VOL. IX. 1895. PART I.

TRANSACTIONS

OF THE

Canadian Society of Civil Engineers

JANUARY TO JUNE, 1895

. . . CONTENTS . . .

ILLUSTRATIONS.

PROCEEDINGS.

Montreal:

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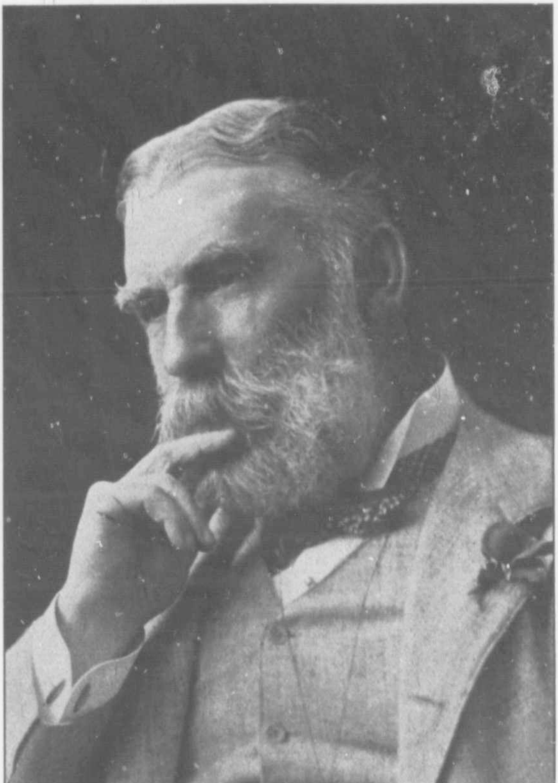
1895



Yours very truly
Thomas Monro.

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OF

The Canadian Society of Civil Engineers.

VOL. [illegible], PART I.

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CONTENTS.

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PROCEEDINGS.

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In writing papers, or discussions on papers, the use of the first person should be avoided. They should be legibly written on foolscap paper, on one side only, leaving a margin on the left side.

Illustrations, when necessary, should be drawn on tracing paper to as small a scale as is consistent with distinctness. They should not be more than 10 inches in height, but *in no case* should any one figure exceed this height. Black ink only should be used, and all lines, lettering, etc., must be clear and distinct.

When necessary to illustrate a paper for reading, diagrams must be furnished. These must be bold, distinct, and clearly visible in detail for a distance of thirty feet.

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All communications must be forwarded to the Secretary of the Society, from whom any further information may be obtained.

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Friday, 25th January.

THOMAS MONRO, President, in the Chair.

Paper No. 101.

THE STRENGTH OF CANADIAN DOUGLAS FIR, RED PINE, WHITE PINE AND SPRUCE.

By HENRY T. BOVEY, M.INST.C.E., LL.D.

In the present Paper it is proposed to give a statement of the results which have been obtained up to the present time, from the numerous experiments which have been carried out in the Testing Laboratories, McGill University, on the strength of Canadian Douglas Fir, Red Pine, White Pine and Spruce.

These experiments, which have now extended over a period of more than two years, will still be continued, and it is hoped that the results will be set before the profession in a Paper on some future occasion.

In order that the subject may be treated in as comprehensive a manner as possible, the engineers and lumber-merchants, who must necessarily be most particularly interested, are earnestly requested to give their co-operation. They can render valuable service by sending to the University Laboratories timbers of any and all sizes. These timbers should, in each case, be accompanied by a history giving the treatment of the timber from the time when the tree was felled, as, for example, the locality in which the tree grew should be specified, the manner in which the log was brought to the mill, the length of time during which it was kept in water (salt or fresh), the time during which it was kept in the pile at the mill, and, if the timber has already been in service, the length of this service. Any other details respecting the history of the timber may also be given, so that the information may in every case be as complete as circumstances will permit.

The attention of members is specially directed to the tables showing the deflection of beams under transverse loading, and also to tables showing the extension of specimens under direct tension.

These tables tend to prove conclusively the statement made by the author many years ago, *i.e.*, that timber, unlike iron and steel, may be strained to a point near the breaking point without being seriously injured. It will be observed that in almost all cases the increments of deflection and extension, almost up to the point of fracture, are very nearly proportional to the increments of load, and it seems impossible to define a limit of elasticity for timber. This probably accounts for the continued existence of many timber structures in which the timbers have been and are still continually subjected to excessive stresses, the factor of safety being often less than $1\frac{1}{2}$. Whether it is advisable so to strain timber is another question, and experiments are still required to show how timber is affected by frequently repeated strains.

TRANSVERSE STRENGTH.

The following Table gives in inches the distances between the centres of the end bearings (l), the mean depths (d) and the mean breadths (b) of the Beams I to LXI referred to in this Paper :—

Beams	I	II	III	IV	V	VI	VII
l	96	66	66	69	69	69	69
	×	×	×	×	×	×	×
d	12.125	12.125	5.375	9.125	9.125	6.125	6
	×	×	×	×	×	×	×
b	9	5.625	4.25	5	5	6	5.8125

Beams	VIII	IX	X	XI	XII	XIII	XIV
l	69	204	198	204	204	204	204
	×	×	×	×	×	×	×
d	5.125	14.875	14.875	14.875	14.875	14.75	14.75
	×	×	×	×	×	×	×
b	5.5	9	6	8.6875	8.8125	6	6

Beams	XV	XVI	XVII	XVIII	XIX	XX	XXI
l	198	198	138	138	138	138	138
	×	×	×	×	×	×	×
d	15	15	15.125	17.8	12.1	12	8.98
	×	×	×	×	×	×	×
b	6.125	6.125	9	8.76	9.1	8.88	5.95

Beams	XXII	XXIII	XXIV	XXV	XXVI	XXVII	XXVIII
l	162	186	132	144	210	210	210
	×	×	×	×	×	×	×
d	15.6875	14.35	16.2	15.65	13.25	13.125	11.25
	×	×	×	×	×	×	×
b	7.75	8.78	7.75	8.2	6.375	6.1875	6.34375

Beams	XXIX	XXX	XXXI	XXXII	XXXIII	XXXIV	XXXV
l	210	174	174	180	180	156	156
	×	×	×	×	×	×	×
d	11.25	7.25	7.125	8.125	11.125	9.125	11.15
	×	×	×	×	×	×	×
b	6.25	6.1875	6.21875	3.1	3.1	3.125	3.325

Beams	XXXVI	XXXVII	XXXVIII	XXXIX	XL	XLI	XLII
l	288	288	114	102	120	120	288
	×	×	×	×	×	×	×
d	18	18	18	18	18	18	18
	×	×	×	×	×	×	×
b	9	9	9	9	9	9	9

Beams	XLIII	XLIV	XLV	XLVI	XLVII	XLVIII	XLIX
l	120	120	288	120	120	150	150
	×	×	×	×	×	×	×
b	18	18	18	18	18	15.1875	15.375
	×	×	×	×	×	×	×
b	9	9	9	9	9	9.375	9.125

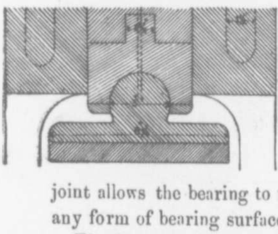
Beams	L	LI	LII	LIII	LIV	LV
l	186	192	180	180	288	120
	×	×	×	×	×	×
d	15	15.12	14.85	15	17.5	17.5
	×	×	×	×	×	×
b	9.0625	9	9.05	9.05	8.875	8.875

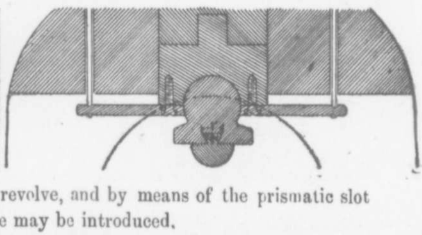
Beams	LVI	LVII	LVIII	LIX	LX	LXI
l	120	180	180	180	138	186
	×	×	×	×	×	×
d	17.5	15	14.75	15	11.25	14.5
	×	×	×	×	×	×
b	8.9375	9	6	9	8.875	5.625

The transverse tests were carried out with the Wicksteed 100-to machine by means of a specially designed arrangement shown in the photograph on the opposite page.

By this arrangement the two ends are gradually forced downwards while the centre is supported upon the addle suspended from the lever of the machine. Thus the two halves of the beam are really equivalent to two cantilevers loaded at the ends. By means of a very simple device, the pressure can be increased so regularly as to ensure an absolute equality in these end loads.

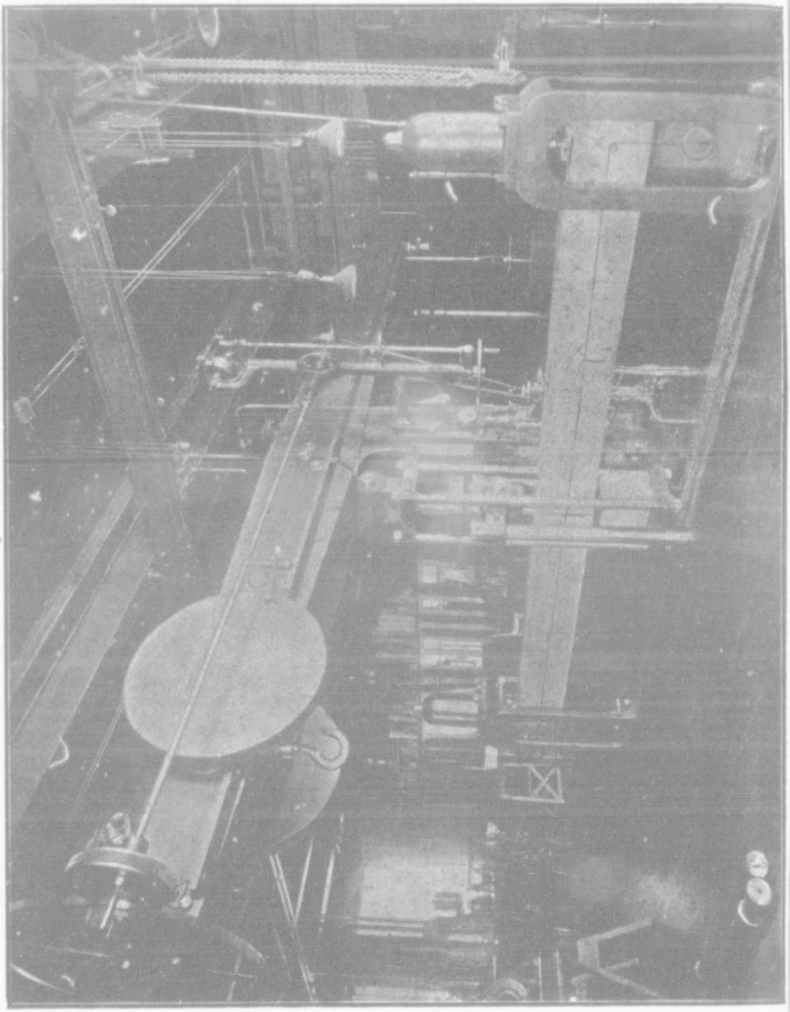
Figures 1 and 2 show the device employed to keep the pressure on the ends of the beam always normal to the surface. The spherical





joint allows the bearing to revolve, and by means of the prismatic slot any form of bearing surface may be introduced.

The formula used in calculating the skin-strengths and co-efficients of elasticity have been deduced by means of the ordinary theory of flexure which is based upon assumptions which actual experience shows to be far from being true. These assumptions are :—



(*a*) That the beam is symmetrical with respect to a certain plane.

(*b*) That the material of the beam is homogeneous.

(*c*) That sections which are plane before bending remain plane after bending.

(*d*) That the ratio of longitudinal stress to the corresponding strain is the ordinary (i. e. Young's) modulus of elasticity, notwithstanding the lateral connection of the elementary layers.

(*e*) That these elementary layers expand and contract freely under tensile and compressive forces.

In each case, the skin stress at the point of fracture in lbs. per sq. in. has been determined by means of the formula,

$$f = \frac{3}{2} \frac{l\,(2\,W_1 + W_2)}{b\,d^2}$$

W_1-lbs. being the weight at an end, W_2-lbs. half the weight of the beam, l-ins. the length of the beam between the two end centres of pressure, b-ins. the breadth and d-ins. the depth at the section of fracture.

In practice, the breaking weight, $W_1 + \frac{1}{2} W_2$, is usually determined from the formula,

$$W_1 + \tfrac{1}{2} W_2 = C \frac{b\,d^2}{l},$$

C being the co-efficient of rupture. Hence, $f = 3$ C.

It may perhaps be well to point out that a very small error in estimating the depth of a beam may lead to a considerable error in the calculated skin stress. Thus from the formula just given it appears that if Δf be the change in the skin stress corresponding to a change Δd in the depth, then

$$\Delta f = -\,2\frac{f}{d}\Delta\,d,$$

and the skin stress will be increased or diminished by this amount according as the estimated depth is too small or too great by the amount Δd.

For instance, in the case of the Spruce Beam No. *L*, the calculated skin stress, disregarding the diminution of depth due to compression, is 5,123 lbs. The initial depth (d) of the beam was 17.5 ins., and the amount of the compression (Δd) 2 ins. Thus the error (Δf) in the skin tress is

$$\Delta f = -\,2\frac{5123}{17.5}2 = 1171 \text{ lbs. per sq. in.},$$

and the actual stress becomes $5123 + 1171 = 6294$ lbs. per sq. in., showing an increase of 22.8 per cent.

Now, in every example of transverse testing, the material is more or less compressed at the central support. The central support in the following examples was a hardwood block of 44 ins. diameter. The amount of the compression at this support depends not only upon the nature of the material of the beam and upon the character of the support, but also very especially upon the ratio of the length of the beam to its depth. In calculating the skin stress corresponding to the breaking weight, therefore, three assumptions may be made :—

1st. That the compression at the support may be disregarded.

2nd. That the effective depth of the beam may be taken as equal to the initial depth minus the amount of the compression, and that the usual law may be assumed to hold good for the whole of this effective depth.

3rd. That the compression portion of the beam is alone affected, so that the so-called neutral plane remains in the same position relatively to the tension face of the beam from the commencement of the test to the end.

Calculations based upon these three assumptions have been made in several of the following cases, and it will be observed that in all cases the skin stress calculated upon the first assumption is invariably less than the skin stress determined upon either of the remaining assumptions. Thus any error is on the safe side.

It should be remembered, however, that it is possible, and even probable, that neither of these assumptions is even approximately correct, at all events, beyond the limit of elasticity, which in the case of timber still remains indefinite. The portion in compression doubtless acquires increased rigidity, and thus exerts a continually increasing resistance, so that there is produced a more or less perfect equalization of stress throughout the portion of the beam under compression, and this equalization will doubtless materially affect both the elasticity and the strength.

An interesting paper on the surface-loading of beams was presented by Prof. C. A. Carus-Wilson to the Physical Society of London, (Eng.), and an abstract of this Paper is to be found in the author's treatise on the Theory of Structures.

The co-efficient of elasticity, as determined by the tranverse loading, is deduced from the formula

$$E = \frac{1}{4} \frac{\Delta W}{\Delta D} \cdot \frac{l^3}{bd^3}$$

W being the increment of weight corresponding to the increment D of the deflection.

Here again an error Δd in the estimated depth will produce an error ΔE in the calculated co-efficient of elasticity measured by

$$\Delta E = -3 \frac{E}{d} \Delta d.$$

DOUGLAS FIR.

Beams I to III were sent to the Testing Laboratory by Mr. John Kennedy, Chief Engineer of the Montreal Harbour Works.

Beams I and II were of good average quality.

Beam I was tested on March 1st, 1893, with the annual rings as in Fig. 3. The load was gradually increased until it amounted to 45,000 lbs., when the beam failed by the tearing apart of the fibres on the tension face.

The maximum skin stress corresponding to the breaking weight of 45,000 lbs. is 4,897 lbs. per square inch.

The co-efficient of elasticity, as deduced from an increment in the deflection of .23 in. between the loads of 3,500 and 22,500 lbs, is 1,138,900 lbs.

Table A shows the several readings.

Beam II was tested on March 2nd, 1893, with the annual rings running as in Fig. 4.

The load was gradually increased until it amounted to 36,575 lbs., when the beam failed by shearing longitudinally.

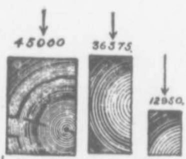


Fig 3. Fig. 4. Fig. 5.

The maximum skin stress corresponding to this breaking weight is 4,378 lbs. per square inch.

In connection with this experiment it is of interest to note that the timber, although it had failed by longitudinal shear, still possessed a very large amount of transverse strength, and similar facts will be subsequently referred to in the case of other beams. After the fracture, the load upon the beam was again gradually increased to 34,000 lbs. before a second failure occurred.

The co-efficient of elasticity, as determined by the increment in the deflection of .1 in. between the loads 2,000 and 18,000 lbs., is 1,146,-900 lbs.

Table B shows the several readings.

Beam III was tested on March 2nd, 1893, with the annual rings as in Fig. 5.

This Beam was of especially excellent quality, with clear, close, parallel grain, perfectly sound and free from knots.

The load was gradually increased until it amounted to 12,950 lbs., when it failed by shearing longitudinally.

The maximum skin stress corresponding to the breaking load is 10,441 lbs. per square inch.

The co-efficient of elasticity, as determined by an increment in the deflection of .2 in. between the loads of 500 and 4,500 lbs., is 2,178,-100 lbs.

Table B gives the several readings.

Beams IV to VIII were sent to the laboratory by the British Columbia Mills Timber & Trading Company through Mr. C. M. Beecher.

These beams were cut out of trees grown on the coast section of British Columbia, and felled in the fall or during the winter. The whole of the beams were free from knots, of good quality, and with the grain running straight from end to end.

Beam IV was tested May 17th, 1893, with the annual rings somewhat oblique as shown in Fig. 6. Under a load of 16,720 lbs. it

Figure. 6.

failed by shearing longitudinally along a plane AB at right angles to the annual rings, the distance between the ends of the portions above and below the plane of shear being $\frac{1}{4}$ in. The plane of shear extended to a distance of about 36 ins. from the end of the beam.

The maximum skin stress corresponding to the breaking load is 4,156 lbs. per square inch.

The co-efficient of elasticity, as determined by an increase in the deflection of .14 in. between the loads of 2,000 and 8,000 lbs., is 926,500 lbs.

Table B shows the several readings.

After the beam had sheared longitudinally, the jockey weight was run back, and the load again gradually applied until it amounted to 15,000 lbs., when fracture occurred by the tearing apart of the fibres on the tension face. Under this load of 15,000 lbs. an opening of ½ in. was developed in the end at the plane of shear.

On May 11th this beam weighed 56 lbs. 13 ozs., or 28.59 lbs. per cubic foot. On May 17th, the weight of the beam was 56 lbs. 3 ozs., or 28.27 lbs. per cubic foot, so that while in the laboratory this beam lost in weight at the rate of .0533 lb. per cubic foot per day.

Beam V was tested on May 19th, 1893, with the annual rings somewhat oblique as shown in Fig. 7. It failed by the tearing apart of the fibres on the tension face under a load of 23,610 lbs.

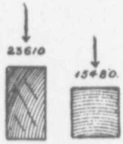


Fig. 7 Fig. 8.

The maximum skin stress corresponding to this load is 5,869 lbs. per square inch.

The co-efficient of elasticity, as determined by an increase in the deflection of .24 in. between the loads of 1,000 lbs. and 11,500 lbs., is 946,270 lbs.

Table B shows the several readings.

The weight of the beam on May 11th was 59 lbs., or 29.59 lbs. per cubic foot. The weight of the beam on May 19th was 58 lbs. 3 ozs., or 29.18 lbs. per cubic foot, so that the loss in weight in the laboratory was at the rate of .05125 lb. per cubic foot per day.

Beam VI was tested May 22nd, 1893, with the annual rings as in Fig. 8. Under a load of 15,480 lbs. it failed by the tearing apart of the fibres on the tension face.

The corresponding maximum skin stress is 7,116 lbs.

The co-efficient of elasticity as determined by an increase in the deflection of .3 in. between the loads of 500 lbs. and 8,000 lbs. is 1,489,215 lbs.

Table B shows the several readings.

The weight of the beam on May 11th was 49 lbs. 6 ozs., or 31.05 lbs. per cubic foot, and the weight on May 22nd was 48 lbs. 1 oz., or 30.23 lbs., showing a loss of weight while in the laboratory at the rate of .0745 lb. per cubic foot per day.

Beam VII was tested on May 19th, 1893. In this beam the annual rings ran somewhat obliquely as in Fig. 9. Under a load of 17,615 lbs., the beam sheared longitudinally along the plane AB, Fig. 10, the distance between the ends of the portions above and below the plane of shear being 3-16ths of an inch. The plane of shear extended to a distance of 46-ins. from the end of the beam.

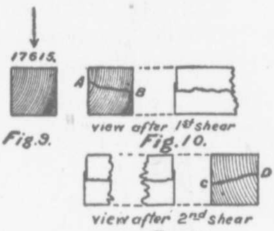


Fig. 9. Fig. 10. Fig. 11.

The maximum skin stress corresponding to this breaking weight of 17,615 lbs. is 8,712 lbs.

The co-efficient of elasticity, as determined by an increase in the deflection of .255 in. between the loads of 500 lbs. and 8,500 lbs., is 2,052,250 lbs.

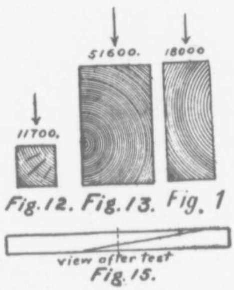
Table B shows the several readings.

Immediately after the longitudinal shear the jockey weight was run back until it indicated a load of 5,090 lbs. when the lever again floated. The weight was then gradually increased until it amounted to 11,840 lbs., when there was a second longitudinal shear along the plane CD at the other end, Fig. 11. The lap at the plane AB was now increased from 3-16ths in. to 3-10ths in., and the distance between the ends of the portions above and below the plane of shear at the other end of the beam was 3-20ths of an inch.

After this second shear the jockey weight was run back to 6,840 lbs. when the lever floated. The load was gradually increased until it amounted to 8,990 lbs., when the beam was fractured by the tearing apart of the fibres on the tension face.

On May 11th, this beam weighed 60 lbs. 4 ozs., or 40.69 lbs. per cubic foot, and the weight on May 19th was 59 lbs. 2 ozs., or 39.92 lbs. per cubic foot, showing a loss of weight in the laboratory at the rate of .09625 lb. per cubic foot per day.

Beam VIII was tested May 22nd, 1893. In this beam the annual rings were oblique as in Fig. 12. Under a load of 11,700 lbs. it failed at the support by the tearing apart of the fibres on the tension face.



The maximum skin stress due to this load is 8,382 lbs. per square inch.

The co-efficient of elasticity, as determined by an increase in the deflection of .32 in. between loads of 1,000 lbs. to 5,500 lbs., is 1,559,950 lbs.

Table B shows the several readings.

The weight of this beam on May 11th was 44 lbs., or 36.76 lbs. per cubic foot, and its weight on May 22nd was 42 lbs. 14 ozs., or 35.74 lbs. per cubic foot, showing a loss of weight in the laboratory at the rate of .0927 lb. per cubic foot per day.

Beams IX to XVI were sent to the laboratory by Mr. P. A. Peterson, chief engineer of the Canadian Pacific Railway.

Beam IX was grown on the mainland half way between Vancouver and New Westminster, in a flat country not much above the sea level. It was cut from a log 26 ins. in diameter and 34 feet in length, which was felled about the month of May, 1892. The log was floated to the mill at Vancouver, and lay in fresh water for ten months.

The timber corresponded to first quality in the market, its grain being straight and running parallel to the axis. It contained a season crack on the widest face, about 11 feet long, $3\frac{1}{2}$ ins. below the edge, and about $1\frac{1}{2}$ in. deep. The beam was tested Nov. 13th, 1893, with the annual rings as in Fig. 13, the heart of the tree being in one of the vertical faces. Under a load of 51,600 lbs. this beam failed at the support by the tearing apart at the centre of the fibres on the tension face.

The maximum skin stress corresponding to this load is 7,974 lbs. per square inch.

The co-efficient of elasticity, as determined by an increment in the deflection of .77 in. between the loads of 1,000 lbs. and 20,000 lbs., is 1,767,990 lbs.

Table C shows the several readings.

The weight of the beam was 603 lbs., or 36.49 lbs. per cubic foot on Oct. 3rd, 590 lbs. 13 ozs., or 35.76 lbs. per cubic foot on Nov. 10th, and 590 lbs. on Nov. 13th, showing a loss of weight while in the laboratory at the rate of .0195 lb. per cubic foot per day.

Beam X. This beam was tested Nov. 11th, 1893, with the annual rings as in Fig. 14. It was cut from a log 32 ins. in diameter grown on the mainland 120 miles north and west of Vancouver, on a hill-side about 100 feet above the sea-level. The log was felled in the winter of 1892–93, and was then towed to the mill, and remained in salt water six months.

The grain in this beam ran crosswise, and it failed by a cross fracture along the plane AB, Fig. 15.

The fracture occurred under a load of 18,000 lbs., corresponding to a maximum skin stress of 4,027 lbs. per square inch. The co-efficient of elasticity, as determined by an increase in the end deflections of .84-in. between the loads 1,000 lbs. and 15,000 lbs., is 1,637,806 lbs.

Table C shows the several readings.

The weight of the beam was 407 lbs. 2 ozs., or 38.94 lbs. per cubic foot on Oct. 3rd, 406 lbs. 8 ozs., or 37.80 lbs. per cubic foot on Nov. 10th, and 404 lbs. 13 ozs., or 37.79 lbs. per cubic foot on Nov. 13th, showing a loss of weight in the laboratory at the rate of .03 lbs. per cubic foot per day.

Beam XI. This beam was tested November, 7th, 1893, with the annual rings as in Fig. 16. Its history is the same as that of Beam

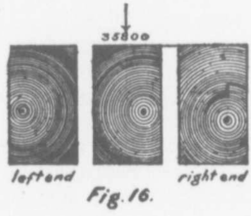


Fig. 16.

X. The timber was of a quality corresponding to first quality in the market, and the grain for the most part was parallel with the axis. It contained a few season cracks. On the tension face of the beam the fibres crossed from back to front in a distance of 3½ ft., commencing about five feet one end. The beam contained the heart of the tree, the annual rings being as in the Figure.

Under a load of 35,800 lbs. the beam failed by the tearing apart of the fibres on the tension face.

The maximum skin stress corresponding to this load is 5,698 lbs. per square inch.

The co-efficient of elasticity, as determined by an increase in the deflection of .545 ins. between the loads of 2,500 and 15,500 lbs., is 1,770,563 lbs.

Table D shows the several readings.

The weight of the beam was 595 lbs. 2 ozs., or 37.76 lbs. per cubic foot on October 3rd, and 583 lbs., or 36.99 lbs. per cubic foot on Nov. 14th, showing a loss of weight in the laboratory at the rate of $.018\dot{3}$ lbs. per cubic foot per day.

Table D shews the several readings.

The time occupied by the test was 29 minutes.

Beam XII was tested Nov. 18th, 1893, with the annual rings as in Fig. 17. This beam was cut from a log 28 ins. in diameter, grown probably about 30 feet above the sea-level at Port Grey, about eight miles from Vancouver. The tree was felled in August, 1892; it remained in salt water nine months, being alternately wet and dry according to the tide; it was then towed to the mill and cut up.

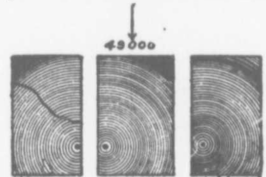


Fig. 17.

The grain was straight and parallel to the axis, and the timber was of good quality corresponding to first quality in the market. It shewed several knots of medium size and a few season cracks. The beam contained the heart of the tree, the annual rings being as in Fig. 17.

Under a load of 49,000 lbs. the beam failed by shearing longitudinally along the season crack AB.

Under this load the maximum skin stress is 7,645 lbs. per sq. in.

The co-efficient of elasticity as determined by an increment in the deflections of .545 ins. between the loads 2,500 lbs. and 15,000 lbs. is 1,678,300 lbs.

Table D shews the several readings.

The time occupied by the test was 37 minutes.

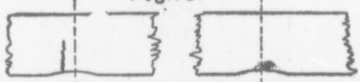
The weight of the beam was 572 lbs., or 35.65 lbs. per cubic foot on Oct. 3rd, and 558 lbs. 4 ozs., or 34.79 lbs. per cubic foot on Nov. 17th, showing a loss of weight in the laboratory at the rate of .0191 lbs. per cubic foot per day.

Beam XIII. The history of this beam is the same as that of Beam IX. The beam was tested on Nov. 17th, 1893. The heart of the tree was in one of the faces, the annual rings being as in Fig. 18.

The timber was in good condition and of a quality corresponding to first quality in the market; there were small season cracks along the back of the beam, in the neighbourhood of the neutral plane, and there were also small season cracks along the whole of the front about 3-ins. above the face in compression.

Under a load of 29,300 lbs. this beam failed by the crippling of the fibres on the compression face, commencing at a small knot at the back, Fig. 19.

Fig. 18.



front and back views, at centre, after test.

Fig. 19.

The maximum skin stress corresponding to this load is 6,912 lbs. per square inch.

The co-efficient of elasticity as determined by an increase in the deflection of .805 ins. between the loads 1,000 lbs. and 13,000 lbs. is 1,643,193 lbs.

Table E shows the several readings.

The beam weighed 381 lbs. 15 oz., or 34.56 lbs. per cubic foot on Oct. 3rd, and 375 lbs., or 34.13 lbs. per cubic foot on Nov. 15th, showing a loss of weight in the laboratory at the rate of .01 lb per cubic foot per day.

The time occupied by the test was 45 minutes.

Beam XIV is in reality Beam XIII re-tested, the second test having been made Dec. 2nd, 1893. The beam was replaced in the machine with the crippled side reversed so as to be in tension. The load was then gradually applied until it amounted to 17,600 lbs., when the beam failed on the tension side by the tearing apart of the fibres along the surface at which the crippling took place on the previous test.

The maximum skin stress corresponding to this load is 4,082 lbs. per square inch as compared with 6,912 lbs. per square inch in the first test. The co-efficient of elasticity, as determined by an increment in the deflection of .51 ins. between the loads of 1,000 lbs. and 8,000 lbs., is 1,513,950 lbs, as compared with 1,643,193 lbs. in the first test.

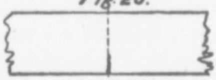
Table E shews the several readings.

This experiment therefore shews that although the beam may have been crippled by undue pressure, it still retained a large amount of strength as well as elasticity.

Table E gives the several readings.

Beam XV. This beam was tested Nov. 18th, 1893. The timber was excellent in quality, equal to first quality in the market, clear and straight grained and free from knots. Its history is the same as that of Beam XII. The annual rings were oblique as in Fig. 20.

Fig. 20.



front view, at centre, after test

Fig. 21.

Under a load of 37,000 lbs. the beam failed by the crippling of the fibres on the compression face, Fig. 21.

The maximum skin stress corresponding to this load is 8,020 lbs. per square inch.

The total compression of the material was .34 in., and the maximum skin compressive stress, taking 1,466 in. as the effective depth, is 8,189 lbs. per sq. in., the corresponding skin tension stress being 8,577 lbs. per in. sq.

Assuming the ordinary law to hold good for the whole of the effective depth, the maximum skin stress would be 8,511 lbs. per sq. in.

The co-efficient of elasticity as determined by an increment in the deflection of .755 ins. between the loads, 2,000 lbs. and 18,000 lbs., is 1,989,400 lbs.

Table E shews the several readings.

The time occupied by the test was 30 minutes.

The weight of the beam was 445 lbs. 6 ozs., or 39.99 lbs. per cubic foot on Oct. 3rd, and 433 lbs. 13 ozs., or 38.92 lbs. per cubic foot on Nov. 17th, showing a loss of weight in the laboratory at the rate of .0237 lbs. per cubic foot per day.

Beam XVI. This is really Beam XV re-tested, the second test having been made on Dec. 8th, 1893. In the first test the beam had failed by crippling on the compression face; the beam was now reversed, and under a load of 25,580 lbs. it failed by the tearing apart of the fibres on the tension face along the surface at which the crippling had previously taken place. The tensile fracture extended 2 inches below the skin. The jockey weight was now run back until the lever again floated, and the load was gradually increased until it amounted to 32,000 lbs., when the beam fractured a second time on the tension side, the fracture extending to a depth of 5 inches below the skin. The first fracture was accompanied by a longitudinal opening (as in Fig.) about 60 inches in extent. A second longitudinal opening, also about 60 inches long, occurred at the second fracture.

The maximum skin stress corresponding to the breaking load of 25,580 lbs. is 5,466 lbs. per square inch.

The co-efficient of elasticity, as determined by an increment in the deflection of .54 ins. between the loads of 1,000 lbs. and 11,500 lbs., was 1,825,450 lbs.

Table E gives the several readings.

The weight of the beam was reduced to 428 lbs., or 38.40 lbs. per cubic foot, showing a loss between the test on Nov. 17th and that on Dec. 8th at the rate of .02476 lbs. per cubic foot per day.

Beams XVII to XXI were sent to the testing laboratories by the

British Columbia Mills Timber & Trading Company through Mr C. M. Beecher. The whole of these timbers were cut on the coast section of British Columbia. The trees from which Beams XVII, XVIII, XX and XXI were cut, were felled during the summer of 1893, and came from Hartney's Camp, Seymour Creek, while Beam XIX was cut from a tree felled in the spring of 1894, and came from Rowling's Camp, Salmon Arm.

Beam XVII was tested June 24th, 1894. This beam was coarse grained, the grain running very nearly parallel with the axis, and it contained a number of small knots on the compression side. It was cut from the heart of the tree, and was tested with the annual rings as in Fig. 22.

Under a load of 48,600 lbs. it failed by the tearing apart of the fibres on the tension face, the corresponding maximum skin stress, neglecting the compression of the timber, being 4,906 lbs. per square inch. The tensile fracture was followed immediately by a longitudinal shear, coincident with the neutral plane at the centre of the beam, and extending for a distance of 8 feet from the end, Fig. 25. The distance between the portions of the beam above and below the plane of shear at the end was 3-10ths of an inch. Figs. 23 and 24 are sections at the end and at the centre showing the nature of the fractures.

The total compression of the material was 1.83 ins., and the maximum skin compressive stress, taking 13.295 ins. as the effective depth, is 5,193 lbs. per square inch, the corresponding stress in the tension skin being 6,851 lbs. per square inch.

Assuming the ordinary law to hold good for the whole of this effective depth, the maximum skin stress would be 6,350 lbs. per square inch.

The co-efficient of elasticity as determined by an increment in the deflection of .335 ins. between the loads 10,000 lbs. and 30,000 lbs., is 1,259,600 lbs.

Table F gives the several readings.

The weight of the beam, when shipped from Vancouver about April 21st, was 428 lbs., or 37.21 lbs. per cubic foot; on reaching the Laboratory on June 9th, the weight was found to be 411 lbs. 10 ozs., or 35.78 lbs. per cubic foot, and on the day of the test, namely, June 24th, the weight was 404 lbs. 8 ozs., or 35.17 lbs per cubic foot, showing a loss at the rate of .02918 lb. per cubic foot per day between Vancouver and the laboratory, and a loss at the rate of .04067 lb. per cubic foot per day while in the laboratory.

