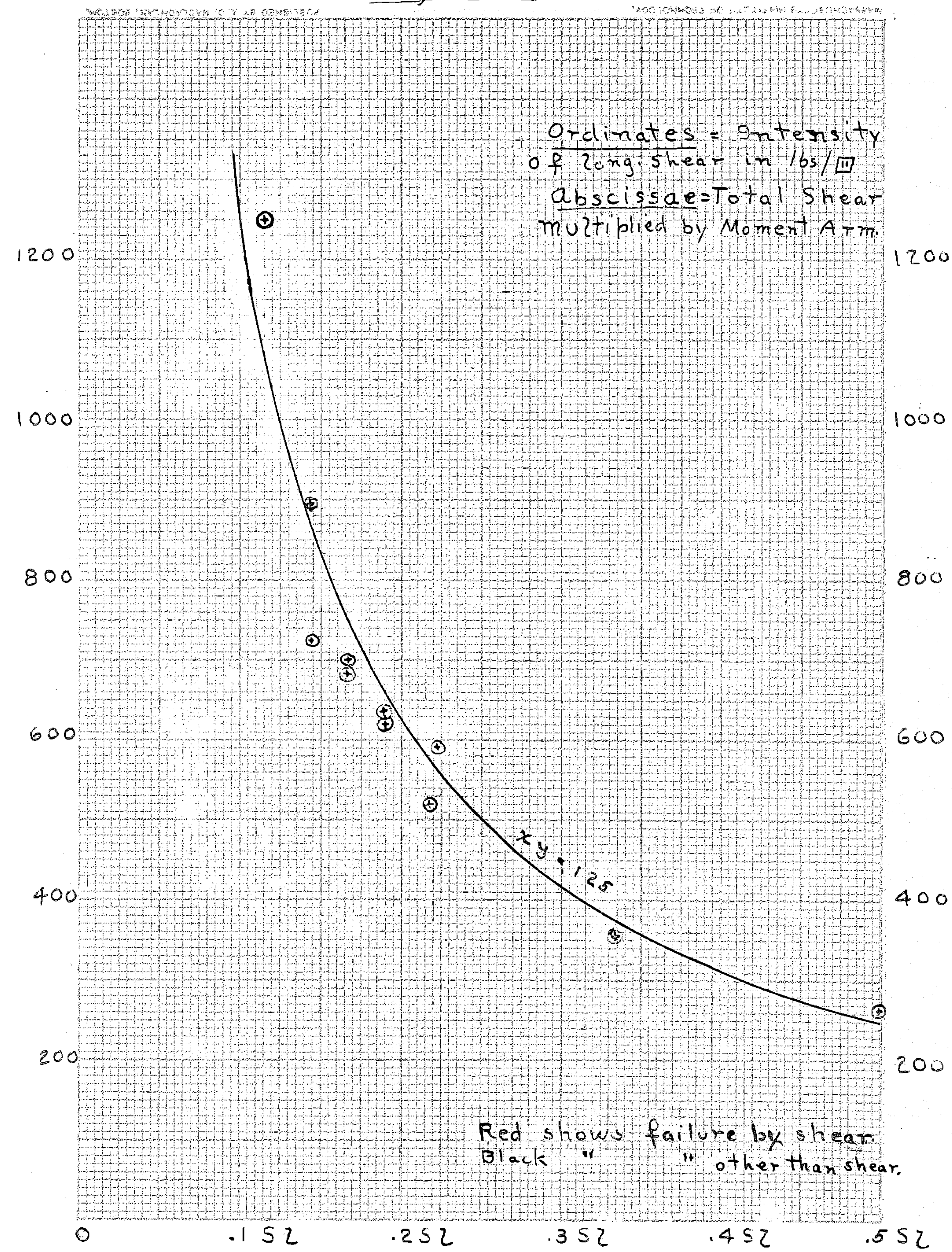
(1). Seven tests made at the University of Illinois give an average intensity of 373 lbs. per square inch.

(2.) Nine tests made by the U. S. government give an average of 335 lbs. per square inch.

The mean average of the above 16 experiments becomes 352 lbs. per square inch, the figure we have used.

The remaining points on the diagram are averages from our own tests. Of these the majority of points represent accurate averages, while in two cases we have neglected tests in which the timber was in an exceptionally different condition than the others.

Diagram B.