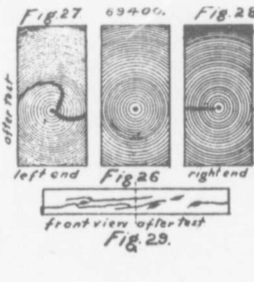
Beam XVIII. This beam was coarse grained, and contained several large and small knots; it was cut from the heart of the tree. It was tested Sept. 28th, 1894, with the annual rings as in Fig. 26.

The load on the beam was gradually increased to 12,000 lbs. The beam was now gradually relieved from strain until the load had been reduced to 1,000 lbs. without showing any set. The load was again gradually increased from 1,000 lbs. up to 19,000 lbs., when the beam was again relieved from load and the readings were taken for each difference of 1,000 lbs.

When the load had been reduced to 1,000 lbs., the deflection at the centre was observed to be .015 in. as compared with .005 in. in the forward movement, and as soon as the beam was relieved of this 1,000 lbs., it returned to its initial condition without showing any set whatever.

The time occupied by the first loading was 10 minutes, by the second loading 12 minutes, and by the relieving from load 8 minutes.

In the final test the load was gradually increased from nil until it amounted to 69,400 lbs., when the beam failed by shearing longitudinally, the shear being immediately followed by the tearing apart of the fibres on the tension face, Figs. 27, 28, 29.



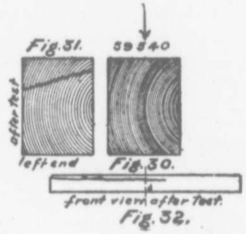
The maximum skin stress corresponding to the breaking load was 5,196 lbs. per square inch.

The co-efficient of elasticity, as deduced from an increment in the deflection of 1-10th of an inch between the loads of 2,000 lbs. and 12,000 lbs., being 1,329,900 lbs.

Table F gives the several readings.

The weight of the beam at the date of shipment from Vancouver, April 21st, was 512 lbs., or 39.08 lbs. per cubic foot. On reaching the laboratory, on June 9th, this weight was 492 lbs. 10 ozs., or 37.60 lbs. per cubic foot, and the weight on Sept. 25th was 466 lbs. 6 ozs., or 35.59 lbs. per cubic foot, showing a loss in weight between Vancouver and the laboratory at the rate of .0302 lb. per cubic foot per day, and a loss of weight in the laboratory at the rate of .0181 lb. per cubic foot per day.

Beam XIX. This beam was of exceptionally good quality, with clear close grain and no knots. It was tested Oct. 2nd, 1894, with the annual rings nearly vertical, as in Fig. 30.



The load on the beam was gradually increased up to 16,000 lbs. when it was gradually relieved from load, the readings being taken for each diminution of 4,000 lbs. The corresponding readings are indicated in Table F.

When it was completely relieved from load, the scales showed readings of .005 in. at the centre, .001 in. and .003 in. at the ends. These readings were probably due to inequalities in the timber or a possible sliding of the scales, as the beam showed no evident sign of set.

The load was again immediately increased gradually from nil until it amounted to 59,540 lbs., when the beam failed by longitudinal shear, followed by the splintering of the upper edges on the tension side, Figs. 31, 32. Fracture was also indicated by the crippling of the fibres on the compression side taking place between 58,000 and 59,540 lbs.

The distance between the portions of the beam above and below the plane of shear at the end was .36 in. as in the figure.

The maximum skin stress corresponding to the breaking load is 9,043 lbs. per square inch.

The co-efficient of elasticity, as deduced by an increase in the deflection of .3 in. between the loads of 2,000 lbs. and 16,000 lbs., is 1,934,-600 lbs.

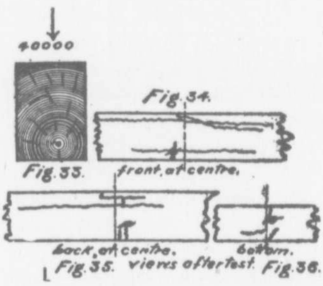
Table F shows the several readings.

The time occupied by the first loading was $10\frac{1}{2}$ mins., by the relieving from the load $6\frac{3}{4}$ mins., and by the second loading from nil to the max., $15\frac{1}{2}$ mins.

The weight of this beam on April 21st, the date of its shipment from Vancouver, was 410 lbs., or 44.99 lbs. per cubic foot. On reaching the laboratory the weight was 392 lbs. 8 ozs., or 43.07 lbs. per cubic foot, and the weight on Oct. 2nd, the date of the test, was 375 lbs. 10 ozs., or 41.22 lbs. per cubic foot, showing a loss of weight at the rate of .0392 lb. per cubic foot per day between Vancouver and the laboratory, and a loss at the rate of .0161 lb. per cubic foot per day while in the laboratory.

Beam XX. This beam was cut from the heart of the tree, and was tested Nov. 3rd., 1894, with the annual rings as in Fig. 33.

It was coarse grained, the grain being very nearly parallel with the axis, and contained a number of knots.



The load was gradually increased until it amounted 12,000 lbs., and at this point the beam was gradually relieved from load, readings being taken for every diminution of 2,000 lbs. When the load had been reduced to 500 lbs., the reading at the centre was .001 in., probably due to a movement of the scale. The load was again gradually increased until it amounted to 40,000 lbs., when the beam failed by the crippling of the fibres on the compression side in the neighbourhood of a small knot $1\frac{1}{4}$ in. above the compression face, Figs. 34, 35, 36. The crippling extended about 4 ins. above this face. The load was still gradually increased until it amounted to 49,600 lbs., when the beam again failed by the tearing apart of the fibres on the tension face.

The maximum skin stress corresponding to the load of 40,000 lbs., and disregarding the compression of the timber, is 6,559 lbs., and the skin stress corresponding to the load of 49,600 lbs. is 8,127 lbs. per square inch.

The total compression of the timber was .345 ins., so that taking the effective depth under this load to be 11.655 ins., the maximum skin compressive stress would be 6,710 lbs. per square inch, the corresponding skin tension stress being 7,125 lbs. per square inch.

Assuming the ordinary law to hold good for the whole of the effective depth, the maximum skin stress would be 6,936 lbs per square inch.

The co-efficient of elasticity, as deduced from a change in the deflection of .22 in. between the loads 4,000 lbs. and 12,000 lbs., both forwards and while being relieved from load in the first reading, and also during the second loading, is 1,571,150 lbs.

Table G shows the several readings.

The weight of this beam when shipped from Vancouver, April 21st, was 349 lbs, or 41.16 lbs. per cubic foot ; when delivered at the laboratory on June 9th, it weighed 329 lbs., or 36.70 lbs. per cubic foot, and on Nov. 3rd it weighed 311 lbs. 6½ ozs., or 34 92 lbs. per cubic foot, showing a loss of weight between Vancouver and the laboratory at the rate of .091 lb. per cubic foot per day, and a loss while in the laboratory at the rate of .0121 lb. per cubic foot per day.

The time occupied by the test was 26 mins.

Beam XXI. This beam was tested Nov. 3rd, 1894, with the annual rings as in Fig. 37.

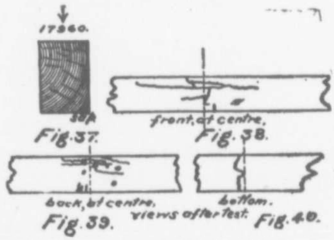


Fig. 37. Fig. 38. Fig. 39. Fig. 40.

The load upon the beam was gradually increased until it amounted to 6,000 lbs, when it was gradually relieved of oad, at the rate of 1,000 lbs. for each observation, and the beam returned to its initial condition without showing any sign of set. The load was again gradually increased until it amounted to 17,960 lbs., when a sharp fracture took place by the tearing apart of the fibres on the tension side, and this was accompanied by a simultaneous crippling of the fibres on the compression side, Figs. 38, 39, 40.

The maximum skin stress corresponding to the load of 17,960 lbs. is 7,787 lbs. per square inch.

The total compression of the timber at the centre was .16 in., so that, taking the effective depth at the centre to be 8.82 ins., the maximum skin compressive stress at the point of fracture is 7,901 lbs. per square inch, the corresponding skin tensile stress being 8,221 lbs. per sq. in.

Assuming the ordinary law to hold good for the whole of the effective depth, the max. skin stress would be 8,100 lbs. per sq. in.

The co-efficient of elasticity, as deduced by a change in the deflection of .48 in. between the loads of 1,000 lbs. and 6,000 lbs., during the first loading, and while being relieved of load, is 1,588,400 lbs.

Table G shows the several readings.

The weight of this beam when shipped from Vancouver, April 21st, was 164 lbs., or 38.86 lbs. per cubic foot; when received at the laboratory on June 9th, the weight was 151 lbs. 4 ozs., or 33.02 lbs. per cubic foot, and on Nov. 13th, the date of test, the weight was 139 lbs. $10\frac{1}{2}$ ozs., or 30.83 lbs. per cubic foot, showing a loss of weight between Vancouver and the laboratory at the rate of .1192 lbs. per cubic foot per day, and a loss of weight while in the laboratory at the rate of .0149 lbs. per cubic foot per day.

The time occupied by the test was $18\frac{1}{2}$ mins.

OLD DOUGLAS FIR.

Beams XXII–XXV were sent to the laboratory by Mr. P. A. Peterson, Chief Engineer of the Canadian Pacific Railway.

These beams were four old stringers taken from trestles numbered 428, 35, 316 and 789.

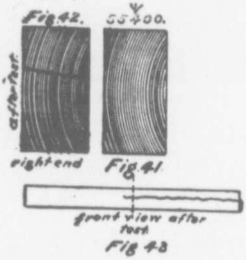
Trestle 428 is about half way between Cisco Cantilever Bridge and Lytton. It was erected in the early summer of 1884, and the timbers had consequently been in position for nine years. It is in a dry country, with very little rainfall, and subject to a hot sun in summer. The stringer from this structure was cut out of a log probably grown on a flat about three miles west of Hope, where most of the trees were windshaken.

Trestle No. 35 is about one mile west of Port Moody, and was built in the early spring of 1887, so that the stringer was in position for a period of $6\frac{1}{2}$ years in a place subject to the heaviest rainfall in the province. The stringer was cut from a log most probably grown at Point Grey, about eight miles from Vancouver.

Trestle No. 316 is two miles east of Spuzzum. The stringer from this trestle was cut from a log grown on a bench near Spuzzum about 500 feet above the sea-level. It was prepared and framed in 1881, and erected in 1882, so that it was eleven years in position in a district with a climate similar to that of Nova Scotia. As the railway here runs north and south, the sun had not the same effect upon the stringers as on other parts of the line.

Trestle No. 789 is on Kamloops Lake, six miles east of Savona, and was erected in the spring of 1885, so that the timbers had been in service for a period of eight years. The neighbourhood is dry, but the trestle, being situated under a high bluff, is protected from the afternoon sun. The stringer from this structure was cut out of a log probably grown about three miles west of Hope, at the same place as the timbers used in structure No. 428.

Beam XXII from Trestle 428 was tested Nov. 25th, 1893, with the annual rings as in Fig. 41



There were two vertical 1 in. bolt holes in the timber,—one near the centre and one at the end. There were also several season cracks in the timber, one being somewhat large.

The load upon the beam was gradually increased until it amounted to 55,400 lbs., when the beam failed by a longitudinal shear, as in Figs. 42, 43.

The distance between the portions of the beam above and below the plane of shear at the end was $\frac{3}{8}$ths of an inch.

The maximum skin stress corresponding to the breaking load is 7,086 lbs. per square inch.

The total compression of the timber at the centre was .63 in., so that, taking the effective depth at 15.0575 ins., the maximum skin compressive stress is 7,264 lbs. per square inch, the corresponding tensile skin stress being 7,898 lbs. per square inch.

Assuming the usual law to hold good for the whole of the effective depth, the maximum skin stress would be 7,382 lbs. per square inch.

The co-efficient of elasticity, as deduced by an increase in the deflection of .39 in. betweeen the loads of 2,000 lbs. and 20,000 lbs., is 1,639,500 lbs., while it is 1,691,620 lbs. for an increment in the deflection of .42 in. between the loads 2,000 lbs. and 22,000 lbs.

Table H gives the readings under the several loads.

The weight of the beam on the day of test was 33.75 lbs. per cubic foot, and the total weight on Oct. 3rd was 438 lbs. 7 ozs.

Beam XXIII from Trestle No. 789 was tested Nov. 28th, 1893, with the annual rings as in Fig. 44, and showing the heart in one of the faces.

Fig 45.

Fig 44.

Fig 46.

The load upon the beam was gradually increased until it amounted to 47,560 lbs., when the beam failed by the tearing apart of the fibres on the tension face, which was immediately followed by a longitudinal shear, as in Figs. 45, 46.

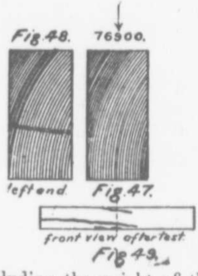
The maximum skin stress corresponding to the load of 47,560 lbs. is 7,339 lbs.

The co-efficient of elasticity, as deduced from an increment of .66 in. in the deflection between the loads of 2,000 lbs. and 22,000 lbs., is 1,878,950 lbs.

Table I shows the readings under the various loads.

The total weight of the beam on Oct. 3rd was 654 lbs. 12 ozs., or 38.95 lbs. per cubic foot; the total weight on Nov. 28th, the date of test, was 549 lbs. $8\frac{1}{2}$ ozs., or 38.59 lbs. per cubic foot, showing a loss of weight in the laboratory at the rate of .00643 lbs. per cubic foot per day. Estimating the weight of this beam from a solid block cut out of the beam, it was found to be 39.13 lbs. per cubic foot, or .54 lb. per cubic foot heavier than the weight deduced from the total weight of the whole beam.

Beam XXIV from Trestle No. 35. This beam was tested Nov. 25th, 1893, with the annual rings as in Fig. 47. It contained two vertical $\frac{3}{4}$ in. bolt holes about half way between the centre and ends, and a few knots of average size appeared on the face. It also contained several season cracks.



The initial load, including the weight of the beam, was 5,000 lbs., and the load was gradually increased up to 41,000 lbs., when the material at one end of the beam was crushed in. The ends of the beam were found to be very much the worse for wear and in a rotten condition. Releasing the beam from load the ends were sawn off and the beam was replaced at 9 ft. centres, when the load was gradually increased until it amounted to 76,900 lbs. Under this load the beam failed by longitudinal shear, which was accompanied by a certain amount of crippling of the fibres on the compression side of the centre, as in Figs. 48, 49.

The maximum skin stress corresponding to the breaking load of 76,-900 lbs. was 6,135 lbs. per square inch.

The total compression under a load of 41,000 lbs. at the centre was 1.7 in., and taking the effective depth of the beam to be 14.5-ins., the corresponding maximum skin compressive stress is 6,495 lbs. per square inch, the corresponding skin tensile stress being 8,221 lbs. per square inch.

Assuming the ordinary law to hold good for the whole of the effective depth, the maximum skin stress would be 7,662 lbs. per square inch.

The co-efficient of elasticity, as determined by an increase in the deflection of .16 in. between the loads of 11,000 and 22,000 lbs., is 1,199,741 lbs.; as determined by an increment of the deflection of .33 in. between the loads 10,000 lbs. and 32,000 lbs., it is 1,163,384 lbs.; and as deduced from an increment in the deflection of .29 in., the mean between .285 in. and .295 in., the increments between the loads of 5,000 and 25,000 lbs. and 10,000 and 30,000 lbs. respectively, it is 1,203,500 lbs.

Table H shows the several readings.

The total weight of the beam on Nov. 25th, the date of test, was 331 lbs. 9 ozs., or 32.8 lbs. per cubic foot. After cutting off the ends, the weight of a length of 9 feet was 262 lbs. 5 ozs., or 33.4 lbs. per cubic foot. The total weight of the beam on October 3rd was 339 lbs. 9 oz.

Beam XXV from Trestle 316. This beam was tested Nov. 28th, 1893, with the annual rings as in Fig. 50, and showing the heart on one of the faces.

It contained one vertical bolt hole, several knots, and many season cracks. The grain was straight.

The load upon the beam was gradually increased until it amounted to 42,900 lbs., when a large splinter broke off on the tension face, and the beam failed by longitudinal shear, as in Figs. 51, 52.

The maximum skin stress corresponding to this breaking load is 4,613 lbs. per square inch.

The co-efficient of elasticity, as determined by an increment in the deflection of .335 in. between the loads of 4,000 lbs. and 20,000 lbs., is 949,720 lbs.

Table I shows the readings for the several loads.

The total weight of the beam on October 3rd was 422 lbs., or 34.44 lbs. per cubic foot, and on Nov. 28th, the date of test, the weight was 406 lbs., or 33.11 lbs. per cubic foot, showing a loss of weight in the laboratory at the rate of .237 lbs. per cubic foot per day.

The time occupied by the test was 30 minutes.

The following table gives a summary of the results obtained for Douglas Fir :—

BEAM.	Dimensions in inches.	Weight in lbs. per cubic foot at date of test.	Maximum skin stress in lbs. per sq. in.	Co efficient of elasticity in lbs.
	NEW TIMBER, SPECIALLY SELECTED.			
	l × *d* × *b*			
III.	66 × 5.375 × 4 125		10,441	2,178,100
XIX.	138 × 12.1 × 9.1	41.22	9,043	1,934,500
VII.	69 × 6 × 5.8125	39.92	8.712	2,044,115
XV.	198 × 15 × 6.125	38.92	8,020	1.989,400
	NEW TIMBER, FIRST QUALITY.			
	l × *d* × *b*			
X	198 × 14.875 × 6	37.80	4,027	1,629,616
XI	204 × 14.875 × 8.6875	36.99	5,698	1,770,563
IX	204 × 14.875 × 9	35.76	7,694	1,764,939
VIII	69 × 5.125 × 5.5	35.74	8,382	1,584,692
XVIII	138 × 17.8 × 8.76	35.59	5.196	1,329,900
XVII	138 × 15.125 × 9.	35.17	4.907	1,259,600
XX	138 × 12. × 8.88	34.92	6,559	1,571,150
XII	204 × 14.875 × 8.8125	34.79	7,645	1.673,300
XIII	204 × 14.75 × 66	34.13	6,912	1,643,193
XXI	138 × 8.98 × 5.95	30.83	7,784	1,588,400
VI	69 × 6.125 × 6	30.23	7,116	1,489,215
I	96 × 12.125 × 9.		4,897	1,138,900
II	66 × 12.125 × 5.625		4,378	1,146,900
V	69 × 9.125 × 5.	29.18	5,869	946,270
IV	69 × 9.125 × 5.	28.27	4,156	926,500
	OLD TIMBER.			
	l × *d* × *b*			
XXIII	186 × 14.35 × 8.78	38.59	7,339	1,878,950
XXII	162 × 15.6875 × 7.75	33.75	7,086	1,665,560
XXV	144 × 15.65 × 8.2	33.11	4,613	949,720
XXIV	132 × 16.2 × 7 75	32.8	6,135	1,201,620

The following data may be adopted in practice :—

In the case of specially selected timber, free from knots, with sound clear and straight grain, and cut out of the log at a distance from the heart :

Average weight in lbs. per cubic foot = 40.

Average co-efficient of elasticity in lbs. per sq. in. = 2,000,000.

Average maximum skin stress in lbs. per square inch = 9,000.

Safe working skin stress in lbs. per square inch = 3,000 lbs.

In the case of first quality timber, such as is ordinarily found in the market:

Average weight in lbs. per cubic foot = 34.

Average co-efficient of elasticity in lbs. per square inch = 1,430,000.

Average maximum skin stress in lbs. per square inch = 6,000.

Safe working skin stress in lbs. per square inch = 2,000.

In specifying these data it will be observed that 3 is adopted as the factor of safety. Upon this hypothesis the factor of safety for the stick giving the minimum skin stress is more than 2, and this, in the opinion of the author, is an ample factor for a material which experience and all experiments show, may be strained without danger very nearly up to the point of fracture.

Further, the results obtained in the experiments with the old stringers shew that the strength of the timber had been retained to a very large extent, and that the rotting had not extended to such a depth below the skin as to sensibly affect the efficiency of the sticks, which still possessed ample strength for the work they were designed to do.

Thus in Beam XXII a diminution in the skin stress of 1,058 lbs. per square inch, which is equivalent to a diminution in the effective depth of $\frac{15.6875 \times 1058}{2 \times 7058} = 1.076$ ins. would still leave 6,000 lbs. per square inch as the skin stress. Thus if the rotting had extended to depth of 1.176 ins., the factor of safety would still remain 3.

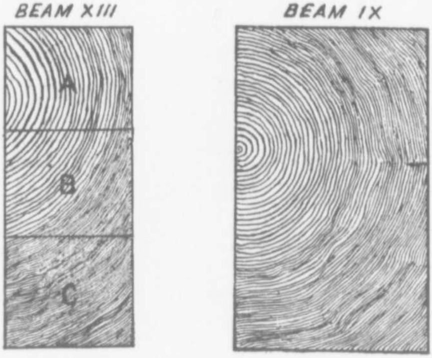
If 2 is adopted as the factor of safety, and, in the opinion of the author, 2 is an ample factor for the great majority of cases, the rotting might extend without danger to a depth of 3.398 ins.

In the case of Beam XXV, which is the old stringer giving the least co-efficient of strength, namely, 4,613 lbs. per square inch, taking 2 as the factor of safety, the effective depth might be diminished by an amount of $\frac{15.65 \times 613}{2 \times 4613} = 1.04$ ins. and rot might safely extend to this depth.

Again, it will be observed that the skin stress and the elasticity are subject to a wide variation. This variation is due to many causes, of which the most important are the presence of knots, obliquity of grain, and, more than all, the locality in which the timber was grown, the original position of the stick in the log from which it was cut, and the proportion of hard to soft fibre, or of the summer to the spring growth.

The tensile shearing and compressive experiments upon specimens cut out of different parts of the same log all shew that the timber near the heart possesses much less strength and stiffness than the timber at a distance from the heart.

The accompanying photograph is given to show the variation of



thickness in the growth rings from the heart outwards, and a careful study of the results obtained up to date would seem to indicate that the best classification defining the strength of the timber would be found by dividing the section of a log into three parts by means of two circles, with the heart as the centre, and by designating the central portion as third quality, the portion between the two circles as second quality, and the outermost portion as first quality.

A most interesting paper on the structural characteristics of Douglas Fir from a botanical standpoint was read by Professor Penhallow, F.R.S.C., at the meeting of the Royal Society of Canada in Ottawa, in 1894, in connection with a paper by the author on the strength of the timber.

RED PINE.

Beams XXVI to XXXIII were sent to the laboratory by Messrs. McLachlin Bros., of Arnprior.

These beams were not specially selected, but were the ordinary scantlings in the market. They were cut from logs felled in February or March, 1893, in the neighbourhood of the Bonnechère River, Nipissing

District, County Renfrew. The logs remained in the water from April until October, when they were sent to the mill, where they were sawn up and piled.

Beam XXVI. This beam was cut from the heart of the tree, and was tested March 13th, 1894, with the annual rings, as in Fig. 53.

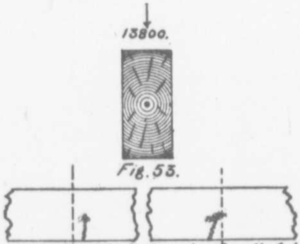


Fig. 53.

front and back views, at centre, after test.

Fig. 54. Fig. 55.

The load upon the beam was gradually increased until it amounted to 13,800 lbs., when the beam failed by the crippling of the fibres on the compression face, Figs. 54, 55. The load was still further increased until complete fracture took place by the tearing apart of the fibres on the tension face under a load of 17,170 lbs. The crippling was in line with a knot running through the timber from back to fornt, as in the Figure.

The maximum skin stress corresponding to the load of 13,800 lbs. is 3,937 lbs. per square inch.

The total compression of the timber at the centre was .2 in., so that, taking the effective depth as 13.05, the maximum skin compressive stress would be 3,994 lbs. per sq. in., the corresponding skin tensile stress being 4,119 lbs. per square inch.

Assuming the ordinary law to hold good for the whole of the effective depth, the maximum skin stress would be 4,059 lbs. per square inch.

The co-efficient of elasticity, as determined by an increment in the deflection of .885 in. between the loads 1,000 and 8,000 lbs., is 1,235,-000 lbs., and as determined by an increment in the deflection of ·5 in between the loads 2,000 and 6,000 lbs., is 1,248,990 lbs.

Table K shows the several readings.

The weight of this beam on March 10th was 392 lbs. 2 ozs., or 37.56 lbs. per cubic foot, and on March 13th it was 379 lbs. 4 ozs., or 36.39 lbs. per cubic foot, showing a loss of weight in the laboratory at the rate of .39 lb. per cubic foot per day.

Beam XXVII was tested April 5th, 1894, with the annual rings as in Fig. 56. The beam was cut from the heart of the tree, and the darkened portion in the Figure was sapwood.

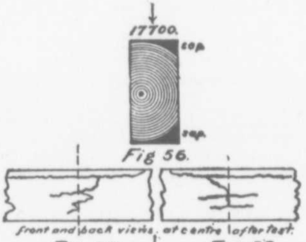


Fig 56.

Fig. 57. Fig. 58.

The load upon the beam was gradually increased until it amounted to 17,700 lbs., when the beam failed by the tearing apart of the fibres on the tension face, Figs. 57, 58, at a resin pocket, the fracture showing a fine resinous surface.

The maximum skin stress corresponding to the breaking load in 5,219 lbs. per square inch.

The total compression of the timber at the centre was .34 in., so that taking 12.785 ins. as the effective depth, the maximum skin compressive stress would be 5,411 lbs. per square inch, the corresponding skin tensile stress being 5,707 lbs. per square inch.

Assuming the ordinary law to hold good for the whole of the effective depth, the maximum skin stress would be 5,501 lbs. per square inch.

The co-efficient of elasticity, as deduced from an increment in the deflection of .7 in. between the loads 1,500 lbs. and 7,500 lbs., is 1,418,-500 lbs.

Table K gives the several readings.

The total weight of the beam on March 10th was 46 lbs. 12 ozs., or 41.51 lbs. per cubic foot; the total weight on April 5th, the date of test, was 397 lbs. 4 ozs., or 36.50 lbs. per cubic foot, showing a loss of weight while in the laboratory at the rate of .192 lbs. per cubic foot per day.

Beam XXVIII. This beam was cut from the heart of the tree, and was tested April 20th, 1894, with the annual rings as shown in Fig. 59.

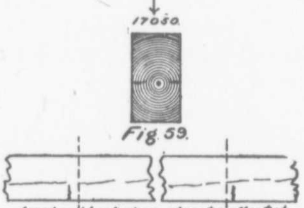


Fig 59.

front and back views, at centre after test.

Fig 60. Fig 61.

The load upon the beam was gradually increased until it amounted to 17,050 lbs., when the beam failed by the crippling of the fibres on the compression face, Figs. 60, 61. The load was still increased until under 19,140 lbs. the beam again failed by the tearing apart of the fibres on the tension face.

The maximum skin stress corresponding to the load under which crippling took place is 6,752 lbs. per square inch.

The total compression of the beam under a load of 17,050 lbs. was .24 in., so that taking the effective depth to be 11.01 ins., the corresponding maximum skin compressive stress would be 6,886 lbs. per square inch, the corresponding skin tensile stress being 7,193 lbs. per square inch.

Assuming the usual law to hold good for the whole of the effective depth, the maximum skin stress would be 7,050 lbs. per square inch.

The co-efficient of elasticity, as determined by an increase in the deflection of 1.435 in. between the loads of 2,000 and 12,000 lbs., is 1,786,000 lbs.; it is 1,858,400 lbs., as determined by an increment in the deflection of .81 in. between the loads 3,500 and 9,500 lbs., and is 1,681,100 lbs., as determined by an increment in the deflection of 1.135 in. between the loads of 2,000 and 10,000 lbs

Table K shows the several readings.

The test occupied 26 minutes.

The weight of the beam on March 10th was 379 lbs. 10 ozs., or 44.20 lbs. per cubic foot; upon April 20th, the date of test, the weight was 322 lbs. 8 ozs., or 37.55 lbs. per cub. ft., showing a loss of weight at the rate of .1622 lb. per cubic foot per day.

Beam XXIX. This beam was cut from the heart of the tree, and was tested March 13th, 1894, with the annual rings as in Fig. 62.

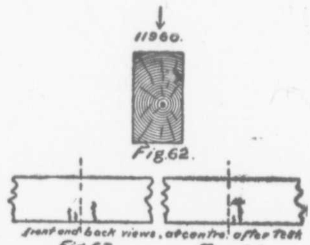


Fig. 62.
Fig. 63. Fig. 64.

The load upon the beam was gradually increased until it amounted to 11,960 lbs., when the beam failed by the crippling of the fibres on the compression face, Figs. 63, 64. The load was still further gradually increased to 12,460 lbs, when the beam was completely fractured by the tearing apart of the fibres on the tension face.

The maximum skin stress corresponding to the breaking load of 11,960 lbs. is 4,818 lbs. per square inch.

The total compression of the timber at the centre was .15 in., so that taking 11.1 in. as the effective depth, the maximum skin compressive stress would be 4,883 lbs. per square inch, the corresponding skin tensile stress being 5,016 lbs. per square inch.

Assuming the usual law to hold good for the whole of the effective depth, the maximum skin stress would be 4,949 lbs. per square inch.

The co-efficient of elasticity, as determined from an increment of .86 in. in the deflection between the loads of 1,000 and 5,000 lbs., is 1,210,100 lbs. The co-efficient of elasticity, as deduced from an increment of 1.315 in. in the deflection between the loads of 1,000 lbs. and 7,000 lbs., is 1,187,000 lbs.

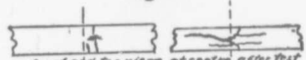
Table L shews the several readings.

The test occupied 27 minutes.

The total weight of the beam was 290 lbs., or 32.89 lbs. per cubic foot on March 10th, and 282 lbs. 6 ozs., or 32.03 lbs. per cubic foot on March 13th, showing a loss of weight in the laboratory at the rate of .2866-lb. per cubic foot per day.

Beam XXX. This beam was tested May 3rd, 1894, with the annual rings, as in Fig. 65. When the beam was placed in position, it showed an upward camber of 24 ins.

Fig. 65.



front and top views, at centre, after test

Fig. 66. Fig. 67.

The load upon the beam was gradually increased until it amounted to 5,700 lbs., when the beam failed by the crippling of the fibres on the compression face, Fig. 66, the crippling extending $2\frac{1}{2}$ ins. upwards from the skin. The load was still increased, and when it amounted to 6,580 lbs., the beam broke right across the tension face about $2\frac{1}{2}$ inches from the middle of the beam, and vertically above the second line of crippling on the compression side, Fig. 67.

The maximum skin stress corresponding to the breaking load of 5,700 lbs. is 4,634 lbs. per square inch, and the maximum skin stress corresponding to the load of 6,580 lbs. is 5,340 lbs. per square inch.

The co-efficient of elasticity is 1,322,000 lbs., as determined by an increment in the deflection of 1.69 in. between the loads of 1,000 and 5,000 lbs.; it is 1,329,900 lbs., as deduced from an increment in the deflection of .84 in. between the loads of 2,000 and 4,000 lbs.

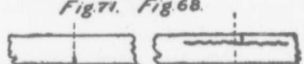
Table L shows the several readings.

The weight of this beam on May 4th, the day after the test, was 150 lbs. 11 ozs., or 30.96 lbs. per cubic foot.

Beam XXXI. This beam was tested May 4th, 1894. It was cut from the heart of the tree, and the annual rings were situated as in Fig. 68. Season cracks ran intermittently from end to end of the beam

centre after test.

Fig. 71. Fig. 68.



front and back views, at centre, after test

Fig. 69. Fig. 70.

in the neighbourhood of the neutral plane, the cracks extending radially outwards from the heart. The beam was free from knots for a distance of 7 inches on one side and 1 inch on the other, and the grain ran parallel to the axis.

The load upon the beam was gradually increased until it amounted to 6,500 lbs., when it failed by a crippling of the fibres on the compression face, Fig. 69. The crippling occurred exactly at the centre and extended 1.5 in. upwards from the skin. The load was then continued, and, when it amounted to 7,900 lbs., the beam failed by the tearing apart of the fibres on the tension face, Figs. 70, 71, and a line of crippling on the compression side timber opened upwards for a distance of about 2 ins. or $3\frac{1}{2}$ ins. The fracture on the tension side took place about $5\frac{1}{2}$ ins from the centre, and the timber opened along the annual rings for a distance of 24 ins. on each side of the centre as in the figure.

The maximum skin stress corresponding to the breaking load of 6,500 lbs. is 5,442 lbs. per square inch.

The co-efficient of elasticity, as deduced from an increment in the deflection of 1.085 ins. between the loads of 2,000 lbs. and 5,000 lbs., was 1,618,900 lbs.

Table L shews the several readings.

This beam when first placed in position, also had a camber of .35 ins. in a central length of 14 ft. 6 ins.

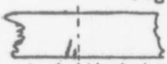
The weight of the beam on May 4th, the date of test, was 165 lbs. 6 ozs., or 34.97 lbs. per cubic foot.

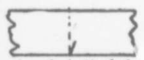
Beams XXXII to XXXV might perhaps more properly be designated 3 ins. planks.

Beam (Plank) XXXII was tested May 7th, 1894. The heart was in one of the faces, and the annual rings were situated as in Fig. 72.

Fig. 72.

Fig. 73





front and back views, of centre, after test

Fig. 74. Fig. 75.

The load upon the beam gradually increased until it amounted to 5,200 lbs., when it failed by a crippling of the fibres on the compression side. The crippling occurred about 1½ ins. away from the centre of the beam and extended upwards about 1.5 ins. The load was still increased, and when it amounted to 5,860 lbs. the beam again failed by the tearing apart of the fibres on the tension side. A line of crippling also extended upwards a further distance of about 2 ins., or about 3½ ins. from the skin.

The maximum skin stress corresponding to the breaking load of 5,200 lbs. is 6,928 lbs. per square inch.

The co-efficient of elasticity, as deduced from an increment in the deflection of 1.67 ins. between the loads 1,000 lbs. and 4,000 lbs., is 1,575,200 lbs. per square inch.

Table L shews the several readings.

The weight of this beam on May 7th, the date of test, was 102 lbs., or 31.56 lbs. per cubic foot.

Beam (Plank) XXXIII was tested May 7th, 1894, with the annual rings as shown in Fig. 73.

The load upon the beam was gradually increased to 9,250 lbs., when failure took place by the crippling of the fibres on the compression side, Figs. 74, 75. There were two lines of crippling on the front and one at the middle of the beam at the back. The crippling at the back probably occurred first, as the folding of the timber extends across the section of the beam along the central line at the lower edge, but not up to the point where the failure due to compression was apparently the greatest. In the neighbourhood of the crippling in front, the timber was clear, and the grain ran straight and parallel with the axis; at the back there were three knots, which were primarily the cause of the crippling.

When the load on the beam had been increased to 9,900 lbs., fracture occurred on the tension side.

The maximum skin stress corresponding to the breaking load of 9,250 lbs. is 6,554 lbs. per sq. in.

The co-efficient of elasticity, as determined by an increment in the deflection of .76 in. between the loads 2,600 and 6,200 lbs., is 1,618,000 lbs.

Table M shews the several readings.

The weight of the beam on May 7th, date of test, was 128 lbs. 8 ozs., or 31.87 lbs. per cubic foot.

Beam (Plank) XXXIV. This beam was tested May 8th, 1894, with the annual rings as in Fig. 76.

Fig. 76.

Fig. 77. *Fig. 78.*

The load upon the beam was gradually increased until it amounted to 5,600 lbs., when the fibres on the compression face crippled to a small extent. On still further increasing the load, the fibres on the compression face were completely crippled, Figs. 77, 78, and fracture also simultaneously occurred on the tension side when the load amounted to 8,400 lbs.

The grain of this beam was straight and parallel with the axis, and the timber was apparently free from knots for a distance of about 24 inches on each side of the centre.

The maximum skin stress corresponding to the breaking load of 5,600 lbs. is 5,079 lbs. per square inch, and the skin stress corresponding to the load of 8,400 lbs., which caused the fracture on the tension side, is 7,597 lbs. per square inch.

The co-efficient of elasticity, as deduced from an increment in the deflection of 1.14 ins. between the loads of 500 and 5,600 lbs., was 1,784,800 lbs.

Table M shews the several readings.

The weight of the beam on May 8th, date of test, was 96 lbs. 2 ozs., or 36.59 lbs. per cubic foot.

Fig. 79.

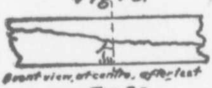


Fig. 80

Beam (Plank) XXXV was tested May 8th, 1894, with the annual rings as in Fig. 79. The heart of the tree was very nearly coincident with the axis of the beam, and the grain ran in the same direction. Season cracks occurred intermittently throughout the beam.

The load upon the beam was gradually increased until it amounted to 7,600 lbs., when the beam failed by the crippling of the fibres on the compression face, Fig. 80. The load was still increased, and well defined crippling occurred when it amounted to 10,050 lbs. When the load had reached 13,700 lbs. the beam failed by the tearing apart of the fibres on the tension face, Fig. 80.

The maximum skin stress corresponding to the breaking load of 7,600 lbs. is 4,339 lbs. per square inch.

The co-efficient of elasticity, as determined by an increment in the deflection of .92 in. between the loads of 500 and 7,600 lbs., is 1,589,250 lbs., and as determined by an increment in the deflection of .025 in. for the corresponding increase of 200 lbs. it is 1,642,900 lbs.

Table M shews the several readings.

The weight of the beam on May 8th, date of test, was 128 lbs. 12 ozs. or 37.69 lbs. per cubic foot.

The following table gives a summary of the results obtained for Red Pine:—

Beam.	Dimensions in inches.	Weight in lbs. per cubic foot at date of test.	Maximum skin stress in lbs. per sq. inch.	Co-efficient of elasticity in lbs.
	New Timber.			
	l × d × b			
XXXV.	156 × 11.15 × 3.325	37.69	4,339	1,616,075
XXVIII.	210 × 11.25 × 6.34375	37.55	6,752	1,802,633
XXXIV.	156 × 9.125 × 3.125	36.59	5,079	1,784,800
XXVII.	210 × 13.125 × 6.1875	36.50	5,219	1,418,500
XXVI.	210 × 13.25 × 6.375	36.39	3,937	1,241.950
XXXI.	174 × 7.125 × 6.21875	34.97	5,442	1,618,900
XXIX.	210 × 11.25 × 6 25	32.03	4,818	1,198,550
XXXIII.	180 × 11.125 × 3.1	31.87	6,554	1,618,000
XXXII.	180 × 8.125 × 3.1	31.56	6,928	1,575,200
XXX.	174 × 7.25 × 6.1875	30.96	4,634	1,325.950

Hence,

The average weight in lbs. per cubic foot = 34.61.

" co-efficient of elasticity in lbs. per sq. in. = 1,520,056.

" maximum skin stress " " = 5370.

If, however, the plank results are omitted,

The average weight in lbs. per cubic foot = 34.78.

" co-efficient of elasticity in lbs. per sq. in. = 1,434,747.

" maximum skin stress " " = 5137.

In general, the following data may be adopted in practice: —

The average weight in lbs. per cubic foot = 34.6.

" co-efficient of elasticity in lbs. per sq. in. = 1,430,000.

" maximum skin stress " " = 5,100.

" safe working skin stress " " = 1,700,

3 being a factor of safety.

In the accounts of the several beams it will be observed that the failures are almost invariably due to the crippling of the material on the side in compression, indicating that the tensile strength of the timber exceeds its compressive strength, and this was subsequently verified by the direct tension and compression experiments.

WHITE PINE.

Beams XXXVI and XXXVII are two pieces cut out of one large piece of square pine, made and taken out in the Gatineau Valley, Ottawa County. The timber was brought down via the Gatineau and Ottawa Rivers to Montreal, and remained in the water until late in the fall of 1892, when it was piled on the land for winter sawing.

This timber was purchased from Messrs. J. & B. Grier.

Beam XXXVI was tested February 16th, 1893, with the annual rings as in Fig. 81.

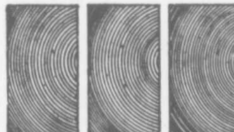


Fig. 81. Fig. 83. Fig. 84.

The load upon the beam was gradually increased until it amounted to 19,600 lbs., when it failed by the tearing apart of the fibres on the tension side.

The maximum skin stress corresponding to this load is 2,993 lbs. per square inch.

The co-efficient of elasticity, as determined by an increment in the deflection of 1.12 ins. between the loads of 5,000 and 10,000 lbs., is 503,440 lbs.; as deduced from an increment in the deflection of .84 in. between the loads of 5,000 and 12,500 lbs., is 463,768 lbs., and as deduced from an increment in the deflection of 2.13 ins. between the loads of 5,000 and 15,000 lbs., is 534,169 lbs.

Table N shows the several readings.

The weight of this beam per cubic foot on Feb. 16th was 37.25 lbs., and on March 14th, 34.78 lbs., showing a loss of weight at the rate of .095 lb. per cubic foot per day.

Beam XXXVII was tested on February 24th, 1893, with the annual rings as in Fig. 82.

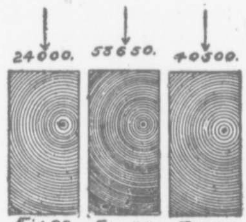


Fig. 82. Fig. 85. Fig. 86.

The load was gradually increased until it amounted to 24,000 lbs., when the beam failed by the tearing apart of the fibres on the tension face.

The maximum skin stress corresponding to this load is 3,555 lbs. per square inch.

Beams XXXVIII and XXXIX were the two ends of Beam XXXVI which was tested February 16th, 1893, the central portion containing the fracture having been cut out.

Beam XXXVIII was tested on March 14th, with the annual rings as in Fig. 83.

The load on the beam was gradually increased until it amounted to 52,450 lbs., when it failed by the tearing apart of the fibres on the tension side.

The maximum skin stress corresponding to this load is 3,075 lbs. per square inch.

The co efficient of elasticity, as determined by an increment in the deflection of .37 in. between the loads of 10,000 and 25,000 lbs., is 622,640 lbs.

Table N shows the several readings.

Beam XXXIX was tested with the annual rings as in Fig. 84.

The load was gradually increased until it amounted to 51,400 lbs., when the beam failed by the tearing apart of the fibres on the tension side.

The maximum skin stress corresponding to this load is 2,696 lbs. per square inch.

The co-efficient of elasticity, as determined from an increment in the deflection of .175 in. between the loads of 10,000 and 25,000 lbs., is 433,250 lbs.

Table N shows the several readings.

Beams XL and XLI are the two ends of Beam XXXVII which was tested on Feb. 24th, 1893, the central portion of the beam containing the fracture having been cut out.

Beam XL was tested on March 17th with the annual rings as in Fig. 85. The load was gradually increased until it amounted to 53,650 lbs., when the beam failed by the tearing apart of the fibres on the tension side.

The maximum skin stress corresponding to this load is 3,311 lbs. per square inch.

The co-efficient of elasticity, as determined by an increment in the deflection of .19 in. between the loads of 12,000 and 26,000 lbs., is 693,090 lbs.

Table N shows the several readings.

The weight of the beam per cubic foot on the day of the test was 36.13 lbs.

Beam XLI was tested on March 17th, 1893, with the annual rings as in Fig. 86. The load upon the beam was gradually increased until it amounted to 40,500 lbs., when it failed by the tearing apart of the fibres on the tension side.

The maximum skin stress corresponding to this load is 2,500 lbs. per square inch.

The co-efficient of elasticity, as deduced from an increment in the deflection of .19 in. between the loads of 10,000 lbs. and 22,000 lbs., is 519,820 lbs. per square inch.

Table N shows the several readings.

The weight of the beam on the day of test was 36.13 lbs. per cubic foot.

Beams XLII and XLVI were cut out of one large piece of square pine made on the Pettewawa, a tributary of the Ottawa, in 1888. The piece was driven over 1,300 miles, and lay in water for four years until it was taken out in the fall of 1892 and piled for winter sawing.

This timber was purchased from Messrs. Shearer & Brown.

Beam XLII was tested March 8th, 1893, with the annual rings as in Fig. 87.

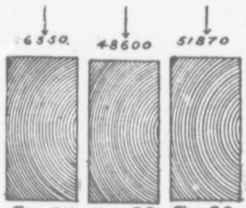


Fig. 87. Fig. 88. Fig. 89.

The load on the beam was gradually increased until it amounted to 26,350 lbs., when the beam failed by the tearing apart of the fibres on the tension side.

The maximum skin stress corresponding to this load is 3,815 lbs. per square inch.

The co-efficient of elasticity, as determined by an increment in the deflection of 1.22 ins. between the loads of 2,500 lbs. and 13,000 lbs., is 979,220 lbs.

Table O shows the several readings.

The weight of the beam per cubic foot at the date of test was 41.49 lbs.

Beams XLIII and XLIV are the two ends of Beam XLII tested March 8th, the central portion of the beam containing the fracture having been cut out.

Beam XLIII was tested March 31st, with the annual rings as in Fig. 88.

The load was gradually increased until it amounted to 48,600 lbs., when the beam failed by the tearing apart of the fibres on the tension side.

The maximum skin stress corresponding to this load is 3,000 lbs. per square inch.

The co-efficient of elasticity, as determined by an increase in the deflection of .19 in. between the loads of 10,000 and 25,000 lbs., is 649,780 lbs. per square inch.

Table O shows the several readings.

Beam XLIV was tested March 31st, 1893, with the annual rings as in Fig. 89.

The load upon the beam was gradually increased until it amounted to 51,870 lbs., when it failed by the tearing apart of the fibres on the tension side.

The maximum skin stress corresponding to this load is 3,148 lbs. per square inch.

The co-efficient of elasticity, as determined by an increment in the deflection of .19 in. between the loads of 1,000 and 25,000 lbs, is 649,780 lbs. per square inch, the same co-efficient as in beam XLIII.

Table O shows the several readings.

Beam XLV was tested March 11th, 1893, with the annual rings as in Fig. 90.

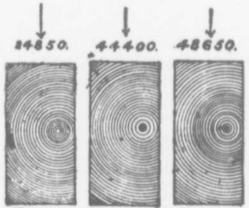


Fig. 90 *Fig. 91* *Fig. 92*

The load upon the beam was gradually increased until it amounted to 24,850 lbs., when it failed by the tearing apart of the fibres on the tension side.

The maximum skin stress corresponding to this load is 3,681 lbs per square inch.

The co-efficient of elasticity, as determined from an increment in the deflection of .81 in. between the loads of 2,500 and 12,000 lbs., is 956,540 lbs.

Table P shows the several readings.

Beams XLVI and XLVII are the two ends of Beam XLV, tested on March 11th, 1893, the central portion containing the fracture having been cut out.

Beam XLVI was tested March 30th, 1893, with the annual rings as in Fig. 91.

The load upon the beam was gradually increased until it amounted to 44,400 lbs., when it failed by the tearing apart of the fibres on the tension side.

The maximum skin stress corresponding to this load is 2,740 lbs. per square inch.

The co-efficient of elasticity, as determined by an increment in the deflection of .23 in. between the loads of 10,000 and 25,000 lbs., is 536,770 lbs.

Table P shows the several readings.

Beam XLVII was tested March 30th, 1893, with the annual rings as in Fig. 92.

The load upon the beam was gradually increased until it amounted to 48,650 lbs., when it failed by the tearing apart of the fibres on the tension side.

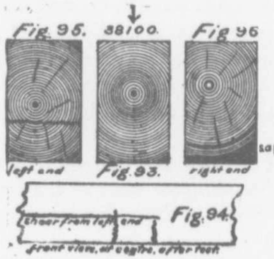
The maximum skin stress corresponding to this load is 3,003 lbs. per square inch.

The co-efficient of elasticity, as determined by an increment in the deflection of .2 in. between the loads 10,000 and 25,000 lbs., is 617,283 lbs.

Table P shows the several readings.

Beams XLVIII to L were sent to the laboratory by Mr. P. A. Peterson. These beams were purchased from the Pembroke Lumber Company, and are supposed to have been similar in quality to the timber used on the Pembroke section of the Canadian Pacific Railway.

Beam XLVIII was tested March 1st, 1894, with the annual rings as in Fig. 93. The darkened portion, Fig. 96, represents sapwood.



The load upon the beam was gradually increased until it amounted to 38,100 lbs., when the beam failed by the crippling of the material at the support on the compression side, Fig. 94. The load was still gradually increased until it amounted to 47,960 lbs., when a complete fracture took place by the tearing apart of the fibres on the tension side at the centre, and simultaneously by a longitudinal shearing throughout one-half of the length of the beam, as in Figs. 94, 95.

The maximum skin stress corresponding to the breaking load of 38,100 lbs. is 3,991 lbs. per square inch; the maximum skin stress corresponding to the load of 47,960 lbs. is 5,017 lbs. per square inch.

The total compression of the timber at the centre was .93 in., so that, taking the effective depth to be 14.3875 ins., the maximum compressive skin stress at the support would be 4,161 lbs. per square inch, the corresponding maximum tensile skin stress being 4,652 lbs. per square inch.

Assuming the usual law to hold good for the whole of the effective depth, the maximum skin stress would be 4,447 lbs. per square inch.

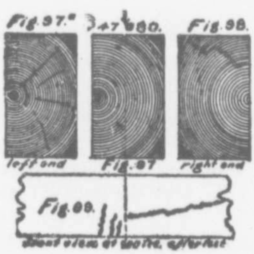
The co-efficient of elasticity, as determined by an increment in the deflection of .375 in., between the loads of 2,000 lbs. and 19,000 lbs., is 1,164,700 lbs.

Table Q gives the several readings.

The total weight of the beam on March 1st, the date of test, was 524 lbs. 10 ozs., or 41.08 lbs. per cubic foot, and on February 1st the weight was 597 lbs., or 46.73 lbs. per cubic foot, showing a loss of weight at the rate of .209 lb. per cubic foot per day.

The time occupied by the test was 48 minutes.

Beam XLIX was tested March 2nd, 1894, with the annual rings as in Fig. 97. The darkened portions represent sapwood.



The load upon the beam was gradually increased until it amounted to 47,080 lbs., when the beam failed by the tearing apart of the fibres on the tension side, accompanied simultaneously by a longitudinal shear and a crippling of the material in the compression side, Figs. 98, 99.

The maximum skin stress corresponding to the breaking load is 4,936 lbs. per square inch.

The total compression of the material at the centre was 2.8 ins., so that taking 13.095 ins. as the effective depth, the maximum skin compressive stress would be 5,156 lbs. per square inch, and the corresponding skin tensile stress would be 7,353 lbs. per square inch.

Assuming the usual law to hold good for the whole of the effective depth, 6,835 lbs. per square inch would be the maximum skin stress.

The co-efficient of elasticity, as determined by an increment of .435 in., between the loads of 3,000 and 21,000 lbs., is 1,052,600 lbs.

Table Q shows the several readings.

The weight of the beam was 525 lbs. 12 ozs., or 41.33 lbs. per cubic foot February 1st, and 473 lbs. 12 ozs., or 37.24 lbs. per cubic foot on March 2nd, showing a loss of weight at the rate of .141 lbs. per cubic foot per day.

The time occupied by the test was fifty minutes.

Beam L was tested March 10th, 1894, with the annual rings as in Fig. 100.

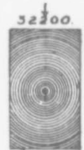


Fig. 100.

The load upon the beam was gradually increased until it amounted to 32,200 lbs., when it failed by the tearing apart of the fibres on the tension side.

The maximum skin stress corresponding to this load is 4,370 lbs. per square inch.

The co-efficient of elasticity, as deduced from an increment in the deflection of .805 in., between the loads of 1,000 and 19,000 lbs., is 1,184,240 lbs.

Table Q shows the several readings.

The weight of the beam was 509 lbs. 12 ozs. or 33.64 lbs. per cubic foot on March 10th, the date of test, and 575 lbs. 8 ozs., or 37.25 lbs. per cubic foot, on February 1st, showing a loss of weight at the rate of .0975 lb. per cubic foot per day.

OLD WHITE PINE.

Beams LI to LIII are three old white pine stringers sent to the laboratory by Mr. P. A. Peterson. These stringers had been in service since 1885, *i.e.*, for about eight years; they were removed from the trestles during the summer of 1892.

Beam LI was tested December 1st, 1893, with the annual rings as in Fig. 101.

The load upon the beam was gradually increased until it amounted to 22,730 lbs. when the beam failed by shearing, longitudinally as in Figs. 102, 103, the distance between the portions of the beam above and below the plane of shear being $\frac{1}{4}$ in.

The maximum skin stress corresponding to this load is 3,212 lbs. per square inch.

The co-efficient of elasticity, as determined by an increment in the deflection of .55 in., between the loads of 2,500 lbs. and 12,000 lbs., is 982,480 lbs.

Table R shows the several readings.

The total weight of the beam on December 1st, date of test, was 445 lbs., or 28.3 lbs. per cubic foot. The weight of a length of 14 ft. $1\frac{3}{4}$ ins. was 376 lbs., or 28.12 lbs. per cubic foot on December 2nd, and 367 lbs. 5 ozs., or 27.47 lbs. per cubic foot on December 8th, showing a loss of weight at the rate of .1083 lb. per cubic foot per day.

Beam LII was tested December 9th, 1893, with the annual rings as in Fig. 104.

The load upon the beam was gradually increased until it amounted to 26,320 lbs., when the beam failed by the tearing apart of the fibres on the tension side.

The maximum skin stress corresponding to this breaking load is 3,589 lbs. per square inch.

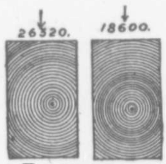


Fig 104 *Fig. 105*

The total compression of the material at the support was .37 in., so that, taking 14.85 ins. as the effective depth, the maximum skin compressive stress is 3,671 lbs. per square inch, the corresponding maximum tensile stress being 3,863 lbs. per square inch. Assuming the usual law to hold good for the whole of the depth, the maximum skin stress per square inch would be 3,774 lbs.

The co-efficient of elasticity, as determined from an increment in the deflection of .635 in. between the loads of 2,500 lbs. and 14,500 lbs., is 929,690 lbs.

Table R shows the several readings.

The weight of the beam on November 29th was 430 lbs., or 28.71 lbs. per cubic foot, and on December 9th, the date of test, the weight was 415 lbs. 6½ ozs., or 26.08 lbs. per cubic foot, showing a loss of weight at the rate of .263 lb. per cubic foot per day.

Beam LIII was tested December 9th, 1893, with the annual rings as in Fig. 105.

The beam was a poor specimen, being full of knots and season cracks, and partly decayed. The grain on the top was parallel, while on the sides it was somewhat oblique.

The load upon the beam was gradually increased until it amounted to 18,600 lbs., when it failed by the tearing apart of the fibres on the tension side.

The maximum skin stress due to this breaking load is 2,495 lbs. per square inch.

The co-efficient of elasticity, as determined by an increment in the deflection of .55 in. between the loads of 1,500 lbs. and 10,000 lbs., is 650,930 lbs.

Table R shows the several readings.

The weight of the beam was 450 lbs. 12 ozs., or 29.02 lbs. per cubic foot on Nov. 9th, and 438 lbs. 13 ozs., or 28.25 lbs. per cubic foot on Dec. 8th, showing a loss of weight at the rate of .0855 lb. per cubic foot per day.

The time occupied by the test was 20 minutes.

The following table gives the summary of the results obtained for White Pine :—

NEW TIMBER.

BEAMS.	Dimensions in inches.	Weight in lbs. per cubic foot at date of test.	Maximum skin stress in lbs. per sq. in.	Co-efficient of elasticity in lbs.
	l × d × b			
XLII.	288 × 18 × 9	41.49	3,815	979,220
XLV.	288 × 18 × 9	41.49	3,681	956,540
XLVIII.	150 × 15.1875 × 9.375	41.08	3,991	1,164,700
XLVI.	120 × 18 × 9	39.53	2,740	536,770
XLVII.	120 × 18 × 9	39.40	3,003	617,283
XLIII.	120 × 18 × 9	39.50	3,000	649,780
XLIV.	120 × 18 × 9	39.40	3,148	649,780
XXXVI.	288 × 18 × 9	37.25	2,993	500,000
XLIX.	150 × 15.37 × 9.125	37.24	4,936	1,052,600
XXXVII.	288 × 18 × 9	36.43	3,555	
XL.	120 × 18 × 9	36.13	3,311	693,090
XLI.	120 × 18 × 9	36.13	2,500	519,820
XXXVIII	114 × 18 × 9	34.78	3,075	622,640
XXXIX.	102 × 18 × 9	34.78	2,696	433,250
L	186 × 15 × 9.0625	33.64	4,370	1,184,240
OLD TIMBER.				
LIII.	180 × 15 × 9.05	28.25	2,495	650,930
LI.	192 × 15.12 × 9	28.3	3,212	982,480
LII.	180 × 14.85 × 9.05	26.08	3,589	929,690

Hence, for the new timber,

The average weight in lbs. per cubic foot = 37.88.

" co-efficient of elasticity in lbs. per sq. in. = 754,265.

" maximum skin stress " " = 3388.

The following data are suggested for practice :—

The average weight in lbs. per cubic foot = 37.8.

" co-efficient of elasticity in lbs. per sq. in. = 754,000.

" maximum skin stress " " = 3,300.

" safe working skin stress in lbs. per sq. in., 3 being at factor of safety = 1100.

Further experiments will probably show that these data require some modification. In fact, the actual skin stress and co-efficients of elasticity are certainly greater than those given in the preceding table, which have been calculated on the assumption that the amount of the compression at the central support is sufficiently small to be disregarded, but it has been shewn, as for example, in the case of Beam XLIX, that the skin stresses are largely affected by this compression. The co-efficients of elasticity are also necessarily increased by the diminution in the effective depth. Similar remarks apply to the other timbers.

From the experiments with the old White Pine stringers, it might be inferred that these timbers have lost considerably in weight, but that they have in a great degree retained their strength and stiffness. Other old timbers will require to be tested, however, before any definite statement can be made on the subject.

NEW SPRUCE BEAMS.

Beam LIV was tested Nov. 2nd, 1893, with the annual rings as in Fig. 106.

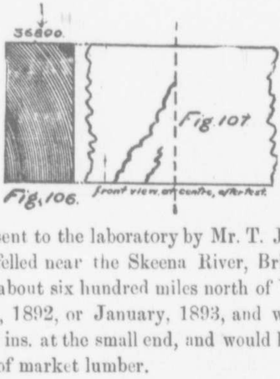


Fig. 106.

This stick was sent to the laboratory by Mr. T. J. Claxton. It was cut out of a tree felled near the Skeena River, British Columbia, on the Pacific Coast, about six hundred miles north of Victoria. The log was felled in Dec., 1892, or January, 1893, and was over 100 ft. in length, squared 36 ins. at the small end, and would have provided from 12,000 to 15,000 of market lumber.

The beam in question was sawn from the log in June, 1893, and was shipped by steamer at the end of June from the town of Claxton, situated at the mouth of the Skeena River, where the mills are located. At Victoria the beam was transhipped and brought down in August via the C.P.R. to Montreal. It was delivered at the laboratory early in September.

It might, perhaps, be of interest to note that the cost of freight for this beam from Claxton to Victoria was $4.00 ; from Victoria to Vancouver $2.00 ; from Vancouver to Montreal $46.00 ; and the cartage to the University $4.00, making a total cost of freight of $56.00.

It is said that the spruce from the Skeena District is of a specially fine quality, having a clear straight grain, and possessing a large amount of toughness.

The load upon the beam was gradually increased until it amounted to 36,800 lbs., when the beam failed by the crippling of the fibres on the compression side, Fig. 107.

The maximum skin stress corresponding to this breaking load is 5,908 lbs. per square inch.

The total compression of the material at the central support was .5 in., so that taking the effective depth as 17 ins., the maximum skin compressive stress is 5,941 lbs. per square inch, the corresponding skin tensile stress being 6,301 lbs. per square inch.

If it is assumed that the usual law holds good for the whole of the effective depth of 17 ins., the maximum skin stress is 6,260 lbs. per square inch.

The co-efficient of elasticity, as deduced from an increment in the deflection of 1.15 ins. between the loads of 1,000 and 15,000 lbs., is 1,528,499 lbs.

Table S shows the several readings.

The weight of the beam on Oct. 3rd was 751 lbs. 6 ozs., or 27.206 lbs. per cubic foot, and on Nov. 3rd, the date of test, it weighed 735 lbs. $2\frac{1}{2}$ ozs., or 26.614 lbs. per cubic foot, showing a loss while in the laboratory at the rate of .019 lbs. per cubic foot per day.

Beams LV and LVI are the ends of Beam LIV, the central portion containing the fracture having been cut out.

Beam LV was tested Nov. 3rd, 1893, with the annual rings as in Fig. 108.

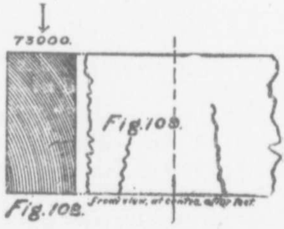


Fig. 108.

The load was gradually increased until it amounted to 73,000 lbs., when it failed by the crippling of the fibres on the compression side, Fig. 109.

The maximum skin stress corresponding to this load is 4,839 lbs. per square inch.

The maximum compression of the material at the central support was 2 ins., so that taking 15.5 ins. as the effective depth, the maximum compressive skin stress is 5,123 lbs. per square inch, the corresponding tensile skin stress being 6,641 lbs. per square inch.

If it is assumed that the usual law holds good for the whole of the effective depth, the maximum skin stress becomes 6,176 lbs.

As soon as the beam was relieved of load, the amount of compression at the support was immediately diminished by .9 in., and at the end of thirteen days the amount of compression was .82 in.

The co-efficient of elasticity, as determined by an increment in the deflection of .17 in., between the loads of 3,000 lbs. and 10,000 lbs., is 1,070,950 lbs.

Table T shows the several readings.

The weight of the beam on Nov. 3rd, date of test, was 26.614 lbs. per cubic foot.

Beam LVI was tested Nov. 4th, 1893, with the annual rings as in Fig. 110.

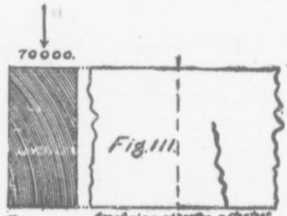


Fig. 110.

The load was gradually increased until it amounted to 70,000 lbs., when it failed by the crippling of the fibres on the compression side, Fig. 111.

The maximum skin stress corresponding to this breaking load is 4,614 lbs. per square inch.

The maximum compression at the centre of support was 1.9 ins., so that taking 15.6 ins. as the effective depth, the maximum compressive skin stress is 4,916 lbs. per square inch, the corresponding tensile skin stress being 6,280 lbs. per square inch.

If it is assumed that the usual law holds good for the whole of the effective depth, then the maximum skin stress becomes 5,806 lbs. per square inch.

Ten days after this beam had been relieved of load, the amount of the compression of the timber at the centre of support was diminished to .77 in.

The co-efficient of elasticity, as determined by an increment in the deflection of .18 in. between the loads of 10,000 lbs. and 30,000 lbs., is 1,011,450 lbs.

Table T shows the several readings.

The weight of this beam on Nov. 3rd was 26.614 lbs. per cubic foot.

OLD SPRUCE.

Beams LVII-LIX were three spruce stringers sent to the laboratory by Mr. P. A. Peterson.

Beams LVII and LVIII were cut at Galbraith's Mill, three miles from Sherbrooke, in 1886, and grew near the same place. They were used in the construction of the bridge near Lennoxville in the winter of 1886-87, and had been in service until the summer of 1894, or for a period of about eight years.

Beam LIX was taken out of Bridge E 61 at Roxton Falls during the summer of 1894, and had been in service since 1885, *i.e.*, for about eight years. This stringer was purchased by Bridge-master MacFarlane, and no further information has been obtained as to its history. The stringer was boxed ½ in. at the ends on the bearings, and several season cracks were shown on the surface.

Beam LVII was tested on the 21st April with the annual rings as in Fig. 112.

Fig. 112. Fig. 113.

front perspective after fracture.

Fig. 114.

The load upon the beam was gradually increased until it amounted to 25,700 lbs., when the beam failed by shearing longitudinally along the surface of a season crack, the distance between the portions above and below the plane of shear at the end being ⅜ in.

Immediately after the fracture the jockey weight was run back until the lever again floated, the load upon the beam being 21,000 lbs. This load was then gradually increased until it amounted to 24,700 lbs., when failure occurred by the tearing apart of the fibres on the tension side and by a further crippling of the fibres on the compression side. The lap at the end of the plane of shear was also increased to $\frac{5}{8}$ in.

The maximum skin stress corresponding to the breaking load of 25,700 lbs. is 3,459 lbs. per square inch.

The maximum compression of the material at the support was .31 in., so that taking the effective depth to be 14.69 ins., the maximum compressive skin stress is 3,526 lbs. per square inch, the corresponding tensile skin stress being 3,678 lbs. per square inch.

If it is assumed that the usual law holds good for the whole of the effective depth, then the maximum skin stress becomes 3,607 lbs. per square inch.

The co-efficient of elasticity, as determined by an increment in the deflection of .7 in. between the loads of 1,500 and 12,500 lbs., is 1,123,400 lbs.

Table U shows the several readings.

The weight of this beam on April 10th was 502 lbs., or 33.82 lbs. per cubic foot; its weight on April 21st, date of test, was 491 lbs. 4 ozs., or 33.09 lbs. per cubic foot, showing a loss of weight at the rate of .0645 lbs. per cubic foot per day.

Beam LVIII was tested May 1st, 1894, with the annual rings as in Fig. 113. Season cracks ran intermittently from end to end of the beam.

The load upon this beam was gradually increased until it amounted to 27,470 lbs. Under this load the beam failed by shearing longitudinally along a season crack, as shown in Fig. 114, with a partial tension fracture near the end of the beam. The season crack for a distance of about 3 ft. from the centre of the beam appears weathered through the entire thickness of the beam.

Previously, however, to this longitudinal shear, the beam had evidently failed by the crippling of the material, Fig. 114, on the compression side along a line near the centre of the beam where the timber was apparently free from knots and where the fibres were parallel with the axis.

The maximum skin stress corresponding to the load of 27,470 lbs., is 5,709 lbs. per square inch.

The co-efficient of elasticity, as determined by an increment in the deflection of .575 lbs. between the loads of 2,000 and 12,000 lbs., is 1,316,900 lbs.

Table U shows the several readings.

The weight of the beam on March 10th was 267 lbs. 1 oz., or 27.36 lbs. per cubic foot, and its weight on May 2nd was 258 lbs. 6 ozs., or 26.47 lbs. per cubic foot, showing a loss of weight while in the laboratory at the rate of .0168 lb. per cubic foot per day.

Beam LIX was tested June 2nd, 1894, with the annual rings as in Fig. 115.

Fig. 115. Fig. 116 Fig. 117.

The load was gradually increased until it amounted to 21,700 lbs., when the beam failed by the tearing apart of the fibres on the tension side.

The maximum skin stress corresponding to this load is 2,963 lbs. per square inch.

The maximum compression at the centre was .7 in., so that taking 14.3 ins. as the effective depth, the maximum compressive skin stress is 3,079 lbs. per square inch, the corresponding tensile skin stress being 3,396 lbs. per square inch.

If it is assumed that the usual law holds good for the whole of the effective depth, then the maximum skin stress is 3,261 lbs. per sq. in.

The co-efficient of elasticity, as determined by an increment in the deflection of .43 in. between the loads of 2,000 lbs. and 10,000 lbs., is 905,601 lbs.

Table U shows the several readings.

The weight of the beam on June 1st was 445 lbs. 13 ozs., or 30.12 lbs. per cubic foot. Its weight on June 8th was 440 lbs, or 29.72 lbs. per cubic foot, showing a loss of weight at the rate of .0571 lb. per cubic foot per day.

Beams LX and XLI are two old spruce stringers sent to the laboratory by Mr. P. A. Peterson.

They had been in use in Culvert E 39 on the north division of the South Eastern Railway, 1½ miles north of Waterloo Station, since Oct., 1891, or for about three years.

These timbers were cut and sawn at Keene & Company's mills at the boundary east of Megantic.

Beam LX was tested on Nov. 10th, 1894, with the annual rings as in Fig. 116.

The upper portion of the stringer, *i.e.*, the part in tension, was partially rotten to a depth of about 1 in., and the effective depth at the centre of the beam did not exceed $11\frac{1}{4}$ ins. The remainder of the section at the centre was in a perfectly sound and good condition.

The load upon the beam was gradually increased until it amounted to 16,050 lbs., when it failed by the tearing apart of the fibres on the tensile side. The load was still increased, and a more complete fracture occurred under a load of 21,240 lbs. Immediately after this second fracture the jockey weight was run back until the lever again floated, when the load was 15,900 lbs. The load was again gradually increased until it amounted to 18,800 lbs., when fracture again occurred.

The maximum skin stress corresponding to the breaking load of 16,050 lbs. is 2,934 lbs.

The maximum compression of the material at the centre was .25 in., so that taking the effective depth to be 11. ins., the maximum compressive skin stress is 3,043 lbs. per square inch, and the corresponding tensile skin stress is 3,184 lbs. per square inch.

If it is assumed that the usual law holds good for the whole of the effective depth, the maximum skin stress becomes 3,118 lbs. per square inch.

The co-efficient of elasticity, as determined by an increment in the deflection of .390 in. between the loads of 2,000 and 12,000 lbs., is 1,352,250 lbs. per square inch.

Table V gives the several readings.

The weight of this beam on Nov. 10th, date of test, was 255 lbs. $12\frac{1}{2}$ ozs., or 27.26 lbs. per cubic foot.