Ordinates = Intensity of long. shear in lbs/□"
Abscissae = Total Shear multiplied by Moment Arm.
1200
1000
800
600
400
200
xy = 125
Red shows failure by shear.
Black " " other than shear.
0
.15l
.25l
.35l
.45l
.55l

The curve shown on Diagram B is the one best representing the points as plotted. It is a rectangular hyperbola, having for its general equation $xy =$ a constant.

In this case $xy = 125$

It is evident from the diagram that the intensity of shear varies inversely with the proportion of the span between the support and the nearest load. From the foregoing discussion we see that the maximum bending moment is dependent on the total shear over the support, and on the proportion of the span between the support and the load. Since we have not brought in the actual span length, nor the area of cross-section, we can eliminate the total shear, S, and the span length, l.

Therefore since $xy = 125$

$$y = \frac{125}{x}$$

Let z = % of span acting as moment arm; also let s = intensity of longitudinal shear at failure in pounds per square inch

Then since $x = z$, and $y = s$
we have

$$s = \frac{125}{z}$$

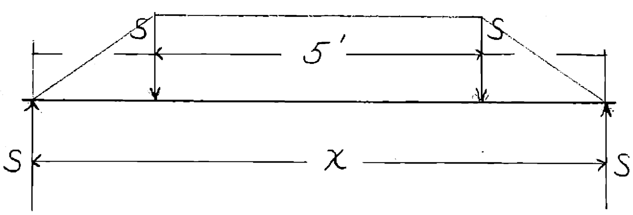
Thus for a case of a yellow pine beam loaded centrally or symmetrically with two loads, we find that the ultimate intensity of longitudinal shear is equal to a constant, 125, divided by the proportion of the span between the support and the nearest load.

We submit this general formula with the limitations noted at the beginning of this discussion.

Table VI. gives a summary of the results of our tests on short specimens. The first column of this table shows the actual span length, and the second column represents the corresponding span for the full-size specimen. This latter span was determined as follows:-

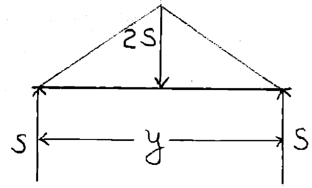
The loading of the full size specimen is shown in "A".

"A"



The loading of the short specimen is shown in "B"

"B"



Thus in Case "A" the bending moment, is a maximum under the load and remains constant between the loads and is equal to

$$S\left[\frac{x-5}{2}\right].$$

The curve of bending moments is shown by the red line on Fig. "A"

In Case "B" the maximum bending moment occurs at the center and is equal to

$$S\left[\frac{y}{2}\right]$$

The curve of bending moments is shown by the red line on Fig "B".

As our idea in these tests was to eliminate the reinforcing action of that part of the beam between the loads, we applied a central load, determining the span length, y, to obtain the same maximum bending moment as given by a corresponding span x on the full size specimen.

We obtained the value of y in terms of x in the following manner.

Since the two bending moments are equal we have

$$\cancel{8}\left[\frac{x-5}{\cancel{2}}\right] = \cancel{8}\left[\frac{y}{\cancel{2}}\right]$$

$$y = x-5$$

Thus for

$x = 6'6''$ ---- $y = 1'6''$

$x = 7'0''$ ---- $y = 2'0''$

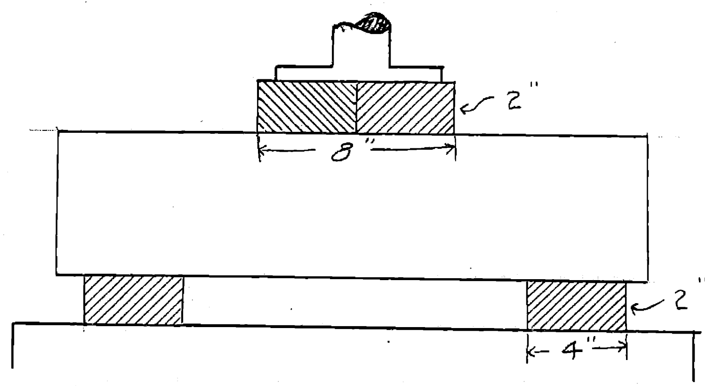
$x = 8'0''$ ---- $y = 3'0''$

$x = 9'0''$ ---- $y = 4'0''$

The remaining part of Table VI. will be evident from inspection.