Beam LXI was tested Nov. 17th, 1894, with the annual rings as in Fig. 117. There were season cracks from end to end on the front face and numerous knots of medium and small size on the sides. The darkened portion indicates sapwood.

The load upon the beam was gradually increased until it amounted to 18,400 lbs., when the beam failed by the tearing apart of the fibres on the tension face.

The maximum skin stress corresponding to this load is 4,309 lbs. per square inch.

The maximum compression of the material at the centre was .21 in., so that taking the effective depth to be 14.29 ins., the maximum skin compressive stress is 4,432 lbs. per square inch, the corresponding tensile skin stress being 4,565 lbs. per square inch.

If it is assumed that the usual law holds good for the whole of the effective depth, the maximum skin stress becomes 4,502 lbs. per square inch.

The co-efficient of elasticity, as determined from an increment of .6 in. in the deflection between the loads of 1,000 lbs. and 9,000 lbs., is 1,250,850 lbs.

The weight of this beam on Nov. 17th, date of test, was 267 lbs., or 28.85 lbs. per cubic foot.

The following table gives a summary of the results obtained for Spruce :—

NEW TIMBER.

BEAM.	Dimensions in inches.	Weight in lbs. per cubic foot at date of test.	Maximum skin stress in lbs. per sq. in.	Co-efficient of elasticity in lbs.
	l × d × b			
LIV.	288 × 17.5 × 8.875	26.614	5,908	1,528,499
LV.	120 × 17.5 × 8.875	26.614	4,839	1,070,950
LVI.	120 × 17.5 × 9.975	26.614	4,614	1,011,450

OLD TIMBER.

BEAM.	Dimensions in inches.	Weight in lbs. per cubic foot at date of test.	Maximum skin stress in lbs. per sq. in.	Co-efficient of elasticity in lbs.
LVII.	180 × 15 × 9	33.09	3,459	1,123,400
LIX.	180 × 15 × 9	30.12	2,963	905,601
LXI.	186 × 14.5 × 5.625	28.85	4,309	1,250,850
LX.	138 × 11.25 × 8.875	27.26	2,934	1,352,250
LVIII.	180 × 14.75 × 6	26.47	5,709	1,316,900

Beams LV and LVI were cut out of Beam LIV as already described. The wide variation in the value of the skin stress and of the co-efficient of elasticity is undoubtedly due to the fact that the amount of the compression at the central support has been disregarded in the calculations. If this compression is taken into account, and if it is assumed that the ordinary theory of flexure holds good for the whole of the effective depth, it has been shewn that the skin stresses in lbs. per sq. in. become 6260 for Beam LIV, 6176 for Beam LV, and 5806 for Beam LVI, the variation in the magnitude of the stresses being comparatively small.

Further experiments will be made with new spruce beams.

The old spruce stringers were found to possess ample strength and stiffness for the work they were designed to do. The experiments gave :—

29.15-lbs. as the average weight per cubic foot.
1,189,800 " " co-efficient of elasticity.
3875 " " maximum skin stress per sq. in.

The following tables A to V give the end deflections and in some cases the deflections at points dividing the beam into four, six, or eight equal parts, the distance of these points from the ends being stated at the heads of the columns.

Tables A to I show the deflections in inches of Canadian New Douglas Fir Beams (I to XXV) under gradually increased loads.

TABLE A.

Deflections of Beam I at ends.

Loads in lbs.	Deflection.	Loads in lbs.	Deflection.	Loads in lbs.	Deflection.	Loads in lbs.	Deflection.	Loads in lbs.	Deflection.
2,000	.02	9,000	.095	16,000	.18	23,000	.27	30,000	.39
2,500	.03	9,500	.10	16,500	.19	23,500	.28	30,500	.40
3,000	.03	10,000	.11	17,000	.195	24,000	.285	31,000	.41
3,500	.035	10,500	.115	17,500	.20	24,500	.295	31,500	.42
4,000	.04	11,000	.12	18,000	.205	25,000	.30	32,000	.43
4,500	.045	11,500	.125	18,500	.21	25,500	.31	32,500	.445
5,000	.05	12,000	.13	19,000	.22	26,000	.315	33,000	.46
5,500	.055	12,500	.14	19,500	.225	26,500	.32	34,000	.49
6,000	.06	13,000	.145	20,000	.230	27,000	.33	35,000	.51
6,500	.07	13,500	.15	20,500	.24	27,500	.34	36,000	.53
7,000	.075	14,000	.155	21,000	.245	28,000	.35	37,000	.56
7,500	.075	14,500	.16	21,500	.25	28,500	.36		
8,000	.08	15,000	.165	22,000	.255	29,000	.37		
8,500	.09	15,500	.17	22,500	.265	29,500	.38		

Breaking weight of Beam I = 45,000 lbs.

TABLE B.

Loads in lbs.	Deflections of Beams.						
	II	III	IV	V	VI	VII	VIII
	Ends.	Ends.	Ends.	Ends.	Ends.	Ends.	Ends.
300	....	....	....	....	....	....	.02
500	....	.05	....	.005	.02	.015	.03
800	....	....	....	....	....	....	.05
1,000	....	.08	.03	.01	.04	.03	.07
1,300	....	....	....	....	....	....	.09
1,500	....	.11	.045	.02	.06	.04	.10
1,800	....	....	....	..	....	....	.12
2,000	.035	.14	.05	.03	.075	.06	.135
2,200	....	....	....	....	....	....	.15
2,400	....	....	....	....	....	....	.165
2,500	....	.155	.055	.05	.10	.075	....
2,600	....	....	....	....	....	....	.18
2,800	....	....	....	....	....	....	.195
3,000	....	.18	.065	.055	.12	.10	.205
3,400	....	....	....	....	....	....	.235
3,500	....	.21	.08	.065	.14	.115	....
3,800	....	....	....	....	....	....	.26
4,000	.05	.23	.095	.07	.16	.125	.28
4,500	....	.25	.105	.08	.18	.14	.315
5,000	....	....	.115	.09	.20	.155	.35
5,500	....	....	.13	.105	.22	.175	.39
6,000	.065	....	.145	.11	.24	.195	....
6,500	....	..	.155	.125	.26	.21	....
7,000	....	....	.165	.135	.28	.22	....
7,500	....	....	.18	.145	.305	.235	....
8,000	.075	....	.19	.16	.32	.25	....
8,500	....	....	.20	.17	....	.27	....
9,000	....	....	.215	.18	....	....	....
9,500	....	....	.23	.195	....	....	....
10,000	.085	....	.245	.205	....	....	....
10,500	....	....	.26	.22	....	....	....
11,000	....	....	.28	.235	....	....	....
11,500	....	....	.30	.25	....	....	....
12,000	.10	....	.315	.26	....	....	....
12,500	....	....	.33	.27	....	....	....
13,000	.105	....	.35	.28	....	....	....
13,500	....	....	.365	.29	....	....	....
14,000	.110	....	.38	.305	....	....	....
14,500	....	....	....	.315	....	....	....
15,000	.115	....	....	.33	....	....	....
15,500	....	....	....	.345	....	....	....
16,000	.12	....	....	....	....	....	....
16,400	....	....	....	....	....	.75	....
17,000	.13	....	....	....	....	....	....
18,000	.135	....	....	....	....	....	....
20,000	.14	....	....	....	....	....	....
21,000	....	....	....	.72	....	....	....
22,000	.15	....	....	....	....	....	....
24,000	.165	....	....	....	....	....	....
26,000	.175	....	....	....	....	....	....
28,000	.190	....	....	....	....	....	....

Breaking Weight of Beam II = 36,575 lbs.
" " III = 12,950 "
" " IV = 16,720 "
" " V = 23,610 "
" " VI = 15,480 "
" " VII = 17,615 "
" " VIII = 11,700 "

TABLE C.

Loads in lbs.	Deflections of Beam IX.					Deflections of Beam X.				
	34 ins.	68 ins.	Ends.	68 ins.	34 ins.	33 ins.	66 ins.	Ends.	66 ins.	33 ins.
1000	.01	.01	.02	.01	.01	.02	.01	.02	.01	.02
1500	.03	.02	.04	.02	.03	.05	.02	.05	.02	.05
2000	.03	.03	.05	.025	.04	.07	.03	.08	.04	.07
2500	.04	.03	.05	.03	.05	.10	.05	.11	.05	.10
3000	.10	.07	.06	.05	.09	.12	.06	.14	.06	.12
3500	.10	.08	.12	.05	.10	.15	.07	.17	.07	.15
4000	.10	.08	.13	.055	.10	.17	.09	.20	.08	.17
4500	.10	.08	.14	.065	.11	.20	.10	.23	.10	.20
5000	.15	.10	.18	.085	.15	.22	.11	.26	.115	.22
5500	.15	.11	.19	.09	.16	.25	.12	.29	.12	.25
6000	.15	.12	.20	.10	.17	.27	.14	.32	.14	.27
6500	.19	.13	.24	.11	.20	.30	.15	.35	.15	.30
7000	.20	.13	.25	.115	.20	.32	.17	.38	.16	.32
7500	.20	.13	.25	.11	.21	.35	.18	.41	.18	.35
8000	.20	.13	.26	.125	.22	.37	.20	.44	.20	.37
8500	.22	.14	.27	.135	.24	.40	.21	.47	.21	.40
9000	.22	.15	.28	.14	.24	.42	.22	.50	.22	.42
9500	.22	.15	.28	.145	.25	.45	.23	.53	.23	.45
10000	.26	.16	.33	.16	.28	.47	.25	.56	.24	.47
10500	.33	.20	.40	.19	.34	.49	.26	.58	.25	.49
11000	.34	.21	.42	.20	.35	.51	.27	.61	.27	.51
11500	.35	.22	.44	.205	.36	.54	.29	.64	.29	.54
12000	.39	.23	.47	.22	.40	.56	.30	.68	.30	.56
12500	.40	.24	.49	.22	.40	.59	.32	.71	.32	.59
13000	.40	.24	.50	.23	.41	.61	.33	.74	.33	.61
13500	.45	.27	.54	.25	.45	.64	.34	.77	.34	.64
14000	.45	.27	.55	.255	.46	.66	.36	.80	.36	.66
14500	.45	.27	.56	.26	.46	.69	.37	.83	.375	.69
15000	.50	.29	.60	.27	.50	.71	.39	.86	.39	.71
15500	.50	.30	.61	.28	.51	.74	.40	.89	.40	.74
16000	.50	.30	.62	.29	.52	.75	.41	.92	.41	.76
16500	.55	.31	.66	.31	.55	.79	.43	.96	.43	.79
17000	.55	.32	.67	.31	.56	.81	.44	.99	.45	.82
17500	.56	.33	.68	.32	.57	.85	.46	1.02	.46	.85
18000	.56	.33	.69	.325	.58	......	......	....	......	......
18500	.60	.36	.75	.35	.62	......	......	....	......	......
19000	.63	.36	.77	.35	.64	......	......	....	......	......
19500	.64	.37	.78	.36	.65	......	......	....	......	......
20000	.65	.37	.79	.365	.66	......	......	....	......	......
40000	......	......	1.75	......	......	......	......	....	......	......
47000	......	......	2.20	......	......	......	......	....	......	......

Breaking Weight of Beam IX = 51,600 lbs.
" " " X = 18,000 "

TABLE D.

Loads in lbs.	Deflections of Beam XI.					Deflections of Beam XII.				
	34 ins.	68 ins.	Ends.	68 ins.	34 ins.	34 ins.	68 ins.	Ends.	68 ins.	34 ins.
1000	......	......	......	......	......	.01	.005	.01	.01	.01
1500	.02	.01	.035	.015	.025	.03	.02	.035	.02	.035
2000	.05	.02	.05	.025	.04	.05	.025	.055	.03	.05
2500	.06	.03	.075	.035	.06	.065	.04	.075	.05	.07
3000	.075	.04	.10	.045	.08	.09	.045	.10	.05	.09
3500	.10	.05	.115	.055	.095	.105	.06	.12	.06	.105
4000	.11	.06	.135	.06	.11	.12	.07	.145	.07	.12
4500	.13	.07	.16	.07	.135	.15	.075	.165	.08	.145
5000	.15	.075	.175	.075	.14	.155	.09	.185	.09	.155
5500	.16	.085	.20	.09	.16	.17	.10	.205	.10	.17
6000	.185	.10	.22	.10	.18	.19	.11	.23	.11	.19
6500	.20	.105	.24	.11	.195	.21	.12	.25	.12	.21
7000	.215	.115	.26	.11	.215	.23	.13	.27	.13	.235
7500	.24	.125	.28	.13	.235	.25	.14	.295	.14	.25
8000	.25	.135	.30	.14	.245	.27	.15	.315	.15	.27
8500	.26	.145	.32	.15	.265	.29	.15	.34	.16	.29
9000	.27	.15	.33	.155	.27	.305	.17	.36	.17	.305
9500	.30	.16	.35	.165	.29	.32	.18	.305	.18	.32
10000	.315	.17	.38	.175	.305	.35	.19	.405	.19	.35
10500	.34	.185	.40	.185	.335	.36	.20	.425	.20	.36
11000	.36	.195	.435	.20	.36	.375	.21	.45	.21	.38
11500	.36	.20	.435	.20	.36	.39	.22	.47	.22	.40
12000	.395	.215	.475	.22	.395	.41	.23	.495	.23	.41
12500	.40	.22	.50	.23	.405	.44	.24	.51	.24	.44
13000	.42	.23	.505	.24	.42	.45	.25	.535	.25	.45
13500	.45	.25	.54	.255	.445	.47	.26	.555	.26	.47
14000	.46	.255	.56	.265	.46	.49	.27	.58	.27	.49
14500	.48	.265	.57	.275	.475	.50	.28	.60	.28	.505
15000	.50	.275	.60	.28	.50	.52	.29	.62	.30	.52
15500	.515	.285	.62	.29	.515	.55	.30	.645	.305	.55
16000	.535	.295	.645	.30	.53	.555	.305	.665	.31	.56
16500	.54	.30	.65	.30	.535	.575	.32	.69	.32	[illegible]
17000	.58	.32	.695	.32	.575	.60	.325	.71	.33	.60
17500	.585	.32	.70	.325	.575	.61	.33	.73	.345	.615
18000	.61	.34	.735	.345	.61	.63	.345	.755	.35	.635
18500	.61	.34	.745	.35	.615	.65	.35	.77	.36	.65
19000	.65	.36	.78	.365	.655	.665	.36	.80	.375	.665
19500	.65	.36	.785	.375	.655	.685	.37	.82	.385	.69
20000	.655	.365	.80	.375	.66	.705	.38	.85	.40	.705
20500	......	......	......	......	......	.73	.395	.87	.41	.725
21000	......	......	......	......	......	.75	.40	.89	.415	.75
21500	......	......	......	......	......	.75	.405	.90	.415	.75
22000	......	......	......	......	......	.78	.42	.935	.435	.78
22500	......	......	......	......	......	.81	.435	.96	.45	.805
23000	......	......	......	......	......	.82	.445	.98	.455	.82
24000	......	......	.94	......	......	......	......	......	......	......
26500	......	......	......	......	......	......	......	1.12	......	......
28000	......	......	1.14	......	......	......	......	1.17	......	......
29000	......	......	......	......	......	......	......	1.22	......	......
32000	......	......	......	......	......	......	......	1.40	......	......
33000	......	......	1.35	......	......	......	......	1.42	......	......
35800	......	......	1.45	......	......	......	......	......	......	......
37000	......	......	......	......	......	......	......	1.67	......	......
39000	......	......	......	......	......	......	......	1.97	......	......
42000	......	......	......	......	......	......	......	2.00	......	......
45000	......	......	......	......	......	......	......	2.28	......	......
48000	......	......	......	......	......	......	......	2.73	......	......
49000	......	......	......	......	......	......	......	2.9	......	......

Breaking Weight of Beam XI = 35,800 lbs.
" " " XII = 49,000 "

TABLE E.

Loads in lbs.	Deflections of Beam XIII.					Deflection of Beam XIV.	Deflections of Beam XV.					Deflection of Beam XVI.
	34 ins.	68 ins.	Ends	68 ins.	34 ins.	Ends.	33 ins.	66 ins.	Ends.	66 ins.	33 ins.	Ends.
760	....	....	....	....	....	.03	....	....	....	....	....	....
1000	25.	.02	.04	.02	.025	.05	.01	.01	.02	.01	.02	.025
1140	....	....	....	....	....	....	....	....	....	....	....	.035
1500	.05	.035	.07	.03	.05	.085	.04	.02	.05	.025	.04	.05
1900	....	....	....	....	....	....	....	....	....	....	....	.075
2000	.08	.05	.105	.05	.08	.115	.055	.035	.08	.045	.06	....
2300	....	....	....	....	....	....	....	....	....	....	....	.09
2500	.10	.065	.14	.065	.11	.15	.08	.045	.095	.05	.075	....
2600	....	....	....	....	....	....	....	....	....	....	....	.10
2800	....	....	....	....	....	.17	....	....	....	....	....	....
3000	.14	.08	.17	.08	.14	.19	.10	.05	.115	.06	.10	.125
3200	....	....	....	....	....	.20	....	....	....	....	....	....
3400	....	....	....	....	....	.22	....	....	....	....	....	.145
3500	.16	.10	.21	.10	.16	....	.11	.065	.14	.07	.12	....
3600	....	....	....	....	....	.225	....	....	....	....	....	....
3800	....	....	....	....	....	.25	....	....	....	....	....	.16
4000	.20	.11	.245	.11	.20	.255	.13	.08	.16	.085	.14	.175
4410	....	....	....	....	....	.275	....	....	....	....	....	.20
4500	.22	.13	.275	.125	.22	....	.155	.095	.185	.095	.16	....
4800	....	....	....	....	....	.315	....	....	....	....	....	.215
5000	.25	.145	.31	.14	.25	.32	.165	.105	.215	.105	.17	.225
5200	....	....	....	....	....	.345	....	....	....	....	....	.23
5400	....	....	....	....	....	.355	....	....	....	....	....	....
5500	.275	.15	.34	.155	.275	....	.19	.11	.24	.115	.20	....
5600	....	....	....	....	....	.36	....	....	....	....	....	.25
5800	....	....	....	....	....	.39	....	....	....	....	....	....
6000	.30	.165	.36	.17	.30	.40	.21	.125	.26	.125	.215	.27
6400	....	....	....	....	....	....	....	....	....	....	....	.29
6500	.33	.18	.40	.185	.33	....	.23	.13	.285	.14	.235	....
6600	....	....	....	....	....	.435	....	....	....	....	....	....
6800	....	....	....	....	....	.465	....	....	....	....	....	.31
7000	.36	.20	.44	.20	.36	.485	.255	.145	.31	.15	.255	.325
7200	....	....	....	....	....	.50	....	....	....	....	....	....
7400	....	....	....	....	....	.505	....	....	....	....	....	.34
7500	.38	.215	.47	.22	.39	....	.27	.155	.335	.16	.275	....
7800	....	....	....	....	....	.54	....	....	....	....	....	.36
8000	.41	.225	.50	.23	.41	.56	.295	.165	.35	.175	.30	.375
8300	....	....	....	....	....	.585	....	....	....	....	....	....
8400	....	....	....	....	....	....	....	....	....	....	....	.40
8500	.45	.245	.54	.245	.45	....	.31	.18	.38	.18	.315	....
8600	....	....	....	....	....	.605	....	....	....	....	....	....
8800	....	....	....	....	....	....	....	....	....	....	....	.42
9000	.46	.255	.57	.26	.47	.64	.34	.19	.40	.19	.34	.425
9200	....	....	....	....	....	.66	....	....	....	....	....	....
9400	....	....	....	....	....	....	....	....	....	....	....	.45

TABLE E.—(*Continued.*)

Loads in lbs.	Deflections of Beam XIII.					Deflections of Beam XIV.	Deflections of Beam XV.					Deflections of Beam XVI.
	34 ins.	68 ins.	Ends	68 ins.	34 ins.	Ends	33 ins.	66 ins.	Ends.	66 ins.	33 ins.	Ends
9500	.50	.275	.605	.28	.50	....	.35	.20	.425	.205	.355	.....
9600	.....	....	....	....	....	.69	....	....	....	....	....	.....
9800	.....	....	....	....	....	.715	....	....	....	....	....	.475
10000	.52	.29	.64	.295	.53	.73	.37	.21	.44	.21	.375	.485
10200	.....	....	....	....	....	.76	....	....	....	....	....	.....
10400	....	....	....	....	....	.765	....	....	....	....	....	.....
10500	.55	.305	.67	.31	.55	....	.40	.22	.475	.22	.40	.....
10600	.....	....	....	....	....	.80	....	....	....	....	....	.....
10800	.....	....	....	....	....	.805	....	....	....	....	....	.....
11000	.585	.32	.705	.325	.585	....	.415	.23	.50	.24	.415	.54
11300	.....	....	....	....	....	.845	....	....	....	....	....	.....
11500	.61	.34	.745	.345	.61	....	.44	.24	.525	.25	.445	.565
11700	.....	....	....	....	....	.88	....	....	....	....	....	.....
12000	.64	.35	.78	.36	.64	.91	.45	.255	.55	.26	.45	.59
12200	.....	....	....	....	....	.935	....	....	....	....	....	.....
12400	.....	....	....	....	....	.95	....	....	....	....	....	.....
12500	.66	.365	.81	.375	.67	....	.47	.265	.57	.27	.465	.61
12600	.....	....	....	....	....	.955	....	....	....	....	....	.....
12800	.....	....	....	....	....	1.00	....	....	....	....	...	.....
13000	.70	.385	.845	.395	.70	1.00	.495	.275	.60	.28	.50	.65
13200	.	....	....	....	....	1.02	....	....	....	....	....	.....
13500	.725	.40	.885	.41	.735	....	.51	.285	.62	.29	.51	.68
14000	.75	.415	.915	.42	.76	....	54	.295	.64	.30	.54	.71
14500	.795	.435	.96	445	.795	....	.55	.305	.66	.31	.55	.73
15000	.81	.45	.99	.46	.82	...	.57	.32	.69	.32	.575	.75
15500	.85	.47	1.025	.475	.85	....	.59	.33	.715	.335	.60	.78
16000	.875	.485	1.065	.49	.875	......	.61	.34	.74	.34	.615	.81
16500	.905	.505	1.10	.515	.915	....	.64	.35	.765	.35	.64	.83
17000	.94	.52	1.135	.525	.94	....	.65	.36	.79	.36	.655	.87
17500	.97	.54	1.18	.545	.975	....	.67	.375	.81	.375	.675	.90
18000	1.00	.55	1.22	.56	1.01	....	.69	.385	.835	.39	.70	.93
18500	1.04	.575	1.265	.58	1.045	....	.71	.395	.86	.40	.71	.95
19000	1.06	.59	1.31	.60	1.07	....	.74	.405	.875	.41	.735	.98
19500	1.1	.615	1.35	.62	1.1	....	.75	.415	.91	.42	.75	1.00
20000	1.14	.63	1.39	.635	1.14	....	.77	.425	.94	.43	.775	1.04
20500	1.165	.65	1.43	.655	1.175	....	....	....	.....	....	....	1.07
21000	1.21	.67	1.485	.68	1.22	....	....	....	1.20	....	....	1.10
21500	1.24	.685	1.515	.69	1.25	....	....	....	.....	....	....	1.13
22000	1.28	.71	1.57	.715	1.29	....	....	....	.....	....	....	1.15
22500	.....	....	.....	....	.....	....	....	....	.....	....	....	1.17
23000	.....	...	.....	....	.....	....	....	....	.....	....	....	1.20
24000	.. ..	....	1.70	....	.....	....	....	....	.....	....	....	.....
25000	.....	....	.....	....	.....	....	....	....	1.30	....	....	.....
26000	.....	....	1.88	....	.....	....	....	....	.....	...	....	.....
26300	.....	....	2.05	....	.....	....	....	....	.....	....	....	.....
27000	.....	....	.....	....	.....	....	....	....	1.45	....	....	.....
29000	.....	....	.....	....	.....	....	....	....	1.55	....	....	.....
29300	.....	....	2.6	....	.....	....	....	...	1.70	....	....	.....
30000	.....	....	.....	....	.....	....	....	....	1.90	....	....	.....
32000	.....	....	.....	....	.....	....	....	....	2.25	....	....	.....
35000	.....	....	.....	....	.....	....	....	....	2.33	....	....	.....
37000	.....	....	.....	....	.....	....	....	....	.....	....	....	.....

Breaking weight of Beam XIII = 29,300 lbs.
" " " XIV = 17,600 "
" " " XV = 37,000 "
" " " XVI = 25,580 to 32,000 lbs.

TABLE F.

Deflections of Beams XVII, XVIII and XIX.

Load in lbs.	XVII.	XVIII.				XIX.						
		1st Loading.	2nd Loading.	Beam gradually relieved of l'd	3rd Loading.	1st Loading.			Beam gradually relieved of load.			2nd loading.
	Ends.	E'ds	Ends	Ends.	E'ds	34½ ins.	End	34½ ins.	34½ ins.	End	34½ ins.	
1000	......	.005	.005	.015	....	.005	.010	.010	.010	.015	.020	....
2000	......	.010	.015	.015	....	.020	.045	.030	....	....	....	....
3000	......	.020	.030	.020	....	.050	.060	.055	......	....	....	....
4000	......	.030	.030	.030	....	.060	.090	.070	.055	.095	.060	....
5000	.07	.040	.040	.045	....	.070	.105	.085	......	....	....	....
6000	......	.050	.050	.050	....	.090	.130	.100	......	....	....	....
7000	......	.060	.060	.060	....	.110	.150	.105	......	....	....	....
7500	......	.065	....	......	....	....	....	....	......	....	....	....
8000	......	.075	.070	.070	....	.115	.170	.125	.120	.190	.120	....
8500	......	....	....	......	....	.120	.185	.135	......	....	....	....
9000	......	.080	.085	.075	....	.130	.200	.140	......	....	....	....
9500	......	....	....	......	....	.140	.215	.145	......	....	....	....
10000	.15	.095	.095	.085	....	.150	.225	.150	......	....	....	....
10500	......	.100	....	......	....	.155	.235	.160	......	....	....	....
11000	......	.100	.100	.095	....	.160	.245	.165	......	....	....	....
11500	......	....	....	......	....	.165	.250	.170	......	....	....	....
12000	......	.110	.110	.100	....	.170	.265	.180	.170	.255	.170	....
12500	.18	....	....	......	....	....	......	....	......	....	....	....
13000	......	....	.130	.110	....	.180	.290	.190	......	....	....	....
14000	......	....	.130	.125	....	.200	.310	.205	......	....	....	....
15000	.22	....	.140	.130	....	.210	.330	.220	......	....	....	....
16000	......	....	.150	.150	..	.225	.345	.230	......	....	....	....
17000	......	....	.165	.165	....	....	....	....	......	....	....	....
17500	.26	....	....	......	....	....	....	....	......	....	....	....
18000	......	....	.175	.170	....	....	....	....	......	....	....	....
19000	......	....	.185	.185	....	....	....	......	......	....	....	....
20000	.30	....	....	......	.200	....	....	....	......	....	....	.420
22000	......	....	....	......	....	....	....	....	......	....	....	.465
22500	.34	....	....	......	....	....	....	....	......	....	....	....
24000	......	....	....	......	....	....	....	....	......	....	....	.510
25000	.385	....	....	......	.220	....	....	....	....	....	....	....
26000	.405	....	....	......	....	....	....	....	......	....	....	.550
27000	.425	....	....	......	....	....	....	....	......	....	....	....
28000	.445	....	....	......	....	....	....	....	......	....	....	.600
29000	.465	....	....	......	....	....	....	.	......	....	....	....
30000	.485	....	....	......	.240	....	....	....	.....	....	....	.640
31000	.510	....	....	......	....	....	....	....	......	....	....	....
32000	.535	....	....	......	....	....	....	....	......	....	....	.695
33000	.560	.	....	.....	....	....	....	....	......	....	...	....
34000	.585	....	....	......	....	....	....	....	......	....	....	....
35000	.61	....	...	......	.310	....	....	....	......	....	....	.730
36000	.64	....	....	.....	....	....	....	....	......	....	....	....
37000	.68	....	....	......	....	....	....	....	......	....	....	.780
38000	.715	....	....	......	....	....	....	....	......	....	....	....

TABLE F.—(*Continued.*)

Deflections of Beams XVII, XVIII and XIX.

Load in lbs.	XVII	XVII.				XIX.						
		1st Loading.	2nd Loading.	Beam gradually relieved of l'd	3rd Loading.	1st Loading.			Beam gradually relieved of Load.			2nd loading.
	Ends.	E'ds	E'ds	Ends.	E'ds	34½ ins.	End	34¾ ins.	34½ ins.	End	34½ ins.	
39000	.75	....	....	....	....	....	....	....	....	....	....	.830
40000	.795	....	....	....	....	....	....	....	....	....	....	....
41000	.850	....	....	....	.340	....	....	....	....	....	....	870
4?000	.980	....	....	....	....	....	....	....	....	....	....	....
42500	1.005	....	....	....	....	....	....	....	....	....	....	.930
43000	1.030	....	....	....	....	....	....	....	....	....	....	....
43500	1.055	....	....	....	....	....	....	....	....	....	....	....
44000	1.085	....	....	....	.350	....	....	....	....	....	....	....
44500	1.125	....	....	....	....	....	....	....	....	....	....	.980
45000	1.150	....	....	....	....	....	....	....	....	....	....	....
45500	1.240	....	....	....	....	....	....	....	....	....	....	....
46000	1.285	....	....	....	....	....	....	....	....	....	....	1.030
46500	1.315	....	....	....	....	....	....	....	....	....	....	....
47000	1.365	....	....	....	.400	....	....	....	....	....	....	....
47500	1.455	....	....	....	....	..	....	....	....	....	....	....
48000	1.600	....		....	.440	....	....	....	....	....	....	1.100
48100	1.640	....	....	....	....	....	....	....	....	....	....	....
48200	1.675	....	....	....	....	....	....	....	....	....	....	....
48300	1.720	....	....	....	....	....	....	....	....	....	....	....
48400	1.830	....	....	....	..	....	....	....	....	....	....	....
48500	1.910	....	....	....	...	....	....	....	....	....	....	....
48600	2.020	....	....	....	....	....	....	....	....	....	....	....
50000	......	....	....	....	.490	....	....	....	....	....	....	1.160
52000	......	....	....	....	.500	....	....	....	....	....	....	1.230
54000	......	....	....	....	....	....	....	....	....	....	....	1.310
56000	......	....	....	....	....	....	....	....	....	....	....	1.510
57000	......	....	....	....	.540	....	....	....	....	....	....	....
58000	......	....	....	....	....	....	....	....	....	....	....	1.525
61000	......	....	....	....	.630		....	....	....	....	....	......
64000	......	....	....	....	.700	....	....	....	....	....	....	....
67000	......	....	....	....	.750	....	....	....	....	....	....	....

Breaking weight of Beam XVII = 48,600 lbs.
" " " XVIII = 69,400 "
" " " XIX = 59,540 "

TABLE G.

Deflections of Beams XX and XXI.

Load in lbs.	XX.							XXI.						
	1st Loading.			Beam gradually relieved of l'd	2nd Loading.			1st Loading.			Beam gradually relieved of l'd	2nd Loading.		
	34½ ins.	End	34½ ins.	Ends.	34½ ins.	Ends	34½ ins.	34½ ins.	E'ds	34½ ins.	Ends.	34½ ins.	Ends	34½ ins.
250	....	....	....	....	....	....	....	.009	.02	.010	.020	.015	.020	.015
500	.0	.0	.0	.001	.003	.005	.005	.025	.035	.025	....	....	....	....
750	....	....	....	....	....	....	....	.0375	.065	.045	....	....	....	....
1000	.015	.016	.015	....	....	....	....	.055	085	.060	.095	.065	.095	.065
1250	....	....	....	....	....	....	....	.075	.110	.075	....	....	....	....
1500	....	....	....	....	....	....	....	.095	.135	.090	....	....	....	....
1750	....	....	....	....	....	....	....	.110	.115	.110	....		....	....
2000	.035	.040	.040	.045	.035	.045	.040	.120	.185	.125	.195	.120	.185	.125
2250	....	....	....	....	....	....	....	.140	.205	.140	....	....	....	....
2500	.048	.050	.050	....	....	....	....	.155	.230	.155	....	....	....	....
3000	.050	.080	.055	....	....	....	....	.185	.275	.185	.280	.190	.285	.185
3500	.055	.090	.065	....	....	....	....	.220	.325	.215	....	....	....	....
4000	.070	.105	.080	.105	.070	.105	.075	.255	.370	.250	.375	.255	.370	.250
4250	..	....	....	....	....	....	....	.270	.405	.270	....	....	....	....
4500	.084	.115	.090	....	....	....	....	.285	.430	.285	....	....	....	....
4750	....	....	....	....	....	....	....	.302	.455	.300	....	....	....	....
5000	.095	.125	.100	....	....	....	....	.317	.470	.315	.470	.315	.470	.315
5250	....	....	....	....	....	....	....	.335	.495	.330	....	....	....	....
5500	.100	.140	.105	....	....	....	....	.350	.520	.345	....	....	....	....
5750	....	....	..	....	....	....	....	.365	.545	.360	....	..	....	....
6000	.110	.155	.115	.160	.105	.160	.110	.380	.565	.375	575	.380	.575	.380
6500	.120	.170	.120	....	....	....	....	....	....	....	....	....	....	....
7000	.130	.185	.135	....	....	....	....	....	....	....	....	.445	.665	.440
7500	.135	.200	.140	....	....	....	....	....	....	....	....	....	....	....
8000	.146	.210	.150	.215	.145	.215	.150	....	....	....	....	.515	.765	.515
8500	.152	.225	.160	....	....	....	....	....	....	....	....	....	....	....
9000	.163	.240	.170	....	....	....	....	....	....	....	....	.580	.870	.575
9500	.175	.255	.180	....	....	....	....	....	....	....	....	....	....	....
10000	.180	.270	.190	.270	.180	.270	.185	....	....	....	....	.645	.970	.640
10500	.194	.285	.200	....	....	....	....	....	....	....	..	....	....	....
11000	.200	.300	.205	....	....	....	....	....	....	....	....	.715	1.075	.700
11500	.210	.315	.220	....	....	....	....	....	....	....	....	....	....	....
12000	.220	.325	.230	.325	.215	.325	.235	....	....	....	....	.785	1.170	.765
13000	....	....	....	....	....	....	....	....	....	....	....	....	1.310	....
14000	....	....	....	....	.255	.380	.260	....	....	....	....	....	....	....
15000	...	....	....	....	....	....	....	....	...	....	....	....	1.515	....
16000	....	....	....	....	.285	.430	.290	....	....	....	....	....	1.670	....
17000	....	....	....	....	....	....	....	....	....	....	....	....	1.850	....
17400	....	....	....	....	....	....	....	....	....	....	....	....	2.000	...
17500	....	....	....	....	....	....	....	....	....	....	....	....	2.40	....
18000	....	....	....	....	.320	.485	.325	....	....	....	....	...	....	....
20000	....	....	....	....	.360	.545	.370	....	....	....	....	....	....	....
22000	....	....	....	....	.400	.505	.410	....		....	....	....	....	....
24000	...	....	....	....	.440	.665	.450	....	....	....	....	....	....	....
26000	....	....	....	....	....	.725	....	....	....	....	....	....	....	....

TABLE G.—(*Continued*).

Deflections of Beams XX and XXI.

Load in lbs.	XX.							XXI.						
	1st Loading.			Beam gradually relieved of l'd	2nd Loading.			1st Loading.			Beam gradually relieved of l'd	2nd Loading.		
	34½ ins.	End	34½ ins.	Ends.	34½ ins.	Ends	34½ ins.	34½ ins.	E'd:	34½ ins.	Ends.	34½ ins.	Ends	34½ ins.
28000	....	....	....	......	....	.791	.	.....	....	....	......	...	.....	....
30000	....	....	....	......	....	.850	....	.....	....	....	......	....	.....	....
32000	....	....	....	......	....	.920	....	.....	....	....	......	....	.....	....
34000	....	....	....	......	....	.990	....	....	....	....	......	....	.....	....
36000	....	....	....	......	....	1.06	....	.....	....	....	......	....	.....	....
38000	....	....	....	......	....	1.50	....	.....	....	....	......	....	.....	....
40000	....	....	....	......	....	2.40	....	.....	....	....	......	....	.....	....
42000	....	....	....	......	....	3.60	....	.....	....	....	......	....	.....	....
44000	....	....	....	......	....	5.05	....	.....	....	....	......	....	.....	....
46000	....	....	....	......	....	6.60	....	.....	....	....	......	....	.....	....
48000	....	....	....	......	....	7.03	....	.....	....	....	......	....	.....	....

Breaking weight of Beam XX = 49,600 lbs.
" " " XXI = 17,960 "

Tables H and I show deflections in inches of Old Douglas Fir, etc.

TABLE H.

Loads. in lbs.	Deflections of Beams XXII and XXIII.									
	XXII.					XXIII.				
	27 ins.	54 ins.	Ends	54 ins.	27 ins.	31 ins.	62 ins.	Ends.	62 ins.	31 ins.
1,000	......	......	.....	......	......	.015	.01	.015	.00	.01
1,500	.02	.01	.02	.01	.01	.025	.02	.025	.01	.02
2,000	.025	.02	.03	.01	.02	.04	.03	.045	.02	.035
2,500	.04	.025	.04	.02	.03	.05	.045	.05	.025	.045
3,000	.045	.03	.05	.025	.04	.065	.05	.065	.03	.06
3,500	.05	.035	.06	.03	.05	.08	.06	.085	.04	.07
4,000	.06	.04	.07	.035	.06	.10	.065	.105	.045	.085
4,500	.07	.04	.08	.04	.07	.11	.08	.12	.05	.11
5,000	.08	.05	.10	.045	.08	.125	.09	.135	.06	.115
5,500	.09	.055	.12	.05	.09	.14	.095	.150	.065	.13
6,000	.10	.06	.13	.055	.10	.16	.10	.175	.075	.15
6,500	.11	.06	.14	.055	.11	.17	.11	.185	.075	.16
7,000	.12	.07	.15	.06	.12	.18	.12	.20	.085	.175
7,500	.13	.075	.155	.065	.13	.20	.13	.225	.095	.19
8,000	.14	.08	.16	.07	.14	.21	.14	.25	.10	.20
8,500	.15	.085	.17	.075	.15	.225	.145	.255	.11	.215
9,000	.16	.09	.18	.08	.16	.24	.155	.275	.12	.225
9,500	.17	.095	.195	.085	.17	.25	.160	.285	.125	.245
10,000	.18	.10	.20	.09	.175	.26	.17	.305	.13	.255

TABLE H.—(*Continued.*)

Loads in lbs.	Deflections of Beams XXII and XXIII.									
	XXII.					XXIII.				
	27 ins.	54 ins.	Ends	54 ins.	27 ins.	31 ins.	62 ins.	Ends	62 ins.	31 ins.
10,500	.19	.105	.21	.095	.18	.275	.185	.325	.14	.265
11,000	.195	.11	.22	.10	.19	.29	.19	.345	.145	.275
11,500	.20	.115	.23	.105	.20	.305	.20	.355	.15	.30
12,000	.21	.115	.245	.11	.21	.32	.205	.375	.16	.305
12,500	.22	.12	.255	.115	.22	.335	.21	.390	.17	.32
13,000	.23	.125	.265	.12	.225	.35	.225	.415	.175	.34
13,500	.235	.13	.275	.125	.235	.365	.235	.425	.18	.355
14,000	.25	.14	.29	.13	.25	.38	.245	.44	.19	.365
14,500	.255	.145	.30	.135	.26	.395	.25	.455	.20	.38
15,000	.265	.15	.31	.14	.265	.41	.26	.475	.205	.395
15,500	.27	.155	.32	.145	.27	.425	.27	.495	.215	.405
16,000	.28	.16	.33	.15	.28	.44	.275	.505	.22	.42
16,500	.29	.16	.34	.16	.29	.455	.285	.525	.23	.445
17,000	.259	.17	.35	.165	.29	.47	.29	.545	.245	.45
17,500	.30	.175	.36	.165	.31	.485	.30	.555	.245	.465
18,000	.31	.18	.37	.175	.315	.50	.305	.575	.25	.475
18,500	.32	.185	.39	.175	.32	.515	.313	.595	.26	.485
19,000	.33	.19	.39	.18	.33	.53	.32	.605	.265	.50
19,500	.34	.195	.40	.18	.34	.545	.33	.625	.275	.51
20,000	.35	.20	.425	.185	.35	.555	.345	.645	.28	.53
20,500	......	......	.....	......	......	.565	.35	.655	.285	.545
21,000	......	.....	.43	......	......	.580	.360	.675	.305	.56
21,500	......	......	.....	......	......	.59	.37	.695	.305	.57
22,000	......	.....	.45	......	.....	.605	.375	.705	.31	.58
22,500	......	......	.....	......	......	.625	.38	.725	.32	.595
23,000	......	......	......	......	... .	.645	.395	.745	.325	.61
23,500	......	......	.....	......		.65	.40	.765	.335	.625
24,000	......	.....	.....	......	.....	665	.41	.780	.34	.64
25,000	......	......	.51	......	......	......	......	.....	....	....
26,000	......	......	.54	......	......	......	......	.85	....	....
27,000	......	......	.555	......	......	......	......	.....	....	....
28,000	......	......	.57	......	......	......	......	.90	....	....
30,000	.....	.....	.....	.....	......	......	......	1.00	....	....
31,000	......	......	.66	....	......	......	......	.....	....	....
32,000	......	.. ..	.67	......	......	.....	......	1.05	....	....
34,000	......	......	.71	....	......	......	......	1.15	....	....
35,000	....	......	.745	......	...	......	......	.....	....	....
36,000	.....	......	.76	......	......	......	......	1.2	....	....
38,000	......	......	.....	......	......	......	......	1.27	....	....
40,000	......	......	.86	......	......	......	......	1.34	....	....
41,000	.....	.....	.90	......	......	......	......	.....	....	....
42,000	......	......	.....	.. ..	......	......	......	1.45	....	....
44,000	......	......	.975	......	......	......	......	1.53	....	....
45,000	......	......	1.02	......	......	......	.....	.....	....	....
46,000	......	......	....	.....	......	......	......	1.60	....	....
47,000	......	......	1.07	......	..	......	......	.....	....	....
49,000	......	... .	1.10	......	......	......	.....	.....	....	....
51,000	......	......	1.15	......	......	......	......		.	....
53,000	......	......	1.20	......	......	......	......	.....	....	....
55,000	......	......	1.27	......	......	......	......	.....	....	....

Breaking weight of Beam XXII = 55,400 lbs.
" " " XXIII = 47,560 "

TABLE I.

Loads in lbs.	Deflections of Beams XXIV and XXV.									
	XXIV.					XXV.				
	22 ins.	44 ins.	Ends	44 ins.	22 ins.	24 ins.	48 ins.	Ends.	48 ins	24 ins.
500	......	......	....	......	....	.01	.005	.01	.005	.01
1,000	.....	......	....	......	......	.015	.01	.015	.005	.015
2,000	.....	......	....	......	.....	.02	.015	.03	.01	.02
3,000	.....	......	....	....	.....	.04	.025	.05	.015	.04
4,000	......	.....	....	......	......	.06	.035	.075	.025	.06
5,000	.045	.03	.05	.04	.04	.075	.045	.095	.04	.08
6,000	.065	.04	.065	.045	.055	.095	.055	.105	.045	.10
7,000	.08	.04	.08	.05	.06	.115	.065	.140	.055	.115
8,000	.10	.05	.10	.06	.08	.125	.07	.15	.065	.125
9,000	.105	.055	.105	.07	.08	.14	.08	.18	.075	.14
10,000	.12	.06	.12	.07	.095	.155	.09	.195	.08	.155
11,000	.13	.07	.13	.08	.11	.17	.10	.225	.085	.165
12,000	.14	.08	.15	.085	.125	.185	.105	.245	.10	.18
13,000	.145	.085	.16	.09	.14	.205	.115	.26	.105	.21
14,000	.16	.09	.17	.10	.15	.215	.12	.285	.115	.22
15,000	.18	.10	.20	.11	.165	.24	.125	.30	.125	.235
16,000	.20	.105	.21	.12	.17	.255	.14	.325	.13	.255
17,000	.21	.11	.22	.125	.18	.265	.15	.345	.145	.265
18,000	.22	.12	.25	.13	.19	.285	.155	.365	.16	.28
19,000	.225	.125	.25	.14	.205	.30	.16	.395	.17	.305
20,000	.24	.13	.26	.15	.22	.315	.17	.410	.18	.315
21.000	.26	.14	.27	.16	.24	.340	.185	.445	.19	.335
22,000	.27	.145	.29	.17	.25	.355	.195	.465	.20	.355
23,000	.28	.15	.31	.175	.26	......	......	......	....	....
24,000	.30	.16	.32	.18	.27	......	....	.50	....	....
25,000	.31	.17	.335	.185	.275	......	......	......	....	....
25,800	....	....	....	....	....	......	......	.54	....	....
26,000	.32	.175	.35	.195	.29	......	......	......	....	....
27,000	.34	.18	.36	.205	.31	......	......	......	....	....
28,000	.36	.18	.38	.21	.32	......	......	......	....	....
29,000	.37	.19	.40	.22	.33	......	......	......	....	....
30,000	.38	.20	.415	.225	.34	......	......	......	....	....
30,200	....	....	....	....	....	......	......	.65	....	....
31,000	.39	.21	.425	.235	.355	......	......	.....	....	....
32,000	.405	.22	.45	.24	.37	......	......	......	....	....
33,000	......	......	.46	......	.....	......	......	......	....	....
33,200	......	......	....	......	......	......	......	.75	....	....
34,000	......	......	.48	......	......	......	......	......	....	....
36,000	......	......	.51	......	......	......	......	......	....	....
37,000	......	......	.54	......	......	......	......	......	....	....
38,000	....	......	.56	......	......	......	......	......	....	....
39 000	......	......	.575	......	......	......	......	......	....	....
39,700	......	......	....	......	......	......	......	.95	....	....
40,000	......	......	.66	......	......	......	......	......	....	....

Breaking weight of Beam XXIV = 76,900 lbs. for beam of reduced length.

Breaking weight of Beam XXV = 42,900 lbs.

Table J showing deflections in inches of two Douglas Fir planks under gradually increased loads.

TABLE J.

Loads in lbs.	Deflections in ins. of Plank 1.	Deflections in ins. of Plank 2.
	Ends.	Ends.
2,000	.05	.06
3,000	.07	.10
4,000	.10	.15
5,000	.12	.19
6,000	.15	.23
7,000	.16	.27
8,000	.18	.35
9,000	.21	...

Breaking weight of Plank 1 = 22,250 lbs.
" " " 2 = 13,250 "

Tables K to M shew deflections in inches of Canadian New Red Pine Beams.

TABLE K.

Loads in lbs.	Deflections of Beams XXVI to XXVIII.						
	XXVI.					XXVII.	XXVIII.
	35 ins.	70 ins.	Ends.	70 ins.	35 ins.	Ends.	Ends.
1,000	.055	.035	.065	.04	.055	.08	.09
1,500	.110	.060	.135	.060	.110	.15	.15
1,800	.145	.080	.175	.080	.150	......	......
2,000	.165	.095	.200	.09	.165	.20	.225
2,300	.195	.110	.235	.110	.200	......	......
2,500	.215	.125	.260	.125	.215	.26	.300
2,700	.235	.130	.285	.130	.240	......	......
3,000	.265	.150	.320	.150	.265	.32	.36
3,200	.290	.160	.350	.160	.295	......	......
3,500	.320	.180	.385	.180	.320	.37	.44
3,700	.345	.195	.410	.195	.350	......	......
4,000	.370	.210	.450	.210	.370	.44	.50
4,200	.395	.225	.475	.225	.400	......	......
4,500	.430	.245	.510	.245	.430	.49	.575
4,700	.450	.255	.535	.250	.450	......	......
5,000	.480	.270	.570	.265	.475	.55	.65
5,200	.500	.280	.600	.275	.500	......	......
5,500	.535	.295	.635	.290	.530	.60	.72
5,700	.560	.310	.660	.305	.550	......	......
6,000	.580	.330	.700	.320	.580	.66	.79
6,200	.605	.340	.725	.335	.600	......	......
6,500	.635	.360	.755	.350	.635	.73	.86
6,700	.655	.370	.790	365	.655	......	......
7,000	.690	.385	.825	380	.685	.79	.93
7,200	.715	.395	.855	.390	.705	......	......
7,500	.745	.415	.890	.410	.740	.85	1.00
7,700	.765	.425	.915	.425	.755	......	......
8,000	.800	.445	.950	440	.800	.92	1.07
8,200	.820	.455	.980	.455	.815	......	......
8,500	.850	.475	1.020	.470	.855	.99	1.14
8,700	.880	.495	1.050	.485	.875	......	......
9,000	.915	.510	1.100	.510	.915	1.05	1.21
9,200	.945	.525	1.135	525	.945	......	......
9,500	.995	.545	1.185	.545	.985	1.13	1.28
9,700	1.015	.560	1.225	.560	1.010	......	......
10,000	1.050	.585	1.265	.580	1.050	1.20	1.36
10,500	......	......	......	......	......	......	1.43
11,000	......	......	1.400	......	......	1.36	1.50
11,500	......	......	......	......	......	......	1.57
12,000	......	......	1.600	......	......	1.54	1.66
12,500	......	......	......	......	......	......	1.72
13,000	......	......	1.700	......	......	1.63	1.80
13,500	......	......	......	......	......	......	1.87
13,800	......	......	......	......	......	......	......
14,000	......	......	2.050	......	......	......	1.95
14,500	......	......	......	......	......	......	2.06
15,000	......	......	......	......	......	2.00	2.15
15,500	......	......	......	......	......	......	2.30
15,600	......	......	2.750	......	......	......	......
16,000	......	......	3.000	......	......	2.20	2.44
16,500	......	......	......	......	......	......	......
17,000	......	......	......	......	......	2.52	......
17.050	......	......	......	......	......	......	2.80

Breaking weight of Beam XXVI = 16,940 lbs.
" " " XXVII = 17,700 "
" " " XXVIII = 17,050 "

TABLE L.

Loads in lbs.	Deflections of Beams XXIX to XXXII.							
	XXIX.					XXX.	XXXI	XXXII.
	35 in s	70 ins.	Ends.	70 ins.	35 ins.	Ends.	Ends.	Ends.
200	......	....	....	....	....	....	....	.035
500	.030	.015	.04	.015	.020	.130	....	.185
600	......	....	....	....	....	....	....	.235
700	......	....	....	....	....	....	....	.290
800	......	....	....	....	....	.245	....	.340
900	......	....	....	....	......	....	....	.385
1,000	.120	.050	.140	.070	.100	.320	.29	.430
1,100	......	....	....	....	....	....	....	.495
1,200	......	....	....	....	....	....	....	.545
1,300	......	....	....	....	....	.440	.385	.600
1,400	.185	.090	.225	.110	.190	....	....	.650
1,500	......	....	......	....	....	.505	.450	.700
1,600	......	....	....	....	....	....	....	.750
1,700	......	....	....	....	....	.590	.520	.800
1,800	.265	.135	.310	.150	.250	....	....	.855
1,900	......	....	....	....	....	....	....	.915
2,000	.300	.150	.350	.170	.290	.710	.615	.960
2,100	......	....	....	....	....	....	....	1.015
2,200	......	....	....	....	....	....	....	1.075
2,300	......	....	....	....	....	.835	.725	1.145
2,400	.370	.190	.440	.205	.360	....	....	1.195
2,500	......	....	....	....	....	.905	.780	1.245
2,600	......	....	....	....	....	....	....	1.300
2,700	......	....	....	....	....	....	....	1.360
2,800	.440	.235	.525	.250	.435	1.040	.900	1.410
2,900	......	....	....	....	....	....	....	1.465
3,000	.480	.250	.565	.265	.460	1.150	.960	1.525
3,100	......	....	....	....	....	....	....	1.585
3,200	......	....	....	....	....	1.210	1.035	1.625
3,300	......	....	....	....	....	....	....	1.700
3,400	.550	.295	.650	.305	.540	....	....	1.750
3,500	......	....	....	....	....	1.340	1.115	1.800
3,600	......	....	....	....	....	....	....	1.865
3,700	......	....	....	....	....	....	....	1.935
3,800	.620	.330	.740	.350	.610	1.456	1.225	1.990
3,900	......	....	....	....	....	....	....	2.025
4,000	.640	.350	.775	.365	.640	1.550	1.320	2.100
4,100	......	....	....	....	....	....	....	2.170
4,200	......	....	....	....	....	1.640	....	2.220
4,300	......	....	....	......	....	....	1.445	2.290
4,400	.740	.390	.865	.410	.730	....	....	2.355
4,500	......	....	....	....	....	1.765	1.510	2.420
4,600	......	....	....	....	....	....	....	2.470
4,700	......	....	....	....	....	....	....	2.530
4,800	.810	.445	.960	.450	.800	1.900	1.615	2.610

TABLE L.—(*Continued.*)

Loads in lbs.	Deflections of Beams XXIX to XXXII.							
	XXIX.					XXX.	XXXI.	XXII.
	35 ins.	70 ins.	Ends.	70 ins.	35 ins.	Ends.	Ends.	Ends.
4,900	....	....	....	....	....	....	....	2.680
5,000	.850	.460	1.000	.470	.835	2.010	1.700	2.755
5,100	......	....	....	....	....	....	...	2.830
5,200	......	....	......	....	....	2.120	....	......
5,300	......	....	....	....	..	....	1.815	......
5,400	.910	.500	1.085	.515	.900	....	....	......
5,500	......	....	....	....	....	2.335	1.895	......
5,700	....	....	....	....	....	2.515	....	......
5,800	.985	.545	1.175	.560	.990	....	....	......
6,000	1.030	.565	1.225	.580	1.005	2.900	2.115	......
6,400	1.110	.610	1.320	.620	1.100	....	....	......
6.500	......	....	...	....	....	....	2.410	......
6,800	1.170	.640	1.405	.660	1.175	....	....	......
7,000	1.220	.665	1 455	.675	1.210	....	....	......
7,400	1.290	.715	1.555	.740	1.300	..	....	......
7,800	1.360	.755	1.660	.775	1.360	....	....	......
8,000	1.410	.785	1.710	.800	1.410	....	....	......
8,400	1.500	.830	1.810	.850	1.510	....	....	......
8.800	1.590	.880	1.915	.900	1.580	....	....	......
9,000	1.640	.910	2.005	.930	1.650	....	....	......
10,000	......	....	2.270	....	....	....	....	......
11,000	......	....	2.650	....	....	....	....	......

Breaking weight of Beam XXIX = 11,960 lbs.
" " " XXX = 5,700 "
" " " XXXI = 6.500 "
" " " XXXII = 5,200 "

TABLE M.

Loads in lbs.	Deflections of Beams XXXIII to XXXV.		
	XXXIII.	XXXIV.	XXXV.
	Ends.	Ends.	Ends.
500	.065	.080	.030
800	......	.145	.065
1,000	.160	.185	.090
1,200	.205	.230	.125
1,400	.250	.275	.150
1,600	.275	.320	.175
1,800	.325	.360	.195
2,000	.375	.405	.220
2,200	.410	.450	.245
2,400	.465	.490	.270
2,600	.500	.535	.295
2,800	.540	.580	.320
3,000	.585	.625	.345
3,200	.630	.670	.370
3,400	.670	.715	.390
3,600	.710	.760	.415
3,800	.750	.810	.445
4,000	.790	.850	.465
4,200	.830	.900	.490
4,400	.870	.945	.515
4,600	.910	.990	.545
4,800	.950	1.035	.565
5,000	1.000	1.080	.590
5,200	1.040	1.125	.615
5,400	1.090	1.175	.640
5,600	1.125	1.220	.670
5,800	1.165	......	.695
6,000	1.220	......	.720
6,200	1.260	......	.745
6,400	1.310	......	.770
6,600	1.3[illegible]5	......	.800
6,800	1·415	......	.830
7,000	1.455	......	.860
7,200	1.545	......	.885
7,400	1.590	......	.915
7,600	1.640	......	.950
7,800	1.690	......	......
8,200	1.790	......	......

Breaking weight of Beam XXXIII = 9,250 lbs.
" " " XXXIV = 5,600 "
" " " XXXV = 7,600 "

Tables N to Q show deflections in inches of Canadian New White Pine Beams.

TABLE N.

Loads in lbs.	Deflections of Beams XXXVI to XLI.											
	XXXVI.							XXXVII.	XXXVIII.	XXXIX	XL	XLI.
	108 ins.	72 ins.	36 ins.	Ends.	36 ins.	72 ins.	108 ins.	Ends.	Ends.	Ends.	Ends.	Ends.
5000	.109	.30	.30	.32	.30	.29	.109	......	......	......	....	......
7500	.375	.70	.93	1.02	.90	.66	.344	......	..	......	....	......
10000	.594	1.00	1.33	1.45	1.29	.95	.516	......	.10	.11	.11	.13
11000	.719	1.34	1.78	1.95	1.74	1.28	.688	......	......	......	....	......
12500	.799	1.47	1.96	2.16	1.93	1.42	.750	......	.125	.14	....	......
15000	.906	1.68	2.24	2.45	2.20	1.62	.875	......	.15	.165	.17	.20
17500	1.125	2.05	2.70	2.97	2.65	1.96	1.047	......	.19	.19	....	......
20000	....	....	....	....	....	....	....	......	.21	.2255	.23	.29
22000	....	....	....	....	....	....	....	......	......	......	.25	.32
22500	....	....	....	....	....	....	....	......	.245	.2555	....	......
24000	....	....	....	....	....	...	....	......	......	......	.27	.35
25000	....	....	....	....	....	....	....	......	.27	.285	....	......
26000	....	....	....	....	....	....	....	......	......	... .	.30	.40
27500	....	....	...	....	....	....	....	......	.30	.31	....	......
28000	....	....	....	....	....	....	....	......	......	......	.33	.44
30000	....	....	....	....	....	....	....	......	.33	.35	.36	.49
32000	....	...	....	....	....	....	....	.....	......	......	.39	.53
32500	....	...	....	....	....	....	....	......	.37	......	....	......
34000	....	....	....	....	....	....	....	......	......	......	.42	.....
36000	....		....	....	....	....	..	......	......	......	.45	......

Breaking weight of Beam XXXIV = 19,600 lbs.
" " " XXXV = 24,000 "
" " " XXXVI = 52,450 "
" " " XXXVII = 51,400 "

TABLE O.

Loads in lbs.	Deflections of Beams XLII to XLIV.								
	XLII.							XLIII.	XLIV.
	108 ins	72 ins.	36 ins.	Ends.	36 ins.	72 ins.	108 ins	Ends.	Ends.
2500	.0312	.05	.07	.08	.07	.055	.031	......	......
3000	.047	.095	.14	.15	.14	.10	.047	......	......
3500	.078	.13	.18	.19	.18	.13	.078	......	......
4000	.094	.17	.24	.26	.24	.17	.109	......	......
4500	.109	.20	.27	.30	.28	.205	.125	......	......
5000	.125	.245	.33	.37	.34	.25	.141	......	......
5500	.141	.275	.38	.42	.39	.28	.156	......	......
6000	.172	.325	.44	.47	.45	.33	.172	......	......
6500	.187	.35	.49	.53	.49	.35	.188	......	......
7000	.219	.39	.54	.60	.54	.40	.219	......	......
7500	.234	.425	.59	.65	.60	.43	.234	......	......
8000	.250	.47	.64	.71	.65	.47	.266	......	......
8500	.281	.505	.69	.76	.70	.52	.281	......	......
9000	.297	.54	.75	.82	.75	.55	.312	......	......
9500	.312	.59	.80	.90	.81	.60	.328	......	......
10000	.328	.61	.84	.93	.85	.63	.344	.10	.11
10500	.359	.66	.91	1.00	.91	.67	.359	......	......
11000	.375	.70	.97	1.07	.96	.71	.375	......	......
11500	.406	.75	1.03	1.14	1.04	.76	.406	......	......
12000	.422	.77	1.06	1.17	1.07	.79	.422	......	......
12500	.438	.80	1.11	1.21	1.11	.82	.438	......	......
13000	.453	.835	1.16	1.30	1.17	.875	.453	......	......
13500	.484	.905	1.24	1.37	1.25	.93	.484	......	....
14000	.500	.945	1.29	1.44	1.31	.97	.510	......	......
14500	.531	.975	1.34	1.49	1.355	1.00	.531	......	......
15000	.547	1.02	1.40	1.55	1.415	1.02	.562	.16	.16
15500	.562	1.06	1.45	1.61	1.48	1.10	.578	......	......
16000	.593	1.105	1.51	1.68	1.53	1.15	.593	......	......
16500	.609	1.15	1.57	1.76	1.60	1.19	.625	......	......
17000	.641	1.19	1.63	1.81	1.65	1.23	.641	.....	......
17500	.656	1.23	1.68	1.87	1.705	1.27	.672	......	......
18000	.687	1.27	1.75	1.96	1.775	1.32	.687	......	......
18500	.719	1.34	1.84	2.05	1.86	1.39	.734	......	......
19000	.750	1.38	1.89	2.11	1.92	1.43	.750	......	......
19500	.766	1.43	1.95	2.19	1.98	1.47	.766	......	......
20000	.781	1.48	2.02	2.27	2.05	1.52	.797	.23	.24
20500	.813	1.53	2.10	2.35	2.13	1.58	.828	......	......
21000	.844	1.58	2.16	2.42	2.19	1.62	.859	......	......
21500	.875	1.665	2.28	2.55	2.31	1.70	.891	......	......
22000	.924	1.72	2.36	2.65	2.39	1.77	.938	......	......
25000	....	....	....	....	....	....	....	.29	.30

Breaking weight of Beam XXXVIII = 26,350 lbs.
" " " XXXIX = 48,600 "
" " " XL = 51,870 "

TABLE P.

Loads in lbs.	Deflections of Beams XLV to XLVII.								
	XLV.							XLVI.	XLVII.
	108 ins	72 ins.	36 ins.	Ends.	36 ins.	72 ins.	108 ins	Ends.	Ends.
2500	.125	.22	.30	.34	.29	.21	.141	......	.02
3000	.141	.27	.35	.39	.34	.31	.156	......	......
3500	.172	.29	.41	.45	.39	.34	.188	......	......
4000	.188	.34	.45	.50	.44	.36	.203	......	......
4500	.203	.37	.50	.55	.49	.44	.219	......	......
5000	.219	.42	.55	.61	.54	.44	.234	......	......
5500	.234	.45	.60	.67	.59	.47	.250	......	......
6000	.250	.49	.65	.73	.64	.51	.266	......	......
6500	.266	.53	.71	.79	.69	.56	.281	......	......
7000	.297	.56	.76	.84	.74	.59	.312	......	......
7500	.312	.60	.81	.90	.79	.62	.328	......	......
8000	.344	.63	.86	.95	.85	.66	.344	......	......
8500	.359	.67	.92	1.03	.90	.69	.359	......	......
9000	.375	.71	.97	1.08	.95	.74	.391	......	......
9500	.391	.75	1.02	1.14	1.00	.78	.406	......	......
10000	.422	.79	1.08	1.20	1.06	.81	.422	.12	.10
10500	.438	.83	1.14	1.26	1.11	.86	.438	......	......
11000	.453	.87	1.20	1.33	1.17	.90	....	......	......
11500	.484	.92	1.26	1.40	1.24	.95	.500	......	......
12000	.500	.96	1.31	1.47	1.28	.98	.516	......	......
12500	.531	1.01	1.36	1.53	1.34	1.02	.531	......	.13
13000	.547	1.05	1.42	1.59	1.39	1.06	.547	......	......
13500	.563	1.08	1.48	1.66	1.45	1.10	.578	......	......
14000	.593	1.13	1.55	1.73	1.51	1.15	.593	......	......
14500	.625	1.17	1.60	1.79	1.57	1.18	.625	......	......
15000	.641	1.21	1.65	1.86	1.62	1.22	.641	.20	.16
15500	.656	1.25	1.71	1.93	1.69	1.27	.656	......	......
16000	.687	1.30	1.78	2.00	1.75	1.31	.672	......	......
16500	.703	1.35	1.85	2.08	1.82	1.36	.687	......	......
17000	.734	1.39	1.90	2.14	1.86	1.40	.734	......	......
17500	.766	1.43	1.97	2.22	1.94	1.45	.750	......	.20
18000	.781	1.50	2.05	2.33	2.02	1.51	.781	......	......
18500	.797	1.54	2.11	2.39	2.08	1.56	.797	......	......
19000	.828	1.59	2.19	2.48	2.15	1.60	.828	......	......
20000	.875	1.68	2.31	2.63	2.29	1.70	.875	.26	.23
20500	.924	1.75	2.41	2.76	2.38	1.77	.924	......	......
21000	.953	1.82	2.50	2.88	2.47	1.83	.953	......	......
22500	....	....	......	......	......	....	......	......	.26
25000	....	.....	......	......	......		......	.35	.30
27500	....	.....	......	......	......	......	......	......	.34
30000	....	.....	......	....	......	......	......	......	.39

Breaking weight of Beam XLI = 24,850 lbs.
" " " XLII = 44,400 "
" " " XLIII = 48,650 "

TABLE Q.

Loads in lbs.	Deflections of Beams XLVIII to L.								
	XLVIII.			XLIX.			L.		
	37½ ins.	Ends.	37½ ins.	37½ ins.	Ends.	37½ ins.	46½ ins.	Ends.	46½ ins.
1000	.01	.01	.01	.005	.01	.005	.015	.015	.01
2000	.025	.03	.02	.02	.04	.02	.04	.055	.035
3000	.04	05	.035	.035	.06	.035	.07	.105	.065
4000	.055	.065	.052	.05	.08	.05	.10	.15	.10
5000	.065	.085	.06	.065	.10	.065	.135	.195	.135
6000	.08	.105	.075	.075	.125	.08	.165	.245	.165
7000	.10	.125	.08	.095	.15	.095	.20	.295	.20
8000	.105	.15	.103	.11	.17	.105	.22	.33	.225
9000	.12	.17	.11	.125	.20	.13	.25	.375	.255
10000	.135	.195	.125	.14	.22	.14	.28	.43	.28
10500	.14	.215	.135	....	....	....	......	......	......
11000	.15	.22	.143	.155	.25	.15	.30	.46	.30
11500	.155	.23	.15	....	...	....	......	......	......
12000	.165	.24	.155	.175	.265	.165	.33	.50	.33
12500	.175	.25	.16	.18	.275	.17	.35	.53	.35
13000	.18	.265	.165	.19	.29	.185	.36	.55	.36
13500	.185	.27	.17	.20	.30	.195	.375	.57	.375
14000	.19	.285	.177	.21	.315	.20	.39	.60	.39
14500	.20	.295	.19	.215	.32	.21	.41	.615	.40
15000	.21	.305	.20	.22	.35	.215	.42	.645	.42
15500	.215	.32	.205	.225	.355	.22	.43	.655	.43
16000	.22	.33	.21	.235	.365	.23	.445	.67	.45
16500	.23	.34	.223	.245	.375	.24	.46	.70	.46
17000	.235	.355	.23	.25	.39	.25	.475	.72	.475
17500	.24	.365	.235	.26	.405	.255	.49	.745	.50
18000	.25	.38	.24	.27	.415	.26	.51	.76	.51
18500	.25	.395	.25	.275	.425	.27	.525	.795	.52
19000	.265	.405	.255	.285	.44	.28	.54	.82	.55
19500	.27	.415	.26	.295	.455	.29	.55	.84	.56
20000	.275	.425	.27	.30	.465	.30	.57	.865	.58
20500	.285	.445	.285	.31	.475	.31	.585	.895	.59
21000	.295	.46	.29	.32	.495	.32	.60	.92	.61
21500	.30	.47	.295	.325	.505	.325	.62	.94	.63
22000	.31	.485	.305	.34	.515	.335	.635	.965	.64
22500	.32	.50	.31	.345	.52	.34	.65	1.00	.65
23000	.33	.515	.32	.35	.535	.345	......	1.03	......
23500	.335	.53	.33	.36	.555	.35	......	......	......
24000	.35	.54	.34	.37	.57	.36	......	1.07	......
24500	.36	.555	.35	.38	.58	.37	......	......	......
25000	.365	.565	.355	.385	.585	.375	......	1.14	......
25500	.375	.585	.365	.39	.60	.385	......	......	......
26000	.385	.60	.38	.40	.61	.395	......	1.16	......
26500	.395	.615	.385	.415	.625	.405	......	......	......
27000	....	.625	....	.42	.645	.41	......	1.25	......
27500	....	....	....	.43	.66	.42	......	......	......
28000	....	....	....	.445	.675	.43	......	1.33	......
28500	....	....	....	.45	.69	.445	......	......	....

TABLE Q.—(*Continued.*)

Loads in lbs.	Deflections of Beams XLVIII to L.								
	XLVIII.			XLIX.			L.		
	37½ ins.	Ends.	37½ ins.	37½ ins.	Ends.	37½ ins.	46½ ins.	Ends.	46½ ins.
29000	....	......	....	.46	.71	.455	......	1.41	......
29500	....	......	....	.465	.725	.46	......	......	......
30000	....	.69	....	.475	.74	.47	......	1.49	......
31000	....	......	....	....	.78	....	......	1.55	......
32000	....	.76	....	....	....	....	......	1.60	......
34000	....	.85	....	....	....	....	......	......	......
36000	....	.94	....	....	.92	....	......	......	......
37000	....	......	....	....	.98	....	......	......	......
37300	....	......	....	....	1.00	....	......	......	......
38100	....	1.18	....	....	....	....	......	......	......
40000	....	1.25	....	....	1.20	....	......	......	......
41000	....	......	....	....	1.30	....	......	......	......
44000	....	......	....	....	1.50	....	......	......	......
45000	....	1.85	....	....	....	....	......	......	......
46000	....	1.97	....	....	1.70	....	......	......	......
47000	....	2.15	....	....	1.95	....	......	......	......

Breaking weight of Beam XLVIII = 38,100 lbs.
" " " XLIX = 47,080 "
" " " L = 32,200 "

Table R shows deflections in inches of Canadian White Pine Beams which have been in service.