Table VI

Span	Corresponding span on full size spec.	Length	Over-hang	Br. Load #	Fibre Stress #/□"	Shear. Int. #/□"	Failure
1'-6"	6'-6"	2'-0"	3"	59000	7380	1240	Sh. & Ten.
1'-6"	6'-6"	2'-0"	3"	67000	8380	1390	Tension
2'-0"	7'-0"	2'-6"	3"	55100	9200	1150	Shear
2'-0"	7'-0"	5'-0"	1'-6"	58800	9800	1220	Tension
3'-0"	8'-0"	5'-0"	1'-0"	34000	8500	709	Shear
4'-0"	9'-0"	5'-0"	0'-6"	41000	13700	854	Shear
4'-0"	9'-0"	5'-0"	0'-6"	41200	13730	858	Shear



The above sketch shows our method of loading the short specimens in the Olsen Machine. We used the same size blocks in these tests as we did in the full size specimens in order to obtain the same bearing areas and crushing intensities.

We do not consider the results shown in Table VI as of any practical value for the following reasons.

(1) Since the average load for short specimens was greater than for long specimens, considerable crushing took place especially at the center under the load. This crush-

ing and consequent deformation of the beam would tend to vary the conditions of loading, especially at the points of support.

(2) The distance between the inside edge of the supporting block and the outside edge of the applied load varied from 3" to 1'-6". Thus for the shortest spans, the distance in clear between the blocks is so small that the tendency is rather for shear at right angles to the grain than for longitudinal shear.

(3) It is evident, from an inspection of Table VI that the ultimate intensities of longitudinal shear, even with practically no overhang of the beam, are so high in comparison with the results of the tests on full size specimens, that we are not meeting the actual conditions. Therefore we do not

feel at liberty to draw any definite conclusions from these tests.

The following computation illustrates the method of determining the maximum intensity of horizontal shear for a bridge tie.

Tie 8"x8" cross-section

Cooper's E 60 Loading.

Assume one wheel load uniformly distibuted over three ties, this being the common assumption.

Load on one tie equals

$$\frac{60,000}{2 \times 3} = 10,000^{\#} \text{ under each rail.}$$

Allowing 100% for impact we have

$$\begin{array}{r} 10000 \\ \underline{10000} \\ 20,000 \end{array}$$

If s = intensity of longitudinal shear we get

$$s = \frac{\overset{10,000}{\cancel{20,000}}}{64} \times \frac{3}{4} = 469 \text{ lbs/sq.inch.}$$

Under ordinary conditions 100 lbs. per square inch has been considered a fair working value for longleaf yellow pine, and 400 lbs per square inch has been used for an ultimate intensity of longitudinal shear. However in the design of ties for railroad bridges, having open floors, the preceding method of computation is generally adopted and a value of from 100 to 400 lbs. per square inch is considered allowable for a working value. In using the above values, the assumption was made that there would be a considerable resistance to longitudinal shear due to the manner of loading on this style of bridge.

Our results prove that this assumption was correct and well on the safe side. For example in the case of the tie assumed above our results according to Diagram B, page 50, give from 550 to 1200 lbs. per square inch for the intensity of longitudinal shear, for the conditions of loading as found on railroad bridges.

In summarizing Part "C" of this thesis we conclude
(1) that the reinforcement due to that part of the beam between the loads offers the most important resistance to longitudinal shear.
(2) that the effect of the overhang is of some importance in increasing the resistance, but to just what extent we were not successful in determining.
(3) that there is a fairly definite relation between the proportion of

the span between the support and the load and the ultimate intensity of longitudinal shear.

(4) that the common methods of computation of bridge ties based on the assumption that the form of loading offers increased resistance to longitudinal shear are very reasonable and well on the safe side.

As a continuance of this subject it has occurred to us that practical results might be obtained

(a) from tests to determine the effect of variations in actual span lengths as compared with variations in the proportionate part of the span between the support and the load in the case of two point loading

(b) from a very great number

of tests on specially picked timber to determine if possible the effect of the overhang on resistance to longitudinal shear.

(c) from some method of applying the results of our tests to beams loaded uniformly or at more than two points.

Finally we wish to acknowledge the assistance rendered to us in our testing by I. Hausman 1911 and S. E. Bates 1911. We also wish to thank the faculty of the Massachusetts Institute of Technology for the timber supplied us for our tests, and for the valuable advice which we have received from the members of both the Civil and Mechanical Engineering Departments.