TABLE R.

Loads in lbs.	Deflections of Beams LI to LIII.														
	LI.					LII.					LIII.				
	32 ins.	64 ins.	Ends.	64 ins.	32 ins.	30 ins.	60 ins.	Ends.	60 ins.	30 ins.	30 ins.	60 ins.	Ends.	60 ins.	30 ins.
1000	.02	.02	.035	.02	.02	.02	.01	.025	.01	.02	.03	.01	.04	.02	.03
1500	.05	.03	.065	.03	.05	.05	.02	.055	.025	.05	.055	.02	.065	.04	.06
2000	.06	.05	.09	.05	.07	.060	.040	.075	.040	.070	.08	.04	.10	.05	.085
2500	.10	.065	.12	.05	.10	.09	.05	.105	.05	.095	.11	.06	.135	.065	.11
3000	.11	.08	.145	.07	.12	....	....	....	....	....	.135	.08	.16	.08	.14
3200	....	....	....	....	....	.12	.06	.135	.07	.125	....	....	....	....	....
3500	.14	.09	.175	.085	.15	.14	.07	.155	.08	.145	16	.095	.20	.09	.16
4000	.17	.10	.21	.10	.175	.16	.08	.185	.09	.16	18	.105	.235	.10	.19
4500	.19	.12	.24	.115	.20	.18	.10	.21	.11	.18	.21	.11	.26	.12	.22
5000	.21	.13	.265	.13	.23	.20	.105	.235	.12	.205	.235	.13	.28	.13	.24
5500	.25	.14	.30	.145	.25	....	....	....	....	....	.26	.145	.325	.15	.27
5700	....	....	....	....	....	.22	.12	.265	.13	.245	....	....	....	....	....
6000	.27	.15	.325	.16	.275	.245	.13	.285	.14	.25	.29	.16	.35	.165	.30
6500	.29	.17	.35	.17	.30	.26	.14	.31	155	.275	.31	.18	.39	.18	.32
7000	.31	.185	.385	.185	.33	.29	.15	.345	.175	.30	.34	.19	.42	.19	.35
7500	.345	.20	.415	.20	.35	....	....	....	....	....	.37	.20	.45	.21	.385
7800	....	....	....	....	....	.31	.16	.375	.19	.325	....	....	....	....	....
8000	.35	.21	.445	.215	.375	.34	.17	.40	.20	.35	.40	.22	.49	.23	.405
8500	.38	.225	.47	.235	.40	.35	.185	.415	.215	.36	.425	.24	.515	.24	.44
9000	.40	.23	.50	.25	.425	.375	.195	.445	225	.39	.455	.25	.55	.255	.46
9500	.425	.25	.53	.26	.45	.40	.21	.475	.24	.41	.47	.27	.585	.27	.495
10000	.45	.26	.555	.285	.48	.42	.22	.50	.25	.435	.505	.285	.615	.285	.52
10500	.47	.27	.585	.29	.50	.45	.24	.535	.27	.46	.53	.29	.65	.30	.55
11000	.50	.29	.615	.305	.53	....	....	..	....	...	.565	.305	.69	.31	.58
11500	.515	.30	.65	.315	.55	.47	.25	.56	.28	.485	.59	.32	.725	.33	.60
12000	.55	.31	.67	.33	.58	....	....	....	....	....	.625	.34	.76	.35	.64
12500	.57	.33	.70	.35	.60	.51	.27	.615	.31	.53	.65	.355	.795	.365	.665
13000	.60	.34	.735	.36	.63	.55	.30	.655	.33	.57	.675	.365	.825	.39	.69
13500	.62	.35	.76	.37	.66	.57	.31	.685	.345	.59	.71	.385	.855	.405	.72
14000	.65	.365	.79	.39	.685	.60	.32	.71	.355	.61	.74	.405	.90	.42	.75
14500	.67	.38	.82	.40	.71	.615	.34	.74	.37	.64	.77	.42	.94	.43	.79
15000	.70	.39	.85	.415	.735	.64	.35	.765	.385	.655	.80	.435	.985	.45	.815
15500	.725	.41	.875	.435	.76	.66	.36	.79	.39	.68	.835	.46	1.02	.47	.85
16000	.75	.42	.91	.445	.785	.69	.38	.83	.415	.71	.87	.47	1.07	.48	.89
16500	.77	.435	.94	.455	.81	...	....	....	....	....	....	....	....	....	....
17000	.80	.45	.97	.47	.84	.72	.395	.865	.43	.74	....	....	1.15	....	....
17500	.82	.47	1.00	.49	.86	.76	.415	.915	.45	.78	....	....	....	....	....
18000	.85	.475	1.03	.51	.89	.79	.44	.95	.47	.81	....	...	....	....	....
18500	.88	.49	1.07	.53	.925	...	....	....	....	...	....	....	....	....	....
19000	.90	.50	1.10	.54	.96	....		.985	....	....	...	...	....	....	....
19500	.93	.52	1.14	.56	.985	....	....	....	....	....	...	....	....	....	....
20000	.96	.54	1.185	.60	1.03	....	....	1.06	...	....	...	....	.	....	....
20500	1.00	....	1.235	...	1.07	....	....	....	....	....	....	....	....	....	....
21000	1.04	...	1.28	...	1.11	....	....	1.10	....	....	....	....	....	...	....
21500	....	....	1.32	....	....	....	....	....	....	....	....	....	....	....	....
22000	....	....	....	....	....	....	....	1.18	....	....	....	....	....	....	....
22650	...	....	1.40	....	....	....	....	....	....	....	...	....	...	....	...
23500	....	....	....	....	....	....	....	1.30	....	....	....	....	....	....	....
24000	....	....	....	....	....	....	....	1.34	....	....	....	...	....	....	....
25000	....	....	....	....	....	....	....	1.46	....	....	...	..	...	....	....

Breaking weight of Beams LI = 22,730 lbs.
" " " LII = 26,320 "
" " " LIII = 18,600 "

Tables S and T shew deflections in inches of Canadian New Spruce Beams (B.C.).

TABLE S.

Loads in lbs.	Deflections of Beam LIV.						
	108 ins.	72 ins.	36 ins.	Ends.	36 ins.	72 ins.	108 ins.
1,000	.14	.22	.30	.30	.26	.20	.11
1,500	.15	.24	.33	.34	.30	.23	.12
2,000	.17	.28	.37	.38	.34	.25	.15
2,500	.18	.31	.41	.43	.38	.28	.16
3,000	.19	.34	.44	.46	.42	.31	.18
3,500	.21	.36	.48	.51	.45	.34	.19
4,000	.22	.39	.52	.56	.50	.37	.21
4,500	.24	.42	.56	.60	.54	.39	.22
5,000	.25	.45	.60	.64	.57	.42	.24
5,500	.26	.47	.63	.68	.60	.45	.25
6,000	.27	.50	.67	.72	.64	.48	.26
6,500	.29	.53	.71	.76	.67	.50	.28
7,000	.31	.56	.75	.80	.71	.52	.30
7,500	.32	.59	.79	.84	.75	.56	.31
8,000	.34	.61	.82	.88	.79	.60	.32
8,500	.35	.65	.86	.92	.83	.61	.34
9,000	.37	.67	.90	.97	.86	.65	.35
9,500	.38	.70	.94	1.01	.90	.67	.36
10,000	.40	.73	.97	1.05	.94	....	.39
10,500	.41	.76	1.01	1.09	.98	.71	.40
11,000	.43	.79	1.05	1.14	1.02	.72	.41
11,500	.44	.84	1.09	1.17	1.05	.75	.43
12,000	.46	.84	1.13	1.21	1.09	.78	.45
12,500	.48	.87	1.16	1.26	1.14	.82	.46
13,000	.49	.89	1.19	1.29	1.16	.83	.48
13,500	.50	.92	1.23	1.34	1.20	.84	.49
14,000	.51	.95	1.27	1.38	1.24	....	.50
14,500	.53	.98	1.30	1.42	1.28	....	.51
15,000	.54	.99	1.32	1.45	1.31	....	.53
15,500	.55	1.00	1.32	1.46	1.32	.99	.54
16,000	.55	1.00	1.33	1.48	1.34	1.01	.54
16,500	.55	1.01	1.34	1.50	1.35	1.02	.55
17,000	.56	1.01	1.34	1.51	1.36	1.03	.56
17,500	.56	1.02	1.35	1.52	1.40	1.05	.57
18,000	.56	1.03	1.35	1.54	1.41	1.06	.58
18,500	.57	1.03	1.36	1.55	1.43	1.07	.59
19,000	.57	1.04	1.36	1.57	1.45	1.09	.60
19,500	.58	1.04	1.36	1.58	1.46	1.11	.60
20,000	.58	1.05	1.37	1.60	1.47	1.12	.61
20,500	.71	1.32	1.52	1.93	1.74	1.30	.70
21,000	.72	1.35	1.80	1.98	1.78	1.33	.71
21,500	.74	1.38	1.85	2.02	1.82	1.36	.73
22,000	.76	1.41	1.90	2.07	1.86	1.38	.75
23,400	....	....	....	2.20	....	....	....
26,200	....	....	....	2.50	.	....	....
27,800	....	....	....	2.75	...	....	....
29,000	....	....	....	2.85	...	....	....
29,900	....	....	....	3.00	....	....	....
30,800	....	....	....	3.15	....	....	....
32,000	....	....	....	3.25	....	....	....
32,500	....	....	....	3.35	....	....	....
33,200	....	....	....	3.70	....	....	....
33,500	....	....	....	3.80	....	....	....
33,800	....	....	....	4.00	....	....	....
34,400	....	....	....	4.10	....	....	....
34,800	....	....	....	4.25	....	....	....
35,600	....	....	..	4.50	....	....	....
36,200	....	....	....	4.60	....	....	....
36,300	....	....	....	4.75	....	....	....
36,600	....	....	....	4.90	....	....	....
36,800	....	....	....	5.00	....	....	....
38,250	....	....	....	5.50	....	....	....

Breaking weight of Beam LIV = 36,800 lbs.

TABLE T.

Loads in lbs.	Deflections of Beams LV and LVI.					
	LV.			LVI.		
	30 ins.	End.	30 ins.	30 ins.	End.	30 ins.
10,000	.05	.09	.05	.1	.07	.0
11,000	.06	.10	.06	.11	.09	.06
12,000	.07	.10	.065	.12	.10	.06
13,000	.07	.11	.07	.13	.10	.07
14,000	.08	.11	.075	.13	.11	.08
15,000	.08	.12	.08	.135	.12	.09
16,000	.09	.13	.085	.14	.13	.09
17,000	.10	.14	.09	.145	.14	.095
18,000	.10	.15	.095	.15	.15	.10
19,000	.11	.16	.105	.16	.15	.105
20,000	.11	.17	.11	.16	.16	.11
21,000	.12	.17	.12	.17	.17	.115
22,000	.12	.18	.125	.175	.18	.12
23,000	.13	.19	.13	.185	.19	.12
24,000	.13	.20	.135	.19	.19	.13
25,000	.14	.21	.14	.195	.20	.14
26,000	.15	.22	.145	.2	.20	.15
27,000	.15	.23	.15	.2	.22	.15
28,000	.16	.24	.16	.215	.24	.16
29,000	.16	.25	.165	.22	.24	.16
30 000	.17	.26	.17	.225	.25	.17
31,000	.17	.27	.18	.23	.26	.17
32,000	.18	.28	.185	.235	.27	.18
33,000	.19	.29	.19	.24	.28	.185
34,000	.20	.30	.20	.245	.29	.19
35,000	.20	.31	.205	.255	.29	.20
36,000	.21	.32	.21	.267	.31	.20
37,000	.21	.33	.215	.27	.32	.21
38,000	.22	.34	.225	.28	.33	.215
39,000	.22	.35	.23	.28	.34	.225
40,000	.23	.36	.24	.285	.35	.235
41,000	.24	.37	.25	.29	.36	.24
42,000	.25	.38	.255	.30	.37	.25
43,000	.25	.39	.26	.31	.39	.255
44,000	.26	.40	.27	.32	.40	.26
45,000	.27	.41	.28	.325	.41	.27
46,000	.27	.42	.29	.335	.42	.28
47,000	.28	.44	.30	.34	.45	.285
48,000	.29	.45	.305	.35	.46	.30
49,000	.30	.46	.315	.36	.47	.305
50,000	.31	.48	.32	.37	.49	.315
51,000	.31	.50	.33	.38	.50	.325
52,000	....	....	....	.39	.52	.34
53,000	....	....	....	.40	.55	.35
54,000	....	....	....	.41	.56	.36
55,000	....	....	....	.42	.59	.37
56,000	...	....	....	.44	.60	.39

Breaking weight of Beam LV = 73,000 lbs.
" " " LVI = 70,000 "

Table U and V show deflections of Canadian Spruce Beams which have been in service.

TABLE U.

Loads in lbs.	Deflections of Beams LVII to LIX.						
	LVII.			LVIII.			LIX.
	45 ins.	Ends.	45 ins.	45 ins.	Ends.	45 ins.	At End.
1.000	.01	.02	.01	.030	.040	.040	......
1,500	.02	.05	.025	.050	.065	.056	......
2 000	.035	.07	.05	.060	.100	.070	.09
2,500	.05	.09	.07	.080	.130	.095	......
3,000	.06	.11	.09	.100	.160	.115	......
3,500	075	.14	.10	.120	.190	.130	......
4.000	.09	.15	.115	.140	.215	.150	.20
4,500	.10	.17	.135	.160	.250	.170	......
5,000	.115	.20	.15	.175	.270	.190	.25
5.500	.13	.22	.165	.200	.300	.205	......
6.000	14	.24	.19	.210	.330	.225	.30
6,500	.16	.26	.20	.240	.360	.248	......
7,000	.17	.28	.21	.255	.390	.251	.36
7,500	.185	.30	.22	.275	.420	.285	......
8,000	.20	.33	.235	.300	.450	.305	.41
8,500	.21	.35	.25	.315	.475	.320	......
9,000	.225	.37	.26	.340	.500	.342	......
9.500	.235	.39	.275	.350	.535	.362	......
10,000	.25	.41	29	.375	.570	.380	.52
10,500	.265	.44	.30	.400	.590	.400	......
11,000	.275	.46	.315	.410	.620	.415	......
11,500	.29	.47	.33	.440	.650	.440	......
12,000	.30	.50	.35	.450	.675	.460	......
12,500	.32	.52	.36	.475	.705	.480	......
13,000	.335	.54	.37	.500	.745	.500	......
13,500	.35	.55	.39	.510	.765	.515	......
14,000	.36	.57	.40	.540	.800	.540	......
14,500	.37	.60	.415	.550	.840	.555	......
15,000	.39	.62	.43	.575	.860	.580	......
15,500	.40	.65	.45	.600	.900	.620	..
16,000	.415	.67	.46	.615	.920	.630	
16,500	.435	.69	.47	.640	.960	.645	..
17,000	.45	.72	.49	.655	.990	.665	......
17,500	.46	.74	.50	......	1.025	......	...
18,000	.475	.76	.52	......	......	......	....
18,500	.50	.78	.54	......	......	......	....
19.000	.51	.80	.56	......	1.120	......	......
19,500	.525	.83	.575	......	......	......	......
20,000	.55	.87	.59	......	1.180	......	......
21,000	......	.92	......	......	1.270	......	......
22,000	......	97	......	......	1.350	......	......
23,000	......	1.10	......	......	1.430	......	......
24,000	......	1.50	......	......	1.570	......	......
25.000	......	2.40	......	......	......	......	......
26,000	......	......	......	......	1.850	......	......
27,000	.....	......	......	......	2.040	......	......

The Breaking weight of Beam LVII = 25,700 lbs.
" " " LVIII = 27,470 "
" " " LIX = 21,700 "

TABLE V.

Loads in lbs.	Deflections of Beams LX to LXI.					
	LX.			LXI.		
	34 ins.	At End.	34 ins.	46 ins.	At End.	46 ins.
500	......	......	......	.015	.02	.01
1,000	.005	.015	.005	.04	.05	.03
1,500	.005	.045	.015	.06	.09	.05
2,000	.020	.050	.020	.085	.14	.07
2,500	.035	.070	.035	.105	.17	.10
3,000	.045	.080	.045	.135	.20	.12
3,500	.055	.100	.055	.150	.24	.15
4,000	.065	.120	.065	.180	.290	.170
4,500	.070	.140	.070	.20	.320	.190
5,000	.080	.145	.080	.23	.350	.210
5,500	.095	.165	.160	.245	.390	.245
6,000	.105	.185	.105	.265	.430	.260
6,500	.115	.200	.115	.29	.46	.28
7,000	.130	.220	.130	.31	.51	.31
7,500	.140	.240	.145	.34	.54	.335
8,000	.155	.255	.155	.36	.57	.355
8,500	.175	.285	.170	.39	.61	.38
9,000	.180	.300	.185	.41	.65	.40
9,500	.190	.320	.195	.435	.70	.43
10,000	.205	.345	.205	.455	.74	.45
10,500	.220	.365	.220	49	.76	.485
11,000	.230	.380	.230	.51	.79	.50
11,500	.250	.415	.255	.54	.85	.54
12,000	......	.440	......	......	92	......
13,000	......	.457	......	......	.95	......
14,000	......	.510	......	......	1.03	......
15,000	......	.565	......	......	1.08	......
16,000	......	.610	......	......	1.20	......
17,000	......	.690	......	......	1.32	......
18,000	......	.750	......	......	1.41	......
19,000	......	.870	......	......	......	......
20,500	......	.090	......	......	......	......

Breaking weight of Beam LX = 16,050 lbs.
" " " LXI = 18,400 "

COMPRESSIVE STRENGTH.

The experiments to determine the compressive strength of the various timbers have been chiefly made with columns cut out of the sticks already tested transversely. These columns were, in the first place, carefully examined to see that they had suffered no injury. The following inferences may be drawn :—

(1) The compressive strength of Douglas Fir and of other soft timbers is much less near the heart than at a distance from the heart.

Attention may be directed to the case of three equal specimens A, B and C (see photograph page 19), cut out of Beam XIII. The compressive strength of C was found to be 7,706 lbs. per square inch as compared with 6,653 lbs. per square inch, the compressive strength of A. The difference of strength is undoubtedly due to the very much larger proportion of soft to hard fibre, or of summer to spring growth in C, as compared with the proportion in the case of A. The compressive strength of the timber increases with the density of the annual rings.

(2) When knots are present in a timber column, the column will almost invariably fail at a knot or in consequence of the proximity of a knot.

(3) Any imperfection, as, for example, a small hole made by an ordinary cant hook, tends to introduce incipient bending, or crippling.

(4) When the failures of average specimens commence by an initial bending, the compressive strengths of columns of about 10 to 25 diameters in length agree very well with the results obtained by Gordon's formula, the co-efficients of direct compressive strength per square inch being 6,000 lbs. for Douglas Fir and 5,000 lbs. for White Pine.

Gordon's formula, however, is not at all applicable in the case of specially good or bad specimens. It is often found that a very clear, sound specimen, of even more than 20 diameters in length, will show no signs of bending, but will suddenly fail by crippling under a load as great as that sufficient to crush a shorter specimen.

(5) The greatest care should be observed in avoiding obliqueness of grain in columns, as the *effective* bearing area, and therefore also the strength, are considerably diminished.

(6) If the end bearings are not perfectly flat and parallel, the columns will in all probability fail by bending concave to the longest side.

(7) The *average* strength per square inch, independent of the ratio of length to diameter, is:

5974	lbs. for New Douglas Fir
6265	" for Old " "
4067	" for New Red Pine
3843	" for New White Pine
2772	" for Old " "
3617	" for New Spruce (B.C.)
5136	" Old Spruce

It should be pointed out that none of the old Douglas Fir columns exceeded 4.4 diameters in length, while the great majority of the new Douglas Fir columns were from 4 to 25 diameters in length. This explains the reason of the greater average compressive strength of the old Douglas Fir. A similar remark applies to the New and Old Spruce.

Table giving in detail the results of the experiments on the different specimens :—

RESULTS OF COMPRESSION TESTS ON

NEW DOUGLAS FIR.

Dimensions in ins. Lengths.	Breaking load in lbs. per sq in.	Weight in lbs. per cub. ft.	Remarks.
3.07 × 3.08 × 3.11	6367		Failed by bulging.
3.06 × 3.08 × 3.10	5760		Failed by folding.
2.63 × 3.63 × 5.81	4923	30.3	Specimen 3 ins. or 4 ins. from heart; grain straight; one small knot on high edge. Failed by crippling at knot on high edge.
3.65 × 3.65 × 6.12	3678	29.8	Heart piece; grain straight but seasoned; annual rings very wide; two knots, one on high edge. Failed at this latter by crippling.
2.19 × 3.74 × 5.40	4761	38.4	Straight grained; one large knot from side to side; specimen 3 ins. or 4 ins. away from centre. Failed at knot.
4.10 × 4.30 × 8.05	5218	32.9	Large knot on one end; many small knots all through piece; also heavy season cracks. Failed by bursting along season cracks and through knots.
2.15 × 2.25 × 9.2	5809	38.8	All clear. Failed by crippling.
2.17 × 2.25 × 9.14	7313	35.1	Sound, clear and straight grained; small deficiency on one side at end. Failed by crippling.
2.12 × 2.16 × 9.15	7294	38.7	Straight grained; clear on three sides; 4th side old, with bad defect 4 ins. from one end. Bulged and failed at defect.

2.22 × 2.22 × 9.07	8177	37.5	Straight grained and clear; one bad season crack. Failed by crippling.
2.13 × 2.20 × 9.15	6850	36.5	Straight grained; small knot near one corner 3 ins. from end. Failed at this knot.
3.32 × 3.32 × 9.62	3810	29.5	Heart piece; straight grained; two heavy season cracks; three or four pin knots. Failed by bulging on season cracks; and crippling through two pin knots on same side.
3.33 × 3.34 × 10.58	4388	33.0	Clear; straight grained. Failed on high side. Specimen 3 ins. or 4 ins. from heart.
3.45 × 3.50 × 10.60	7000	32.6	Clear and straight grain; somewhat shaken; crippled 6 ins. from end.
2.74 × 4.27 × 11.25	6837	35.3	Clear and straight grained; some season cracks; failed by crippling directly across about 1½ ins. from one end.
2.85 × 4.25 × 11.27	5615	30.0	Clear and straight grained, but season cracks along annual rings, and one heavy season crack along medullary rays. Failed first by bursting apart of piece at a season crack, then by crippling of the remainder.
3.94 × 3.95 × 11.97	7069	33.8	Clear straight grain; season crack on one side. Failed by crippling at middle on the highest edge.
2.72 × 2.92 × 11.85	8942	40.0	Clear and straight grain; shaken over 8 ins. crippled 4 ins. from end.
3.46 × 3.48 × 12.04	5481	30.4	Two sets of knots, one at one end, the other at centre. Failed at both by crippling, at same time.
4.05 × 4.10 × 12.01	5542	35.1	Knots (heavy) on one end; also several near other end; grain curved at various places due to knots. Grain bent at knot at end.

2.85 × 3.75 × 12.5	6155	38.3	All clear. Failed by crippling.
2.92 × 3.79 × 12.5	5966	39.3	All clear. Failed by crippling.
2.9 × 4.37 × 12.0	6265	35.5	One old side; grain straight and parallel; one side inclined 1 in. in 12 ins.; on other side, two season cracks. Failed by crippling.
2.79 × 3.43 × 12.0	5363	35.7	One old side; grain straight and nearly parallel; no seasoning cracks. Failed by crippling.
2.92 × 4.42 × 12.0	5262	34.2	One old side; grain straight and parallel; one season crack. Failed by crippling.
2.87 × 3.39 × 12.0	6784	35.1	Two old sides; grain nearly parallel; no season cracks. Failed by crippling.
2.93 × 3.42 × 12.03	5520	33.9	Clear and straight grained; one old side with deep seasoning cracks; a slight crack through centre of piece. Crippled 4 ins. from end, and bulged along season crack.
2.80 × 4.40 × 12.0	5069	36.4	Straight grained; one old side with many season cracks. Failed by splitting down season cracks and afterwards crippling.
2.78 × 4.38 × 12.0	6500	35.5	Straight grained and clear; one old side with season crack nearly across piece. Crippled 3 ins. from one end.
2.82 × 3.48 × 12.02	6010	35.9	Grain straight; two old sides; piece sound, no flaws. Crippled near one end.
3.3 × 3.98 × 12.0	5560	34.2	Grain straight and clear, except small pin knot on a corner 4 ins. from end; had two bad season cracks the whole length. Crippled 4 ins. from end induced by season cracks; also bulged out.

3.38 × 3.43 × 13.53	6816	34.7	Clear; grain bent out of straight at one end, due to proximity of knot, also somewhat shaken. Failed by bursting along fibres out of parallel.
2.20 × 2.24 × 13.78	5638	34.3	Grain out of parallel for 1 in in length; knot on one corner of end. Burst along shaken fibres out of parallel.
3.38 × 3.45 × 13.90	6861	33.8	Straight grained, except one-half of a knot on one end. Failed by crippling near knot at end.
4.03 in diar. × 48.01	5856	31.3	Grain parallel, no knots; two small cracks and a small split; annual rings nearly straight. Failed by bending concave to a high corner.
2.84 × 4.23 × 13.12	5828	31.5	Straight grained; small pin knot 3 ins. from one end; season cracks from end to end through middle, passing through knot. Failure by opening of season cracks, and crippling through knot.
4.10 × 4.45 × 14.47	7188	39.1	Clear; grain out of parallel. Failed by crippling and shearing of unsupported fibres.
2.70 × 2.90 × 15.96	8365	39.5	Clear, straight grain shaken over a length of 11 ins. Crippled 5 ins. from end.
2.16 × 2.20 × 16.29	6442	36.0	Clear, not straight grain; somewhat shaken; sheared along shake in grain which being cut off parallel had no bottom support.
4.08 in diar. × 24.12	6595	31.8	Clear and straight grained. Failed by crippling 10 ins. from end.
2.70 × 4.20 × 16.45	6349	30.8	Straight grained; season cracks on one side; several small pin knots. Failed by crippling 2 ins. from one end through one of the pin knots.

2.38 × 3.56 × 16.74	7143	33.0	Straight grain; some small pin knots. Crippled through the largest one at centre.
1 73 × 5.98 × 17.73	4209	38.7	Grain parallel knot on edge 4 ins. from end; also bad season crack and small deficiency in one corner for 6 ins. from one end. Burs at knot and split along season crack.
17 × 2.25 × 17.42	7700	35.6	Clear, straightgrained. Failed by bending and crippling 3 ins. from end.
3.11 × 4 00 × 17.49	4702	33.2	Two heavy knots at centre, one running from side to side through centre; grain crooked and not parallel. Failed by grain shearing and bursting through knot at centre.
3.12 × 4.03 × 17.70	4217	34.2	One heavy knot at centre running from corner to corner, other smaller knots; grain crooked and out of parallel. Crippled at knot at centre.
1.75 × 5.82 × 17.79	5135	37.8	Grain straight and sound; season cracks in centre. Failed by crippling at both ends and also by bending, which probably first caused failure.
3.95 × 5.81 × 17.80	6432	39.1	Grain clear and straight, but not parallel; slight season cracks. Failed by cripple across 4 ins. from one end.
3.95 × 5.92 × 17.82	5359	38.0	Grain clear and straight; some season cracks. Crippled 6 ins. from end.
4.97 × 4.95 × 17.83	4504	37.9	Grain straight and parallel; bad knot 7 ins. from end passing through piece. Failed by bursting at knot and along grain.
1.71 × 5.95 × 17.84	5464	36.0	Grain parallel and clear; bad season crack through heart. Failed by bending at centre. Crippled on concave side.

1.79 × 6.00 × 17.85	6034	36.3	Grain straight and clear; bad season cracks; also chip out on a corner 4 ins. from one end. Failed at sound end by crippling and by opening of season crack.
3 95 × 5.95 × 17.89	6225	38.9	Clear and straight grained; slight season checks. Crippled 3 ins. from one end.
4.08 × 4.45 × 19.68	6437	36.7	Clear, but badly out of parallel. Failed by bursting along fibres out of parallel.
3.02 × 4.01 × 19.97	3240	30.8	Two heavy knots at centre, one also at one end, several other smaller ones. Failed by bursting down centre through knots.
3.85 × 3.91 × 24.05	5382	35.2	Grain straight; two knots on adjacent sides, one at 8 ins. from each end; season cracks running diagonally at one end. Failed by crippling at large knot.
4.35 × 4.85 × 29.75	3630	28.0	Failed by shearing and crippling; grain clear, but not quite parallel.
2.20 × 2.24 × 21.05	7424	35.0	Clear, and straight grained; tested before as pillar. Failed by bending 4 ins. from end.
2 92 × 3.30 × 24.27	4606	34.6	Straight grain; knot 6 ins. from end passing through a corner. Crippled at knot.
2.60 × 3.23 × 25.4	4416	34.7	Straight grain; large knot 4 ins. from end on an edge. Failed by crippling at knot.
2.27 × 2.28 × 23.46	4363	36.91	Straight grained; clear except part of knot on one end. Failed by crippling at knot.
4.20 × 4.36 × 27.88	2622	32.4	Heart; grain 2½ ins. out of straight; heavy season cracks; two large knots. Failed by bulging along season crack and at knots 14 ins. from end.

4.05 × 4.20 × 24.70	5026	33.9	Tested before as pillar, failed then at 67,200 lbs. This portion had straight grain; two knots close together 8 ins. from one end going through piece. Failed by crippling at these knots.
2.61 × 2.65 × 24.42	6237	36.0	Straight grain; season crack across end running half the length of the piece; knot 3 ins. from other end ½ in. in diameter. Crippled at the knot.
2.65 × 2.66 × 26.24	6865	36.4	Straight grained and clear; season crack running down about 8 ins. Crippled clean across at foot of season crack, apparently not induced by seasoning.
2.00 × 2.01 × 27.40	6841	34.5	Clear and straight grain; heavy season crack. Burst from end to end on season crack.
2.88 × 2.95 × 23.91	8106	38.8	Clear, straight grained. Crippled 8 ins. from one end.
2.87 × 2.93 × 25.00	6600	35.5	Clear, nearly straight grained; slight season crack. Failed by a bulging on season crack and afterwards crippled on reduced section at centre.
2.88 × 2.90 × 24.40	7856	36.4	Clear, straight grained. Failed by direct cripp'g.
2.87 × 2.90 × 24.55	8065	38.0	Clear and straight grained. Failed by direct crippling 8 ins. from end.
2.90 × 2.95 × 25.70	8023	36.3	Clear and straight grained. Failed by direct crippling 15 ins. from end.
2.78 × 2.87 × 25.95	9700	40.9	Deficiency near centre, about ½ in. by 1 in. (resin); fibre crooked through vicinity of knot; otherwise clear and straight grained. Failed at crooked fibres at deficiency.

2.89 × 2.90 × 26.69	8269	33.4	Clear and straight grained; failed by compression of fibres on a corner.
2.82 × 2.97 × 25.15	9104	40.2	Very heavy summer rings; clear; fibres bent 12 ins. from one end at one side due to vicinity of a knot. Failed at crooked fibres.
4.77 × 5.82 × 26.15	7709	36.5	Did not fail.
4.77 × 4.68 × 22.32	8411		Same as preceding with piece cut off; clear and straight grain.
4.70 × 5.85 × 25.78	6653	29.2	Straight grained; one knot from side to side at centre. Failed by crippling and bulging at knot.
2.27 × 2.27 × 31.0	3823	37.2	Grain not straight; one pin knot; also knot on one edge 12 ins. from end. Failed by bending at knot on high corner.
3.38 × 4.33 × 32.20	6425	41.3	Clear, straight grained. Crippled 1 ft. from end.
3.39 × 4.42 × 30.90	5935	37.8	Clear, straight grained; external fibre burst; then crippled near centre.
3.38 × 4.42 × 32.32	6111	43.3	Clear, straight grained; burst, then crippled at centre.
3.37 × 4.38 × 32.5	5420	38.9	Clear, straight grained; season crack on one side; small season crack across end. Crippled near end.
3.35 × 4.36 × 31.55	6486	43.1	Clear and straight grained. Crippled near end.
3.41 × 4.45 × 32.4	5880	37.6	Clear and straight grained. Crippled near end.
3.27 × 3.42 × 31.75	5760	33.5	Straight grained; knot ½-in diar., from side to side. Failed by crippling at this knot 8 ins. from one end.
2.65 × 2.86 × 30.65	8047	36.3	Clear, straight grained. Failed by crippling 8 ins. from one end.
2.67 × 2.88 × 31.83	7607	35.3	Clear straight grained. Failed by crippling and bending at same instant at centre.

3.28 × 3.45 × 33.81	6940	35.7	Clear, and straight grained. Failed by bending 10 ins. from one end.
2.75 × 2.82 × 30.47	5480	33.0	Nearly straight grained; various small knots, one larger knot ¾ in. diar. 3 ins. from one end. Failed by crippling at this knot; also somewhat seasoned at heart.
2.90 × 2.90 × 29.35	6183	32.7	Straight grained; various small knots, one larger knot ⅝ in. diar. 9 ins. from end. Failed by crippling at this knot.
2.75 × 2.88 × 31.50	5871	36.4	Straight grained; knot ¾ in. diar. 12 ins. from end. Crippled at the knot.
2.17 × 2.18 × 30.00	6174	35.0	Straight grained, clear but for one knot 10 ins. from end ½ in. in diar. Crippled at this knot.
2.73 × 2.85 × 28.74	8124	34.8	Clear and straight grained. Failed by a thin layer bursting out, and then a clean cripple 8 ins. from same end.
4.69 × 5.84 × 28.10	6677	31.1	Clear and straight grained; crippled 8 ins. from end.
4.17 × 5.00 × 33.70	4839	32.3	Straight grained, but heavy knot near end and very heavy knot near centre. Crippled at latter knot.
4.30 × 5.01 × 32.72	5566	36.7	Straight grained, but heavy knot on side near centre; also heavy knot 8 ins. from end one side. Failed at the latter knot.
3.95 × 4.33 × 32.28	4479	30.1	A great many knots on each end and at various other points. Failed at a large knot 12 ins. from an end. Also heavy season cracks.
3.98 × 4.10 × 28.65	5735	34.3	One old side badly seasoned and injured by usage; also knots near each end; also a small pin knot near centre at which piece failed by crippling and bursting of fibres.

3.93 × 4.30 × 31.95	5124	32.6	Heavy knots near centre. Crippled at knots.
4.11 × 4.92 × 31.85	7309	35.1	Clear and straight grained, except slight wave 1 ft. from end due to vicinity of knot. Failed at this point by direct crippling.
4.22 × 4.92 × 30.84	7167	39.2	Clear and straight grained. Crippled 8 ins. from end.
2.33 × 2.84 × 28.00	6496	31.7	Clear and straight grained. Failed by bending 10 ins. from end.
2.27 × 2.27 × 33.75	5708	36.0	Clear and straight grained. Failed by bending; short specimen failed at 30,000 lbs.
3.96 × 4.18 × 35.25	5015	36.6	Several knots; crippled at one running from corner to corner 12 ins. from one end.
4.20 × 4.50 × 38.00	5905	35.6	Grain out of parallel; clear. Failed by bursting and shearing along season cracks.
3.33 × 3.40 × 33.55	7615	33.6	Clear, straight grain. Crippled near one end.
3.30 × 3.38 × 33.54	7444	35.6	Clear and straight grained. Failed by crippling 6 ins. from end.
3.35 × 3.40 × 33.50	5338	35.4	Large knot passing through centre side to side; piece split end to end through this knot.
3.30 × 3.40 × 33.55	5909	35.6	Knot near centre, also two small pin knots near end. Crippled through pin knots.
3.30 × 4.00 × 33.50	5416	35.2	Large knot near centre passing from side to side. Split from end to end through knot.
3.30 × 4.00 × 33.50	5023	32.8	Large mass of knots near middle. Crippled at these.
4.25 × 5.75 × 35	5729		
4.25 × 5.87 × 41.75	4090		
4 × 4 × 48	4469	32.75	Grain parallel; knot at centre at corner; other knots near end; centre of tree 12 ins. away. Bent at centre at knots concave to a high corner.

2.86 × 4.06 × 40.02	6330	38.1	Straight grain; small knot 14 ins. from end. Failed by bending in middle.
4.10 × 4.24 × 41.83	3866	36.3	Straight grain; three knots. Crippled at knot 12 ins. from end; no bending.
4.25 × 4.25 × 54.95	3389	34.6	Straight grain; many knots. Burst in two opposite directions at knots 11 ins. from one end and 12 ins. from other end.
1.99 × 2.64 × 52.62	5105	34.3	Straight grain; clear; bent at centre.
4.26 × 4.33 × 60.0	3980	35.5	Straight grain; failed by crippling at knot passing through corner 13 ins. from end and 1-16 in. out of square; no appreciable effect.
4.09 × 4.34 × 59.0	3211	34.4	Straight grain; three or four knots; season crack on one side. Crippled at knot 20 ins. from end and season crack opening.
4.18 × 4.22 × 59.75	3190	35.4	Four knots, two each 18 ins. from ends, several other small knots; grain not straight; large season crack. Failed by shearing and bursting open at season crack across annular rings.
2.46 × 2.51 × 60.5	4619	34.5	Straight grain; several knots. Failed by crippling at knot 12 ins. from end.

RESULTS OF COMPRESSION TESTS ON

OLD DOUGLAS FIR.

Dimension in ins.	Lengths.	Breaking load in lbs. per sq in.	Weight in lbs. per cub. ft.	Remarks.
2.21 × 2.23 ×	9.15	8644	35.9	Grain straight and clear; one old side with season crack. Bulged along season crack, and crippled.
3.45 × 2.78 ×	9.65	6465	32.5	All fresh sides; straight and parallel grain; one edge strained from bolt. Crippled all over.
3.41 × 2.78 ×	9.65	7247	35.4	One old side; grain straight and parallel. Crippled near one end.
3.41 × 2.80 ×	9.70	5696	33.2	All fresh sides; grain straight and parallel; one edge strained from bolt; 1 in. season crack. Crippled one-fourth the way down, slightly helped by season crack.
3.38 × 2.78 ×	9.65	6979	34.5	One old side; grain straight and parallel. Crippled at one end, slightly aided by season crack.
2.76 × 3.76 ×	9.64	7235	35.6	One old side; iron stain at one end; season crack; grain straight and parallel. Crippled at 3 ins. from end.
2.83 × 3.81 ×	9.75	6577	32.9	One old side; grain straight and parallel. Crippled near centre.
4.15 × 4.64 ×	11.32	6660	35.70	Knot 5 ins. from end; next face, knots 1½ ins. and 4 ins. from same end: small pin knot and season crack on third side. Crippled through knots.
4.35 × 4.67 ×	11.95	7900	47.25	Clear and straight; very full of resin; some season cracks; crippled at one end.

3.40 × 3.47 × 12.00	5085	31.7	Grain straight, but slightly curly; three fresh sides; old side crushed by tie; slightly rotten under tie; crippled at small defect near one end.
3.45 × 3.45 × 12.00	5218	30.88	Grain parallel; crushed and rotten for a depth of ½ in. under tie; two adjacent sides new. Crippled at rotten part near one end.
3.45 × 3.47 × 12.0	3838	35.0	Grain parallel, but crooked; knot near corner 4½ ins. from end, 1½ ins. diar., knot extended into piece. Crippled through knot.
3.45 × 3.47 × 12.0	4928	38.7	Grain parallel; three fresh sides; 1¾ ins. knot passing through corner 5 ins. from end. Crippled near one end and split along grain adjacent to knot.
3.45 × 3.45 × 12.0	5461	33.3	Grain parallel; two adjacent fresh sides; season crack on one old side. Crippled near one end and split slightly along season crack.
2.90 × 2.92 × 12.0	5314	34.0	Grain parallel; three fresh sides; small season crack. Crippled near one end.
3.41 × 3.48 × 12.0	5308	34.9	Grain parallel; three fresh sides; knot hole on one corner 3¼ ins. long, 0.8 in. deep; also season cracks. Failed by opening of season cracks.
3.42 × 3.47 × 12.0	4011	30.0	Grain parallel; three fresh sides; old side slightly damaged; also cant hook holes. Crippled near centre at cant hook holes.
3.42 × 3.45 × 12.0	4814	32.0	Grain parallel; two fresh sides; slightly rotten at one end on old side. Crippled at the rotten point.
3.45 × 3.46 × 12.0	5053	30.5	Straight grain; all fresh sides; shows signs of failure; crack at end. Crippled near one end.

2.88 × 2.87 × 12.0	6199	33.2	Grain sound and parallel; three fresh sides. Crippled near one end.
3.44 × 3.46 × 12.0	5703	33.6	Grain parallel; two adjacent fresh sides; season cracks; small cant hook hole 2 ins. from end close to corner; slightly rotten. Crippled at cant hook mark.
3.46 × 3.46 × 12.0	5693	33.8	Grain parallel; three fresh sides; small season crack on one side. Crippled at one end; season crack opened.
2.82 × 3.40 × 12.05	6611	32.7	Parallel grain; four fresh sides. Crippled near one end.
2.77 × 3.36 × 12.0	7519	35.3	Parallel grain; one old side; saw cut and season crack. Crippled near one end.
2.80 × 3.40 × 12.03	6813	32.5	All fresh sides; grain straight and parallel; 1 in. season crack. Split along season crack.
2.79 × 3.35 × 12.03	6845	34.6	One old side; season cracks; grain straight and parallel. Split along season crack.
2.79 × 3.91 × 12.03	7149	34.6	One old side; grain straight and parallel. Crippled at one end.
2.78 × 3.73 × 12.04	7348	35.5	One old side; grain straight and parallel; season cracks 1 in. deep. Crippled at one end.
2.77 × 3.86 × 12.05	7390	33.5	One old side; grain straight and parallel. Crippled near centre at a small defect.
2.80 × 3.80 × 12.06	7481	34.1	One old side; grain straight and parallel. Crippled at end.
2.78 × 3.88 × 12.0	7090	34.2	One old side; grain straight and parallel. Crippled near one end.
2.79 × 3.06 × 12.0	7317	33.4	One old side; grain straight and parallel. Crippled at 3 ins. from end.
3.27 × 3.95 × 12.0	5540	33.45	Grain straight and clear, except small pin knot hole 3 ins. from end; piece shivered by season cracks. Failed by piece splitting off. It then crippled at knot 3 ins. from one end.

3.28 × 3.96 × 12.	5510	32.9	Grain straight; small pin knot on a corner near centre; very heavy season crack on old side. Burst along season crack; also crippled 4 ins. from one end.
3.32 × 4.04 × 12.0	4825	28.85	Grain straight; pin knot on corner near centre; heart decayed; also one season crack. Crippled at pin knot.
3.31 × 4.02 × 12.04	5675	32.85	Grain straight; small pin knot 1½ ins. from end; two bad season cracks. Crippled square across near each end.
3.33 × 4.0 × 12.0	4165	28.95	Grain not quite straight; knot at corner 2 ins. from end; deficiency of heart all along one edge. Crippled at knot.
3.30 × 4.0 × 12.0	6300	33.55	Straight grain; knot on corner 1½ ins. from end; large deficiency on opposite corner at other end; another deficiency and nail gouge at centre of same edge; also one season crack. Crippled at knots.
3.28 × 4.02 × 12.03	5540	32.70	Straight grain; knot on corner 1½ ins. from end; also season cracks. Crippled 4 ins. from end.
4.18 × 4.63 × 12.22	5200	35.3	Knots 3 ins. and 6 ins. from end on same side; also small knot on next face 1 in. from same end; also part of large knot on other end. Failed longitudinally through two knots; upper end was not horizontal, not more than 5-6 ths of the area bearing.
4.35 × 4.65 × 14.15	6735	36.95	Two knots 2 ins. and 6 ins. from end on same side; also knot on next face 3 ins. from same end and two knots on other end; on third and fourth faces, knots 1½ ins. and 4 ins. from first end. Crippled at knot 3 ins. from end:

4.25 × 4.65 × 14.80	7085	36.6	Two knots passing through from face to next face; one 3 ins. from end; the other 7 ins. from same end; deficiency 1 in. × 1¼ in. on opposite edge. Crippled through knot 7 ins. from end.
4.39 × 4.70 × 14.78	6500	45.70	Full of resin; part of large knot on one end; season crack on one face; shaken on a corner. Crippled in solid wood (in resin part) 4 ins. from end.
4.14 × 4.65 × 14.80	6730	41.0	Patch of resin through centre; knot on one corner 6 ins. from end; slight season cracks; slight deficiency on one corner. Crippled through knot.
4.25 × 4.66 × 14.78	6020	37.4	One medium knot 1 in. from end; also many small knots on same face; on next face, knots at 6 ins. and 1 in. from same end. Failed through knots at the centre.
4.16 × 4.60 × 14.50	7410	35.7	Part of large knot on one end; one side covered with small knots; otherwise sound specimen. Failed at large knot at end.
4.28 × 4.70 × 14.78	7490	36.2	Grain parallel; one medium knot 5 ins. from end; also two small knots 1 in. from same end and on same side; also heart shake. Failed at centre by crippling through small knot.
4.17 × 4.70 × 14.78	6400	34.0	Grain parallel; mass of knots at one end; also badly seasoned in resinous portion. Crippled at knotty end.
4.35 × 4.74 × 14.80	6310	47.0	Grain parallel; large knot near one end; bad season cracks in resinous portion. Crippled at large knot.

4.27 × 4.67 × 14.80	7310	37.2	Grain straight and sound, but one large knot on end; also one knot on an edge 3 ins. from end; one knot 5 ins. from other end on same edge; slight season cracks. Failed at the two last knots.
4.14 × 4.57 × 14.75	6960	35.45	Knots in each end; otherwise clear; two old sides badly shaken. Crippled and burst at knot at one end.
4.32 × 4.70 × 14.80	5970	38.05	Groups of small knots about 3 ins. from each end; also full of resin. Crippled at each end through knots.
4.14 × 4.60 × 14.80	6580	35.05	Groups of small knots about 4 ins. from each end; also bad season cracks. Crippled through one group of knots.
4.06 × 4.65 × 14.85	6500	43.70	Large knot at one end; two knots 5 ins. from other end; full of resin; dense and heavy; one season crack. Crippled through both knots 5 ins. from end.

RESULTS OF COMPRESSIVE TESTS ON

RED PINE.

Dimensions in inches.	Lengths in inches	Comprs've Strength in lbs. per sq. inch.	Weight in lbs. per cub. ft.	Remarks.
4.96 in dia.	× 5.9	2497		Failed at knots 26 ins from end; also at another ring of knots 3 ins. from same end; nineteen knots in length.
4.97 in dia.	× 5.8	2742		
2.98 "	× 5.86	2722		
3.00 "	× 5.9	2631		
2.95 in dia.	× 5.65	6870		One knot near one end Failed by crippling above knot.
2.88 in dia.	× 5.69	7057		Clear. Crippled 6 ins from one end.

4.81 in diam. × 13.75	5092			Clear grain. Failed by spreading at bottom.
3.88 " × 13.5	7602	39.9		Nearly straight grain; knot 6 ins. from end passing nearly through centre. Failed at the knot by crippling.
3.80 " × 13.31	6438	35.8		Straight grained; knot on one end. Failed by crippling at knot about ½ in. from end all around
4.02 " × 18.75	4657			Clear wood; straight grained; spread at end, due to curvature of fibre in locality of a knot.
3.90 " × 18.20	7222	35.7		Clear and straight grained. Failed 6 ins. from end by folding.
3.66 " × 22.61	8516	43.2		Grain parallel; one knot 10 ins. from end. Failed through knot by crippling.
4.01 " × 22.73	5637	28.7		Four knots at 8 ins. from one end. Failed by crippling at knots.
4.3 " × 22.8	5983	26.7		
3.93 " × 29.2	7914	38.1		Grain parallel; two knots, one large knot 10 ins. from one end. Failed by crippling at this knot.
6.93 " × 36.12	2698			Failed by crushing at knot, 4 ins. from end. Fourteen knots in length.
7.02 " × 36.12	2087			Failed at knot 8½ ins. from end; ten knots in length.
7.01 " × 36.12	2024			Failed at ring of knots 7 ins. from end; fifteen knots in length.
3.97 " × 3.10	3287			Crushed and failed at knot; straight grain; fairly free from knots.
4.10 " × 3.10	2825			Failed by crushing and bending. Straight grain; crack down length.
4.04 " × 3.10	3482			
4.03 " × 3.10	4247			
3.98 " × 3.10	3223			
3.96 " × 3.10	4001			
4.75 × 4.75 × 60.	3104			

3.97 in diam. × 69.	2585	.985	Not well seasoned. Failed by crushing and bending at a large knot 31 ins. from end; also at 1 in. from end and 4½ ins. from other end; straight grained; six knots in whole length.
4.08 " × 69.	2593		Failed at ring of knots four in number by crushing and bending at 24 ins. from end; also at 2 ins. from same end; fourteen knots in whole length.
4.02 " × 69	3152		Failed by crushing; straight grained; failed at two small knots 27 ins. from end and also at 16 ins. from same end; large knots 39 ins. from same end; ten knots in length.
3.91 " × 69	3280		Failed by crushing 16 ins. from one end at a knot. Twelve knots in whole length.
4.03 " × 69	3158		Failed chiefly by crushing 12 ins. from one end; four knots in length.
3.96 " × 69	3734		Failed at knot 24 ins. from end; six knots in length; also crippled 1 inch from same end.
4.94 " × 66.25	2386		Failed at knots 26 ins. from end; also at another ring of knots 3 ins. from same end; nineteen knots in length.
4.92 " × 66.25	2513		Failed at ring of knots 36 ins. from end; sixteen knots in length.
2.96 " × 66	1977		Failed by crushing and bending at large knot 28 ins. from end. eight knots in length.
3.06 " × 66.25	2433		Failed by crushing at knots 5 ins. from end. Four knots in whole length.

RESULTS OF COMPRESSIVE TESTS ON

NEW WHITE PINE.

Dimensions in inches	Lengths in inches.	Comprs've Strength in lbs. per sq. inch.	Weight in lbs. per cub. ft.	Remarks.
4.187 × 2.44 ×	2.31	3810		
4.687 × 2.312 ×	2.44	2955		
4.812 × 2.312 ×	2.44	4248		
3.0 × 2.94 ×	2.98	5352	24.4	
4.75 × 4.75 ×	3.	3821		
4.8 × 4.8 ×	4.6	3515		
4.75 × 4.75 ×	4.6	4387		
4.75 × 4.80 ×	4.53	3280		
4.75 × 4.44 ×	4.50	3449		
4.75 × 4.78 ×	4.36	4361		
4.75 × 4.75 ×	4.37	4433		
4.75 × 4.75 ×	4.40	4363		
4.75 × 4.70 ×	4.50	3449		
4.75 × 4.80 ×	4.53	3193		
4.75 × 4.75 ×	4.37	3972		
4.75 × 4.75 ×	5.	3548		
4.75 × 4.75 ×	10.375	2826	30.3	Grain clear but not straight. Cracked down one side.
3.01 in diam. ×	11.35	4382	26.7	Clear and straight. Failed by folding near one end.
4.75 " ×	11.125	3500	21.60	Clear grained, but not straight. Failed by folding over at top.
4.75 " ×	11.875	5527	27.50	Clear specimen; deep season cracks across annual rings. Failed by crippling.
4.812 " ×	12.25	3990	23.80	Two large knots. Failed between them.
3.00 " ×	12.80	3762	29.4	Two heavy knots 2 ins. from end. Failed by crippling at the knots
4.75 × 4.75 ×	12.156	5383	26.5	Clear specimen. Crippled without bulging or cracking.
2.98 × 2.98 ×	12.0	5574	29.4	Clear and straight grained. Failed by crippling.
4.74 in diam. ×	13.12	2774		Ring of four knots 6 ins. from one end. Failed by crippling at knots.

4.71 in diam. × 14.562	3400	20.6	One knot and also signs of decay. Failed by crippling at the knot.
2.625 × 3.562 × 14.125	6400		Clear.
4.72 × 4.72 × 14.875	5004	26.3	Clear. Crippled without cracking or bulging.
4.75 in diam × 14.75	4408		
4.71 " × 15.5	3360	21.1	One large knot; decayed near heart Failed at knot.
4.703 " × 15.35	3861	26.60	One knot at bottom of specimen. Failed at this knot by crippling.
2.94 " × 15.30	4272	26.5	Clear and straight, but deep injury from pike pole. Failed at injured part.
4.75 " × 16.	4463		
3.87 " × 16.25	2973	29.9	Straight grained. Failed at one end at a large knot.
4.75 " × 17.35	4232	26.40	Two large knots. Failed between them.
4.71 " × 17.938	4847	27.1	Clear and straight grained. Failed at end.
4.40 × 4.40 × 17.0	3856	30.6	Three large knots in a ring around specimen. Failed at knots.
2.97 × 3.85 × 20.54	6036	30.1	Clear and straight grained; one-third sapwood. Failed by crippling at 7 ins. from one end.
3.85 × 3.83 × 21.65	3933	26.1	Failed previously as pillar under 49,200 lbs. Crippled now at a large knot 8 ins. from end.
3.8 × 3.8 × 22.35	3808	26.7	Two large knots. Crippled at one, 2 ins. from an end.
3.83 × 3.83 × 23.82	3615	25.9	Failed by crippling at two knots near centre.
3.97 × 2.99 × 23.60	5462	24.9	Clear and straight grained; failed previously as pillar under 42,000 lbs. Crippled now near centre.
3.02 diam. × 25.79	5023	24.5	Clear and straight grained. Failed by crippling 8 ins. from one end.
3.40 × 3.80 × 25.4	3610	25.0	Straight grained; bad season cracks; full of knots, failed by crippling through two of them 8 ins. from end.

2.98 × 2.99 × 24.25	4607	23.9	Straight grained; pin knot 10 ins. from one end. Failed by crippling and bending at pin knot.
2.95 × 3.25 × 26.70	3508	24.1	Straight grained, but full of knots. Crippled at one near corner in middle.
4.75 × 4.75 × 21.0	3103		Clear; grain 2 ins. out of parallel; season cracks along grain. At upper corner grain ran out. Failed by sliding along seasoning, due to non support of fibres running from corner.
2.99 × 2.99 × 24.08	4474	26.7	
3.05 in diam. × 24.1	5240	25.8	Clear and straight grained. Failed by crippling and bending at same instant at middle.
3.46 × 4.33 × 27.00	3488	20.4	Failed previously as pillar under 33,300 lbs. Failed now at knot 8 ins. from end on a side.
2.92 in diam. × 36.53	5269	29.8	Clear and straight grained; one-third sapwood. Fail by crippling on sapwood side and then bending afterwards 12 ins. from end.
3.05 " × 48.0	4377	25.9	Clear grain, 1½ in. out of straight; high at one side. Failed by bending 20 ins. from one end on high side.
3. × 3 × 48.0	4666	25.0	Ten knots; long season crack ran three fourths the way down, 1½ ins. deep and ½ in. from edge; a bruise 3 ins. from end on same side; on opposite side, crack 3 ins. long, 1 in. deep; grain and rings both parallel. Failed by bending toward a high corner and then crippling.
4.75 in diam. × 60	2652		
4.75 " × 60	1862		
4.75 × 4.75 × 60	2749		
4.75 × 4.75 × 60	1862		
4.75 × 4.75 × 60	1951		
4.75 × 4.75 × 60	1951		

4.75 × 4.75 × 60	2306		
4.75 in diam. × 61	2676		
4.62 × 4.75 × 60	2370		
4.62 × 4.75 × 60	2826		
4.75 in diam. × 60	2765		
4.00 × 4.00 × 78.24	2937	27.6	Heart; unseasoned; straight grain; four groups of knots 2 ins., $3\frac{1}{2}$ ins., $4\frac{3}{4}$ ins., $5\frac{3}{4}$ ins. from end on each face. Crippled and failed through knot 2 ins. from end on low side.
4.03 × 4.06 × 78.2	3466	28.7	Straight grain; several knots. Failed by bending at knot 30 ins. from one end. Ends square; maximum load 70,500 lbs.
4.03 × 4.03 × 75	4557	28.2	Straight clear grain; one small knot. Failed at knot 3 ft. 4 ins. from end; crippled, then split open; ends square.
3.95 × 3.98 × 75	3260	29.3	Grain straight but for frequent knots; failed at a group of knots about 2 ft. from one end by splitting first slightly open and then crippling on one side; it bent afterwards.

RESULTS OF COMPRESSIVE TESTS ON

OLD WHITE PINE.

Dimensions in inches. Lengths.	Comprs've strength in lbs. per sq. inch.	Weight per cub. ft. in lbs.	Remarks.
3.5 × 4.4 × 11.75	1980	27.35	Large knots on all sides about 2 ins. from an end, otherwise in good condition, except shivered at a corner between two knots. Failed by splintering at shivered corner; also crippled at knots.
3.4 × 4.3 × 11.70	2740	28.10	A large knot appearing on two faces 3 ins. from end; also a slight season crack on one face. Failed by splitting longitudinally along season crack.

3.46 × 4.32 × 11.75	4470	26.45	Medium knot through corner showing on two faces about 1½ ins. from end; otherwise sound and clear. Failure by crippling at centre.
3.50 × 4.25 × 11.74	3850	26.30	Knot on a face 1½ ins. from end, passing to opposite face ½ in. from end; also small deficiency at corner on same end and along one edge; also sapwood. Crippled longitudinally through knot.
3.45 × 4.39 × 11.77	4115	25.35	One small pin knot on corner; also shaken by seasoning; also two small injuries on an edge. Burst at the season cracks; afterwards crippled.
3.50 × 4.41 × 11.75	2735	25.55	Two large knots at an end on opposite faces 2 ins. from end; also slight season cracks. Crippled at knots.
3.47 × 4.38 × 11.75	4330	26.50	Clear and nearly straight grained; slightly shaken by season cracks. Crippled 5 ins. from one end.
3.52 × 4.37 × 11.75	2625	28.55	A large knot 3 ins. from end passing through from opposite faces; also seasoned somewhat. Crippled through at knot.
3.45 × 4.25 × 11.75	4660	23.3	Clear specimen, except deficiency at a corner, partly sapwood; also bad injury (spike hole) in deficient corner. Crippled at centre.
3.45 × 4.36 × 11.70	3975	24.5	Two weathered sides; clear; seasoned. Clear crippled at centre.
3.50 × 4.27 × 11.70	4695	25.0	One old side; clear; shaken by season cracks. Crippled at centre.
3.49 × 4.37 × 11.75	4230	25.8	Grain clear and straight, large cant hook hole 1 in. from one end on old narrow side. Failed by crippling at centre.

3.48 × 4.32 × 11.73	3910	24.4	Large knot on end; seasoned; grain clear and straight. Failed by crippling at centre.
3.48 × 4.40 × 11.74	3830	23.85	Large knot on end; grain clear and straight, season cracks. Failed by splitting longitudinally and crippling slightly at centre.
3.51 × 4.30 × 11.60	4525	25.65	One old side; grain clear and straight; piece badly shaken. Crippled at centre.
4.10 × 4.16 × 12.00	2923	23.2	Grain clear and straight; season cracks on two old sides; injured by cant hook on one old side. Crippled at one end and through defect.
4.21 × 4.19 × 12.00	2183	23.0	Grain parallel; one small pin knot; season cracks on old side; one small defect on corner 2 ins. from end. Crippled at one end.
4.17 × 4.18 × 12.05	2059	25.4	A large knot near centre; badly seasoned on old side; split along seasoning; split from knot. Also crippled.
4.14 × 4.22 × 12.00	2840	22.9	Grain clear and straight, seasoning cracks through centre; small defect on old side. Crippled through defects.
4.19 × 4.20 × 12.00	1716	32.5	A large knot from end to end along one face; another at one end; another at opposite side. Fibre split from knot.
4.18 × 4.22 × 12.00	2228	26.3	A large knot from end to end along one face; another at one end. Crippled at knot at centre, and also a splitting away.
4.14 × 4.18 × 12.00	2794	23.1	Clear and straight; seasoned on two old sides. Crippled at one end.
4.17 × 4.19 × 12.00	1723	25.0	Grain clear and straight, bad season cracks on old side; spike hole $2\frac{1}{2}$ ins deep, 2 ins. from one end. Failed at spike hole.

4.21 × 4.21 × 12.00	2257	22.3	Grain straight; three fresh sides; one large knot near end; season cracks on old side. Crippled through knot at one end.
4.20 × 4.22 × 12.00	2438	23.6	Grain straight; two large knots at opposite ends; season cracks on old side. Crippled on end at a knot.
4.16 × 4.21 × 12.00	2569	23.4	Grain straight and parallel, except at one end, where it is curled by vicinity of a knot; otherwise sound. Crippled at sound end.
4.19 × 4.22 × 12.00	2030	28.0	Two large knots at one end, otherwise straight and clear; fresh sawn on all sides. Crippled at knots at end.
4.13 × 4.20 × 12.00	2686	24.1	Grain straight; three small knots at centre; two old sides injured by several small holes. Fibre split and crippled at small knots.
4.17 × 4.18 × 12.00	2180	25.3	Three large knots at centre; grain parallel; full of season cracks on old side; fibre split. Crippled at knots.
4.20 × 4.21 × 12.00	1883	24.4	Grain crooked by knots; two large knots near centre; large season crack on one old side. Crippled across centre at knots.
4.21 × 4.23 × 12.00	1915	25.0	Four large knots near centre, otherwise clear and straight; one knot at each corner. Crippled across centre at knots.
4.16 × 4.21 × 12.00	2512	23.39	Grain straight; three sides fresh sawn; small pin knot; small defect at one end on old side. Crippled at and near small defect.
4.20 × 4.23 × 12.00	2277	26.1	A large knot hole at an end; three smaller knots near centre; otherwise sound and straight. Crippled at end aided by knot.

4.18 × 4.23 × 12.03	1838	27.2	Two sides fresh sawn; three large knots 2 ins. to 4 ins. from one end; grain twisted; three cant hook marks; cracks in medullary trays. Failed by splitting from large knot.
4.20 × 4.23 × 12.04	2477	25.0	Three sides fresh sawn; grain not parallel, owing to a knot; one season crack on old side; wood decaying somewhat; several small pin knots. Sheared along season crack, caused by adjacent knot.
4.19 × 4.22 × 12.05	2177	26.4	Three fresh sawn sides; two large knots near centre; one pin knot; grain parallel; very large season cracks. Split along season cracks.
4.20 × 4.25 × 12.04	2387	26.1	Four sides fresh sawn; grain parallel; season cracks are through specimen; one large and two small knots at one end, large one at corner. Crippled at knots.
4.17 × 4.20 × 12.02	2752	24.7	Three sides fresh sawn; grain not parallel; season cracks through body of specimen; slightly decayed on one side; several small pin knots. Sheared on rot line and crippled at knots.
4.21 × 4.23 × 12.02	1797	26.7	All sides fresh sawn; two large knots in body; grain parallel; slight decay; cracks in medullary rays. Crippled through knots.
4.18 × 4.20 × 12.05	1789	25.0	Two sides fresh sawn; grain not quite parallel; large knot at one end; season cracks on two old sides; small knot in body. Crippled through knots.

4.19 × 4.22 × 12.05	2099	24.8	Three sides fresh sawn; grain parallel; season cracks on old side; two small injuries in old side near one end. Crippled through very small knot near one end.
4.21 × 4.22 × 12.01	2251	27.3	Three fresh sides; specimen full of knots, two at one end, one large knot and two small knots in body; bad season crack on old side. Crippled through knot at one end.
4.17 × 4.24 × 12.02	1606	28.0	Four fresh sides; two large knots near centre; two pin knots; grain parallel. Crippled and split along fibre from the knots.
4.18 × 4.20 × 12.0	2033	25.4	Three sides fresh sawn; large knot 4 ins. from end; grain parallel; slight decay. Crippled opposite knot.
4.20 × 4.22 × 12.0	2499	25.9	Four sides fresh sawn; large knot near centre; grain parallel. Crippled opposite knot.
3.82 in diam. × 13.65	5770	30.3	Clear and straight grained. Failed by folding through an injury from cant hook 4½ ins. from end.
3.625 × 4.50 × 40.875	2390	22.4	Grain straight; one old side; free from large knots; failed by bursting open along three lines, which pass through various knots and season cracks.
3.75 × 4.31 × 45.25	2970	23.6	Grain straight; one old seasoned side; several knots; failed at one large knot in middle of pillar, which passed through from side to side. Failure by bending across narrow dimension.

3.50 × 4.50 × 45.125	1840	22.6	Grain straight; one old seasoned side; many knots; failed at one large knot in middle of pillar, which passed through from side to side. Failure by bending across narrow dimension.
3.50 × 4.38 × 44.5	2170	21.9	Grain straight; one old side; many small knots; one large knot on old side 15 ins. from one end. Failed by crippling at that knot.
3.73 × 4.35 × 44.5	2650	23.6	Straight grain; fairly clear; some small knots; one old seasoned side. Failed by bending 18 ins. from one end in clear wood across least dimensions.
3.5 × 4.4 × 45	3346	22.8	Grain straight; two old sides; knot at one end; also knot at centre passing through a corner. Failed by direct crippling which started at knot in middle of the piece.
3.5 × 4.4 × 42.5	2082	21.1	Grain nearly straight; one old side; various knots, particularly one near centre passing from corner to corner of section. Failure by bending at this knot on least dimension.
3.5 × 4.45 × 46	2248	21.7	Grain straight; one old side. Failed near centre by bending across least dimension at a knot, which penetrated the heart of piece from one side.
3.83 × 3.83 × 71.3	2862		Two knots on one edge, one large knot at centre, another 12 ins. away; on second face five knots, two near centre, others 12 ins. from ends; grain parallel; centre of tree in corner of specimen, failed by bending at centre knot, induced first by being ½ in. off centre on top bearing.

3.84 × 3.84 × 72.0	3338	26.06	Bad knot 6 ins. from centre on one face; next face knot 2 ins. from end; grain about parallel; many smaller knots; centre of tree on same corner as large knot. Failed by bending at large knot.

RESULTS OF COMPRESSIVE TESTS ON

NEW SPRUCE (B.C.)

Dimensions in inches. Lengths.	Comprs've strength in lbs. per sq. inch.	Weight in lbs. per cub. ft.	Remarks.
4.72 × 2.343 × 1.94	3415		
4.77 × 2.25 × 1.9	2941		
4.75 × 2.375 × 1.875	3020		
4.72 × 2.25 × 1.875	3465		
4.78 × 2.25 × 1.97	3256		
4.75 × 2.25 × 1.94	3118		
4.75 × 2.312 × 1.88	3009		
4.72 × 2.22 × 1.9	3179		
3.75 × 2.34 × 1 62	3854		
4.812 × 2.312 × 1.94	3210		
4.375 × 1.875 × 2	4440		
4.75 × 2.25 × 2.50	3321		
4.73 × 4.73 × 3.9	3451		
3.67 × 3.67 × 3.64	5590		Failed by crippling.
4.75 × 4.75 × 4.0	3325		
4.75 × 4.75 × 4	2838		
4.812 × 4.812 × 4	2986		
4.65 × 4.65 × 5.20	4540		
3.00 × 2.875 × 6.50	7566		Clear and straight.
3.00 × 3.125 × 6.00	6036		
4.7 × 4.7 × 7.75	4299	29.80	Four pin knots; ends not quite parallel.
3.125 × 2.875 × 7.25	6812		
4.687 × 4.687 × 8.66	5305	29.80	Clear and sound; cracks along medullary rays.
4.75 × 4.75 × 11.5	4656		Clear and straight.
4.2 × 3.8 × 11.5	4806	25.9	Crippled at centre.
4.0 × 4.04 × 11.75	3898	33.8	Straight grained. Crippled at large knot on edge near centre.

4.10 × 4.10 × 12.55	4451	28.3	Clear and straight grained; slight axe-cut on one face 3 ins. from end. Failed by crippling at axe-cut.
3.75 × 3.75 × 12.05	4907	29.5	Crippled at a bunch of five knots.
4.72 × 4.72 × 14.09	4063	30.2	
4.75 in diar. × 14.	3328		Five large knots and one large season crack.
3.33 × 4.18 × 14.97	4382	33.9	Clear and straight. Failed by crippling near one end.
4.35 × 4.32 × 20.55	3757	29.6	Failed by crippling.
4.35 × 4.45 × 20.6	3540	27.1	Knot near one end. Failed in centre.
4.41 × 4.45 × 20.6	3850	29.9	Clear.
2.5 × 3.42 × 27.5	3390	26.3	Clear and straight grained, but heavy season crack from side to side. Failed by bulging on season crack and then bending.
3.48 × 3.50 × 32.25	4384		Grain not straight; heavy knot through centre; also ends not square. Burst apart along centre.
2.75 × 4.05 × 41.0	3070	28.3	Straight grained. Failed at large knot 3 ins. from end by crippling.
2.75 × 4.02 × 40.95	3086	28.4	Straight grained; eight large knots. Failed by bending at two knots 19 ins. from one end concave to high side.
4.35 × 4.50 × 20.55	3584	27.4	Grain clear and parallel. Crippled at centre.
4.08 × 4.35 × 22.97	3909	27.5	Grain crinkled near one end. Failed there.
4.18 × 4.35 × 22.95	3271	27.7	Clear; straight; no knots. Failed at one end.
4.29 × 4.35 × 22.96	3617	25.4	Grain not quite parallel; knot near centre of one side at which piece failed.
4.20 × 4.35 × 22.95	2834	28.2	Grain not parallel. Failed by longitudinal shear, which passed through a knot.
4.25 × 4.40 × 22.9	3774	26.1	Failed at a knot near centre of one side.
4.24 × 4.34 × 22.94	2973	25.1	Failed by longitudinal shear.
4.12 × 4.35 × 23.00	3560	27.2	Failed at a knot.

4.10 × 4.41 × 23.60	3680	25.7	Grain parallel. Failed by crippling at a knot 6 ins. from one end.
4.25 × 4.40 × 23.0	3382	27.9	One season crack, did not affect the failure which was by crippling.
4.10 × 4.40 × 23.05	3550	26.4	Knot near one end. Crippled in body of piece at a distance from the knot.
4.09 × 4.35 × 23.06	4229	25.6	Grain clear and parallel. Crippled on one side.
2.97 × 4.0 × 15.1	4908	26.7	Clear and straight grained. Crippled two inches from end.
3.33 × 4.1 × 15.64	3370	26.4	Straight grained; large knot on middle of side. Failed near one end in clear wood.
4.72 in diar. × 15.0	3430	30.86	Four deep medullary weathering cracks; a mass of knots at lower end; small pin knots at centre; ends not quite parallel. Crippled at lower end at knots.
2.6 × 4.1 × 18.5	5253	24.1	Clear and straight grained; failed by crippling and bending 6 ins. from one end.
4.75 in diar. × 60	1862		
4.75 " × 60	2708		
4.75 × 4.75 × 60	2351		
4.75 × 4.75 × 60	2275		
4.75 × 4 75 × 60	3104		
4.75 × 4.75 × 60	2660		
4.75 × 4.75 × 60	2351		
4.75 × 4.75 × 60	2306		
4.75 × 4.75 × 60	2661		
4.62 × 4.63 × 60	2431		
4.62 × 4 75 × 60	2416		
4.62 × 4.62 × 60	2420		
4.75 in diam. × 60	2483		
4.75 " × 61	2483		
4.75 " × 61	3215		

RESULTS OF COMPRESSIVE TESTS ON

OLD SPRUCE.

Dimensions in inches.					Lengths.	Comprs've strength in lbs. per sq. inch.	Weight in lbs. per cub. ft.	Remarks.
2.54	×	3.15	×		5.95	4375	28.4	Clear wood, straight grained; ends out of square; bent over.
2.12	×	2.97	×		10.12	4508	28.4	Clear wood, straight grained; ends out of square; bent over.
2.42	×	2.45	×		10.95	4367	27.9	Clear wood, straight grained; failed by bending; worm eaten.
2.50	×	3.20	×		11.25	3862	28.4	Clear wood, straight grained ends out of square; bent over.
2.18	×	2.18	×		14.00	4842	27.9	Clear wood, straight grained; failed by bending; worm eaten.
2.17	×	2.18	×		13.40	4714	27.9	Clear wood, straight grained; failed by bending; worm eaten.
3.20	×	3.22	×		13.40	5825		Clear; straight grained; crippled at centre.
3.20	×	3.21	×		13.28	5696		Clear; straight grained; crippled at end at a previous injury on surface.
3.17	×	3.21	×		13.62	4900		Straight grained; knot at centre. Crippled at knot.
3.20	×	3.20	×		13.43	5273		Straight grained; knot on corner at centre. Failed at knot.
2.80	×	3.35	×		13.30	5139		Heavy knot through edge near centre. Crippled at knot.
2.80	×	3.34	×		12.50	4818		Straight grained; knots near each end. Crippled and burst through large knot.
2.18	×	2.18	×		16.00	4337	27.9	Clear wood; straight grained. Failed by bending; worm eaten.
3.53	×	3.56	×		14.60	6329		Clear and straight grained. Crippled near end through a small injury like a nail hole.

2.60 ×	2.63 ×	15.45	7339	Clear; straight grained Crippled 5 ins. from end.
2.60 ×	2.75 ×	16.25	3664	One small knot, but badly out of parallel. Failed at knot.
2.66 ×	2.[illegible]5 ×	15.57	6809	Straight grained; one small knot near end. Crippled first near centre through cant hook holes.
2.80 ×	3.37 ×	27.05	5116	Straight grain; knot 12 ins. from end. Crippled at knot.
2.80 ×	3.35 ×	[illegible]6.26	5096	Straight grain; knot 10 ins. from end. Crippled at a knot.
2.62 ×	2.75 ×	17.72	5625	Clear, but grain very much out of parallel, as much as 3 ins. in 18 ins. Burst apart by shearing of unsupported fibre.

TENSILE STRENGTH.

The experiments were especially directed to the comparison of the tensile strength and stiffness of portions of the same stick, in different positions relatively to the heart.

In designing the form of the test-piece, it was of importance to make the head of such a depth as would prevent the central portions from being pulled through the head by shearing along the surface BC, and it was also necessary that the depth should not be inconveniently great. Wedge shaped holders (Fig. H) were adopted which would grip the specimen along the faces AB. This form of holder was intended to increase the resistance to shear which is always much less than the tensile strength. As the tension on the test-piece increases, so also does the normal pressure upon the faces AB, Fig. K, and, therefore, so also does the resistance to shear along the surface BC. At first, the faces of the holders in contact with the specimen were left rough, but it was found that the roughness prevented the specimen from sliding in far enough to be gripped along the whole of the face AB, so that the bearing surface was practically limited to a comparatively small area near the top of the head. Thus it often happened that the specimen still failed by shearing along the surface BC. This difficulty was obviated by planing the faces of the holders.

The test-pieces were prepared from the uninjured portions of the

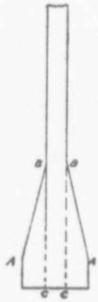


Fig. K.

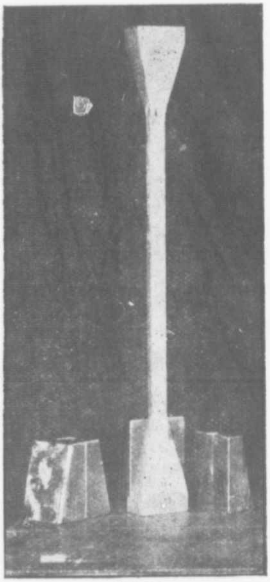


Fig. H.

beams, which had already been fractured transversely. The extensions of a length of ten inches of the specimen under gradually increased loads were measured by means of Unwin's extensometer until the total extension exceeded about one-eightieth of an inch After this the extensometer was removed, and in many cases additional extension readings, up to the point of fracture, of a length of sixteen inches of the specimen, were measured by means of a steel rule and indicator clamped to the specimen at points 16 inches apart and allowed to slide over one another.

The results obtained are given in the following tables, and an examination of these will show :—

1st. That the increments of extension up to the point of fracture are almost directly proportional to the increments of load ;

2nd. That the presence of knots is most detrimental both to the strength and to the stiffness, inasmuch as they practically diminish the effective sectional area, and also produce a curvature in the grain ;

3rd. That wood near the heart possesses much less strength and much less stiffness than that more distant from the heart ;

4th. That the strength and stiffness are also dependent upon the proportion of summer to spring growth ;

5th. That irregularity of readings, both with the extensometer and with the rule, are chiefly due to the presence of a knot, or to curly or oblique grain caused by a knot.

Again, some of the tables give the effects on various specimens, of alternately loading them and relieving them from their load, and from the experiments carried out up to date the following inferences may perhaps be drawn :—

If the specimen is clear, free from knots, and straight in the grain, and if no interval of rest is allowed, then for any given range of loads :

(*a*) The total extension is greatest during the first loading ;

(*b*) The extensions due to the successive loadings continually diminish, tending to a minimum limit, so that the co-efficients of elasticity increase, and therefore so also does the stiffness ;

(*c*) By the successive unloadings a set is produced, which continually increases, but at a diminishing rate, and which tends to a maximum limit ;

(*d*) When the specimen is allowed an interval of rest under the minimum load, the first total extension, when the loading is resumed, is greater than at the commencement, but continually diminishes, tending to a minimum limit, which possibly coincides with the maximum limit reached previous to the interval of rest.

So also, after the interval of rest, when the first set produced the specimen is from load, is greater than that previously produced, but gradually diminishes, in the succeeding releases from load, tending probably to a minimum limit coinciding with the maximum limit reached before the interval of rest.

These inferences are also in accord with similar experiments carried out by Mr. Kerry, B.A.Sc.

Special attention may be directed to the test of specimen 4, beam XXI. This specimen failed simultaneously at two sections, the wood seeming to be very brittle, and the character of the failure pointed to some inherent weakness in the timber itself. After a microscopic examination of the fractured sections, Professor Penhallow described the fractures as being " very regular and devoid of any fibrous character, having the " exact appearance of a piece of glass. The lines of fracture followed " the variations in thickness of structure longitudinally and trans- " versely with great regularity. The peculiar brittleness can only be " referred to some local molecular condition of unknown origin, possibly " to a deficiency in the element of water."

The simultaneous failure at two sections of specimens 2 and 8 from White Pine beam XLVIII may probably be referred to a similar cause, and, as Professor Penhallow says, adequate explanations of such failures are still to be sought.

In the tables the extensometer measurements are given in hundred-thousandths of an inch, and the rule measurements in hundredths of an inch.

With each table a diagrammatic section is also given, showing the part of the stick from which the several specimens have been taken.

DIAGRAMMATIC SECTIONS FOR TENSION SPECIMENS.

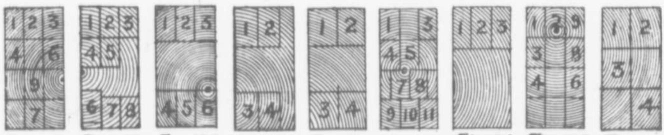


Fig. 118 Fig 119. Fig 120. Fig. 121. Fig 122. Fig 123 Fig. 124 Fig. 125 Fig 126

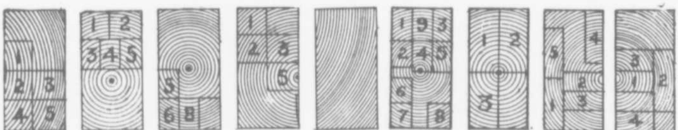


Fig 127 Fig 128. Fig 129. Fig. 130 Fig. 131. Fig. 1 A Fig. 1 B Fig. 132 Fig 133

Results of tension tests on specimens 1 to 9 cut out of Douglas Fir Beam IX, and of repeatedly loading a specimen cut out of the same Beam. (Fig 118.)

Loads in lbs.	Readings taken by Extensometer.								
	Specimen.								
	1 For-ward.	2 For-ward.	3 For-ward.	4 For-ward.	For-ward.	6 For-ward.	For-ward.	7 For-ward.	9 For-ward.
100	0	0	0	0	0	....	......	0	0
200	81	79	65	92	80	......	......	50	82
400	229	227	194	261	240	259	259	162	252
600	372	379	318	430	393	......	......	293	421
800	509	527	435	579	549	564	561	403	579
1,000	644	673	547	737	702	......	.....	520	736
1,200	779	818	664	870	852	863	868	637	890
1,400	914	960	784	1060	1004	1004	1025	752	1047
1,600	1049	1097	894	1226	......	......	1183	869	1200
1,800	1185	1241	1008	1395	......	....	......	984	......
2,000	1323	......	1124	......	......	.. ..	......	1098	......
Total breaking weight in lbs.	9270	6290	10,580	8820	6390	......	......	10,114	6348
Break'g weight in lbs. per sq. in..........									
Coefficient of elasticity in lbs..........									

Results of tension tests on specimens 1 to 7 cut out of Douglas Fir Beam X.

Loads in lbs.	1 Extr.	Rule	3 Extr.	Rule	5 Extr.	Rule	4 Extr.	Rule	4 Extr.	Rule	4 Extr.	Rule	7 Extr.	Rule.	6 Extr.	Rule.
	Readings taken by															
100	0	....	0	....	0	....	0	....	0	....	0	....	0	....	0	....
200	89	....	62	....	53	....	68	....	86	....	99	....	98	....	65	....
400	221	....	157	....	136	....	198	....	260	....	281	....	258	....	208	....
600	333	...	277	....	251	....	319	....	432	....	461	...	418	....	347	....
800	481	...	397	....	367	....	463	....	613	....	633	....	578	....	495	....
1000	614	....	515	....	484	....	679	....	788	....	807	....	738	....	620	....
1200	743	....	636	....	592	....	879	....	943	....	970	....	900	....	755	....
1400	978	....	755	....	712	....	1,049	....	1,135	....	1,132	0	1,057	....	890	....
1600	1,105	....	877	....	819	....	1,175	...	1,303	....	....	....	1,270	....	1025	....
1800	1,258	0	988	....	953	....	1,573	0	1,50	0	....	....	1,429	0	1159	....
2000	....	3	1,121	0	1,058	0	....	2	test-piece cut from neighbourhood of heart	2	....	10	....	5	1297	0
2500	....	12	....	5	....	6	....	8		10	....	17	....	12	....	6
3000	....	19	....	10	....	10	....	13		16	....	23	....	20	....	11
3500	....	29	....	15	....	16	....	19		24	....	30	....	27	....	17
4000	....	37	....	20	....	20	....	23		32	....	37	....	34	....	23
4500	....	47	....	25	....	25	....	29		40	....	45	....	41	....	29
5000	....	55	....	30	....	30	....	36			....	52	....	48	....	35
5500	....	62	....	35	....	35	....	44			....	60	....	55	....	41
6000	....	69	....	40	....	40	....	52			....	73	....	62	....	47
6500	....	79	....	45	....	48	....	67			....	83	....	70	....	52
7000	....	89	....	50	....	60	....	71					....	79	....	60
7500	....	94	....	55	....	68	....	76					....	89	....	66
8000	....	100	....	60	....	73	....	83							....	71
8500			....	65	....	75	....	90								
9000			....	70												
Total breaking weight in lbs.	8,500		9,500		9,240		10,850		4,820		6,960		7,965		8,500	
Brk'g weight in lbs. per sq. in.	11,902		13,532		13,368		15,116		6,914		9,864		11,326		11,623	
Co-effic'nt of elasticity in lbs.	1,830,500		2,364,300		2,510,650		1,710,550		1,650,400		1,665,400		1,686,150		2,009,000	
Time of test in minutes.	18		25		30½		17		18		12		26			

Results of repeatedly loading tension specimens 2 and 5 cut out of Douglas Fir Beam X. (Fig. 119.)

Loads in lbs.	2																5									
	Readings taken by																									
	Extensometer.															Rule	Extensometer.									Rule
	Forward	Return.	Forward	Return.	Forward	Return.	Forward	Return.	Forward	Return.	Forward	Return.	Forward	Return.	Forward	Forward	Forward	Return.	Forward	Return.	Forward	Return.	Forward	Return.	Forward	Forward
100	0	26	26	14	14	3	3	1	1	5	5	8	8	8	8	....	0	29	29	41	41	63	63	63	63	....
200	58	....	48	53	49	62	56	70	62	73	67	78	69	78	67	....	51	91	[illegible]5	103	98	117	109	125	119	....
400	176	178	169	190	189	....	....	....	19[illegible]	....	....	....	....	....	....	....	167	215	213	232	222	247	238	253	238	....
600	294	....	....	316	....	....	....	....	..	....	....	....	....	....	....	....	283	....	....	....	....	....	....	....	....	....
800	418	427	423	440	429	448	439	461	445	461	450	467	452	470	450	....	403	456	447	480	458	489	480	500	472	....
1000	540	....	....	....	....	....	....	....	....	....	....	....	....	....	..	....	526	....	....	....	....	....	....	....	....	....
1200	675	683	680	701	686	704	695	713	701	720	706	721	706	723	713	....	652	693	690	721	700	732	711	743	717	....
1400	791	....	....	....	....	....	....	....	....	....	....	....	....	....	....	....	775	....	....	....	....	....	....	....	....	....
1600	919	930	...	....	...	....	..	....	....	....	....	....	....	....	....	....	900	927	933	955	943	966	954	976	961	....
1800	1049	1049	1068	1068	1073	1073	1080	1080	1087	1087	1087	1087	1091	1091	1098	....	1020	....	....	...	....	....	....	....	......	......
2000															1253	0	1150	1150	1176	1176	1184	1184	1199	1199	12[illegible]3	0
2500																6										6
3000																12										12
3500																18										18
4000																23										23
4500																29										29
5000																35										35
5500																40										40
6000																46										45
6500																51										50
7000																										60
7500																										70
Total breaking weight in lbs.	7,000																7,500									
Break'g weight in lbs p. sq.in.	10,145																10,757									
Co-efficient of elasti'ty in lbs.	2,321,600																2,334,850									
Time of test, in minutes.....	49																45									

Results of tension tests on specimens 1, 2, 5 cut out of Beam X, and of repeatedly loading specimens 3, 4, 6 cut out of same Beam. (Fig. 119.)

Results of tension tests on specimens 1, 2, 5 cut out of Beam X, and of repeatedly loading specimens 3, 4, 6 cut out of same Beam. (Fig. 119.)

Loads in lbs.	Specimen Readings taken by Extensometer. 1 Extr.		2 Rule	3					4									5	6				
	Forward	Forward	Forward	Forward.	Return.	Forward	Return.	Forward	Forward.	Return.	Forward	Return.	Forward	Return.	Forward	Return.	Forward	Forward.	Forward.	Return.	Forward	Return.	Forward
100	0	0	....	0	....	....	....	...	6	....	....	....	....	....	....	....	....	0	0	....	....	....	....
200	62	52	....	65	....	....	....	....	78	....	....	....	....	....	....	....	....	72	71	....	....	....	....
400	172	154	....	213	259	259	262	262	194	201	201	212	212	221	221	225	225	220	196	213	213	218	218
600	284	256	....	358	....	....	....	....	326	....	....	....	....	....	....	....	..	364	329	....	....	....	....
800	403	365	....	497	540	527	551	524	469	481	492	490	500	498	506	503	506	504	467	488	484	490	489
1,000	523	468	....	634	....	....	....	....	612	....	....	....	....	....	....	....	....	647	609	....	...	....	....
1,200	648	576	....	771	801	801	816	803	750	....	757	760	761	765	768	772	774	785	750	763	759	767	762
1,400	769	678	....	907	....	....	....	....	878	887	....	....	....	....	....	....	....	924	890	....	....	....	....
1,600	892	775	....	1050	1050	1070	1070	1074	1019	1019	1027	1027	1033	1033	1039	1039	1041	1061	1029	1029	1031	1031	1037
1,800	1013	873	....	....	....	....	....	1210	....	....	....	....	....	....	....	....	1185	1189	....	....	....	....	1172
2,000	1136	971	....																				
2,200	....	1044	0																				
2,500	Failed at section where grain was curly, due to proximity of a knot.		3												This test-piece was cut out from near the heart.			Failed at two small knots.					
3,000			9																				
3,500			13																				
4,000			18																				
4,500			23																				
5,000			29																				
5,500			31																				
6,000			35																				
6,500			40																				
7,000			43																				
7,500			47																				
8,000			51																				
8,500			57																				
9,000			61																				
9,500			66																				
10,000			70																				
10,500			74																				
11,000			78																				
11,500			83																				
12,000			89																				
Total breaking weight in lbs.	7460	1243	....	7228	....	....	....	....	7340	....	....	....	....	......	....	....	....	6680	8424				
Break'g weight in lbs. p. sq.-in.	10,376	17,492		10,191					10,279									992	11,535				
Co-efficient of elast'ty in lbs.	2,308,650	2,846,900		2,021,350					2,036,900									2,134,450	1,973,150				

Results of tension tests on specimens cut out of Douglas Fir Beam X, and of repeatedly loading another specimen cut out of same Beam (Fig. 119).

Readings taken by Extensometer.

	Specimen 7							8	7				
Loads in lbs.	For. ward.	Return.	For-ward.	Return.	For-ward.	Return.	For-ward.	For-ward.	For-ward.	Return.	For-ward.	Return.	For-ward
100	0	......	......	......	......	......	......	0	0	......	......	......	....
200	58	......	......	......	......	......	......	69	78	......	......	......	....
400	174	172	172	*176	171	179	179	214	193	222	222	228	228
600	299	......	......	......	......	......	......	341	316	......	......	......	....
800	417	410	412	413	416	418	417	468	430	458	453	463	459
1000	534	......	......	......	....	......	......	602	545	......	......	......	....
1200	654	656	656	659	660	665	661	731	760	680	680	689	683
1400	776	......	......	......	......	......	......	860	875	......	......	......	....
1600	898	......	......	......	......	......	......	983	890	......	......	......	....
1800	1019	1019	1023	1023	1027	1027	1029	1121	1005	1015	1017	1017	1020
....	......	......	......	......	......	......	1153	......	1120	1120	......	......	1136
200	7030	......	......	......	......	......	......	7700	9270	......	......	......	....
Total break'g w'ght in lbs.	9743	......	......	......	......	......	......	1,1140	13,071	......	......	......	....
Brk'g weight in lbs per sq. in	2,296,350	......	......	......	......	......	......	2,233,150	22,9,6350	......	......	......	....
Co effic'nt. of elasticity in lbs.													

* After this, the 4th series of readings, the test-piece was allowed to rest for a period of 2 hours. On resuming the testing the reading was ,000171.

NOTE.—In test-pieces 7, 8 and 9, the grain was somewhat oblique to the direction of the axis.

Results of tension tests on specimens 1 to 6 cut out of Douglas Fir Beam XII, and of repeatedly loading specimen 3 cut out of same Beam. (Fig. 120).

Results of tension tests on specimens 1 to 6 cut out of Douglas Fir Beam XII, and of repeatedly loading specimen 3 cut out of same Beam. (Fig. 120).

Loads in lbs.	Readings taken by 1 Extr.	2 Extr.	3 Extr.	4 Extr.	Rule	5 Extr.	6 Extr.	1 Extr.	2 Extr.	3 Extensometer. Forward	Return.	Forward	Return.	Forward	Return.	Forward	Forward
100	0	0	0	0	....	0	0	0	0	0	57	57	70	70	82	82	....
200	19	82	69	59	....	66	73	79	75	54	....	....	...	....	....	..	....
400	282	232	192	142	....	170	220	228	211	172	232	225	241	237	259	248	....
600	343	382	313	263	....	284	369	384	363	299	....	....	....	....	....	....	....
800	505	548	442	387	...	390	517	540	525	419	467	441	478	465	489	475	....
1,000	754	720	600	505	....	509	666	702	670	539	.	....	....	....	....	....	....
1,200	806	875	759	629	....	613	815	856	817	665	692	687	708	697	715	705	....
1,400	1055	1032	907	749	....	722	955	1009	953	792	....	....	....	....	....	....	....
1,600	11 3	1179	1030	869	....	848	1107	1118	1100	914	....	....	....	....	....	....	....
1,800			1214	986	....	949				1032	1032	1040	1040	1042	1042	1051	....
2,000				1106	0	1063										1178	0
2,500				In this test-piece the central portion was pulled through the head, so that its tensile strength exceeded the breaking weight.	5	This test piece commenced to fail at a small knot.	This test-piece failed at a section where the grain was curly from proximity to a knot.									In this test-piece the central por ion was pulled through the head, so that its tensile strength exceeded the br'kg weight.	7
3,000					10												12
3,500					15												17
4,000					20												22
4,500					25												28
5,000					30												33
5,500					35												38
6,000					40												43
6,500					45												48
7,000					50												54
7,500					55												59
8,000					60												64
8,500					66												
9 000					73												
9,500					81												
10,000					90												
10,500					100												
11,000					110												
Total break'g w'ght in lbs.	10,620	10,760	10,760	11,120	....	9,900	5,510	10,220	9,300	10,420							
Brk'g weight in lbs.p.sq.in.	14,886	15,327	15,040	15,655	....	13,909	7,823	14,660	13,066	14,640							
Co-effic'nt of elast'y in lbs	1,791,800	1,805,050	2,001,650	2,307,200	....	2,486,700	1,920,900	1,934,100	1,896,500	1,960,450							
Time of test in minutes..	10	10	10	16	....	12	9	8	9	41							

Results of tension-tests on specimens cut out of Beam XIII, and of repeatedly loading other specimens cut out of the same Beam (Fig. 121).

Readings taken by

Loads in lbs.	Extensometer. 1											Rule	Extensometer. 2	3	4	1	2	3	4				
	Forward	Return.	Forward	Return.	Forward	Return.	Forward	Return.	Forward	Return.	Forward	Forward	Forward	Forward	Forward	Forward	Forward	Forward	Forward	Return.	Forward	Return	Forward
100	0	....	....	....	....	....	....	....	....	....	....	....	0	0	0	0	0	0	0	....	....	....	....
200	64	....	....	....	....	....	....	....	....	....	....	....	70	119	64	101	62	85	67	....	....	....	....
400	180	172	172	*180	182	198	198	199	199	202	202	....	199	306	215	216	181	287	191	239	239	253	253
600	295	....	....	....	....	....	....	....	....	....	....	....	337	491	338	420	300	454	315	....	....	....	....
800	406	409	405	416	418	435	427	454	428	435	430	....	472	613	465	572	418	620	442	490	485	505	499
1,000	510	....	....	....	....	....	....	....	....	....	....	....	610	839	591	723	534	786	575	....	....	....	....
1,200	620	623	627	637	647	659	651	661	653	664	656	....	742	1015	720	876	652	950	706	730	739	748	751
1,400	727	....	....	....	....	....	....	....	....	....	....	....	872	1179	844	1030	772	1110	831	....	....	....	....
1,600	832	....	....	....	....	....	....	....	....	....	....	....	1006	....	962	1185	891	....	957	....	....	....	....
1,800	939	939	951	951	975	975	981	981	983	983	983	....	1140	....	1090	....	1012	....	1078	1078	1102	1102	1118
2,000	....	....	....	....	....	....	....	....	....	....	1092	0	....	....	1210	....	1132	....	....				
2,500	....											5	....	....	....	....	....	....	....				
3,000	....											10	....	....	....	....	....	....	....				
3,500	....											15	....	....	....	....	....	....	....				
4,000	....											20	....	....	....	....	....	....	....				
4,500	....											25	....	....	....	....	....	....	....				
5,000	....											30	....	....	....	....	....	....	....				
5,500	....											34	....	....	....	....	....	....	....				
6,000	....											39	....	....	....	....	....	....	....				
Total break'g w'ght in lbs.	7520												9840	5140	8720	7490	11,620	4370	9320				
Br'kg weight in lbs. per sq. in.	10,638												13,945	7322	1,2337	10,191	15,271	6278	3,721				
Co effic'nt of elasticity in lbs.	2,609,400												2,108,500	1,631,700	2,263,400	1,684,900	2,359,150	1,098,900	2,323,700				

* After this, the 4th series of readings, the test-piece was allowed to rest for 2½ hours. On resuming the testing, the reading was .000182.

Results of repeatedly subjecting to tensile stress a specimen cut out of Beam XV. (Fig. 122.)

Loads in lbs.	Specimen 1. Readings taken by Extensometer.															Rule.
	Forward.	Return.	Forward.	Return.	Forward.	Return.	Forward.	Return.*	Forward.	Return.	Forward.	Return.	Forward.	Return.	Forward.	Forward.
100	0	20	20	20	20	22	22	22	22	44	44	43	43	49	49	......
200	79	....	....	....	....	....	....	....	....	....	....	....	....	....	....	......
400	229	220	216	231	220	232	220	230	....	237	229	238	230	240	237	......
600	379	....	....	....	....	....	....	....	....	....	....	....	....	....	....	......
800	522	509	495	509	494	511	492	510	519	542	529	546	530	547	536	......
1,000	659	....	....	....	....	....	....	....	....	....	....	....	....	....	....	......
1,200	790	790	771	784	772	790	770	788	797	816	807	821	809	821	813	......
1,400	926	....	....	....	....	....	....	....	....	....	....	....	....	....	....	......
1,600	1059	....	....	....	....	....	....	....	....	....	....	....	....	....	....	......
1,800	1186	1186	1181	1181	1183	1183	1181	1181	1215	1215	1219	1219	1219	1219	1220	......
2,000															1358	0
2,500																6
3,000																12
3,500																18
4,000																23
4,500																29
5,000																35
5,500																40
6,000																46
6,500																51
7,000																57
7,500																63
8,000																69
8,500																76
9,000																81
9,500																90
Total breaking load in lbs.	10000															
Break'g load in lbs. per sq. in.	14,474															
Co-effic'nt of elasticity in lbs.	2,092,600															

* After this 8th series of readings the test-piece was allowed to rest for a period of 16 hours under the load of 100 lbs. On resuming the testing the initial reading was found to be unchanged.

Results of repeatedly subjecting to tensile stress specimens 1 to 4 cut out of Beam **XV**.

Readings taken by.
Specimen.

Loads in lbs.	2					3			4			1			3		4
	Extensometer.					Extr.			Extr.			Extr.			Extr.	Rule.	Extr.
	Forward.	Return.	Forward	Return.	Forward	Forward	Return.	Forward	Forward	Return.	Forward	Forward	Return.	Forward	Forward.	Forward.	Forward.
100	0	30	30	39	39	0	39	39	0	21	21	0	31	31	0	......	0
200	62	....	....	....	....	69	....	....	50	....	...	70	....	....	55	......	69
400	187	224	209	229	210	199	239	230	157	174	156	211	234	220	178	......	189
600	313	....	....	....	....	337	....	....	263	....	....	344	....	....	311	......	306
800	439	....	453	479	457	479	509	495	366	395	361	476	499	479	441	......	418
1,000	565	600	....	....	....	614	....	....	473	....	....	607	....	....	572	......	530
1,200	694	....	707	729	710	746	764	756	578	599	573	738	760	739	703	......	642
1,400	820	843	....	....	....	869	....	....	684	....	....	869	....	....	833	......	752
1,600	949	....	....	....	....	999	....	....	785	....	....	999	....	....	962	......	847
1,800	1075	....	1086	1086	1090	1130	1130	1152	884	899	879	1129	1129	1130	1091	......	956
2,000	1201	1201	....	....	1217	....	....	1288	985	....	....	.....	....	....	1220	0	1073
2,200	....	....	....	....	....	....	....	....	1088	....	....	....	....	....	......	......	......
2,500	....	....	....	....	....	....	....	....	....	....	....	....	....	....	......	6	......
																10	
																16	
																21	
																26	
																32	
																37	
																43	
																48	
																53	
																61	
																66	
																69	
																76	
																81	
																88	
																94	
																102	
Total breaking weight in lbs.	10960	....	....	....	....	7420	....	....	7720	....	....	10240	....	....	11000	......	8115
Breaking weight in lbs. per sq. in.	15346					10,61			11,117			15,004			15,619		11,686
Co-efficient of elasticity in lbs.	2,205,250					2,144,850			2,751,550			2,231,300			2,173,350		2,626,200

Results of repeatedly subjecting to tensile stress a specimen cut out of Beam XV. Fig. 122.

Loads in lbs.	Specimen 4. Readings taken by Extensometer.																					Rule.
	Forward.	Re-turn.	For-ward	Re-turn.	For-ward	Re-turn.	For-ward	Re-turn.	For-ward	*Re-turn.	For-ward	Re-turn.	For-ward	Re-turn.	For-ward	Re-turn.	For-ward	Re-turn.	For-ward	Re-turn.	For-ward	For-ward
100	0	58	58	69	69	75	75	75	75	77	0	9	9	16	16	....	....	....	....	....	....	....
200	51	....	....	....	....	....	....	....	....	....	43	....	....	....	....	....	....	....	....	....	....	....
400	163	190	189	202	197	212	203	212	203	214	125	121	111	114	108	114	114	112	112	112	112	....
600	245	....	....	....	....		....	....	....	....	212	....	....	....	....	....	....	....	....	....	....	....
800	338	369	361	385	368	393	374	393	373	394	301	310	285	299	281	295	287	294	288	294	288	....
1,000	430	....	....	....	....	....	....	....	....	....	387	....	....	....	369	....	....	....	....	....	....	....
1,200	524	554	545	567	560	574	556	574	556	576	475	477	461	468	457	463	459	461	458	460	459	....
1,400	620	....	....	....	....	....	....	....	....	....	563	....	....	....	....	....	....	....	....	....	....	....
1,600	713	....	....	....	....	....	....	....	....	....	650	....	....	....	....	....	....	....	....	....	....	....
1,800	809	825	826	826	834	834	833	833	834	834	739	742	723	723	720	720	716	716	716	716	716	....
2,000	904	....	....	....	....	....	....	....	....	....	825	825	....	....	....	....	....	....	....	....	803	....
2,200	1000	1000	....	....	....	....	....	....	....	....	....	....	....	....	....	....	....	....	....	....	893	....
2,400																					982	....
2,600																					1072	....
2,800																					1161	0
3,000																						....
4,000																						7
4,500																						11
5,000																						13
5,500																						17
6,000																						21
6,500																						25
7,000																						29
7,500																						31

Results of repeatedly subjecting to tensile stress a specimen cut out of Beam XV. Fig. 122.—*Continued.*

Loads in lbs.	Specimen 4.																					
	Readings taken by Extensometer.																					Rule.
	Forward.	Return.	Forward	Return.	Forward	Return.	Forward	Return.	Forward	*Return.	Forward	Return.	Forward	Return.	Forward	Return.	Forward	Return.	Forward	Return.	Forward	Forward
8,000																						38
8,500																						41
9,000																						43
9,500																						47
10,000																						50
10,500																						55
11,000																						60
11,500																						64
12,000																						69
12,500																						73
Total breaking weight in lbs.	12500																					76
Break'g w'gt in lbs. per sq. in.	18001																					
Co-efficient of elasticity in lbs.	3,141,900																					

* After this the 10th series of readings, the test-piece was allowed to rest entirely free from load for a period of 46 hours.

Results of tension tests on specimens 1 to 11 cut out of Douglas Fir Beam XVII. (Fig. 123.)

Results of tension tests on specimens 1 to 11 cut out of Douglas Fir Beam XVII. (Fig. 123.)

Loads in lbs.	Readings taken by															
	1 Extr.	Rule.	3 Extr.	Rule.	4 Extr.	Rule.	8 Extr.	Rule.	9 Extr.	Rule.	11 Extr.	Rule.	7 Extr.	Rule.	5 Extr.	Rule.
100	0	..	0	..	0	..	0	..	0	..	0	..	0	..	0	
200	61	..	56		58	..	95	..	71	..	101	..	91	..	93	
400	185	..	165		177	..	289	.	210	..	266	..	289	..	240	
600	286	..	278	..	301	..	471	..	344	..	419	..	496	..	393	
800	408	..	391	..	425	..	655	..	481	..	560	..	680	..	550	
1,000	511	..	505	..	546	..	843	..	612	..	708	..	880	..	699	
1,200	618	..	620	..	669	..	1,057	0	745	..	848	..	1,073	..	854	
1,400	735	..	734	..	787	..	....	..	877	0	996	..	1,271	0	1,006	
1,600	834	..	845	..	909	..	....	..	....	..	1,144	..	....	..	1,159	
1,800	955	..	960	..	1,026	..	....	..	....	..	1,285	0	....	..	1,313	0
2,000	1,060	0	1,023	..	1,153	..	....	12	....	3	....	3	....	9	....	3
2,200		..	1,185	0	1,279	0										
2,500	Failure largely due to shear.	5	....	5	....	5	....	20	....	8	....	9	....	19	....	10
3,000		11	....	10	....	10	....	30	....	13	....	17	....	..	....	18
3,500		18	....	14	....	16				18						25
4,000		22	....	19	....	22	Failed at a knot.		Failed at a knot ⅛ in. in diameter.	25	Oblique grain.		Oblique grain.		Failed at a knot.	
4,500		28	....	24	....	28				32						
5,000		32	....	29	....	32				38						
5,500		..	....	33	....	37										
6,000		..	....	38	....	43										
6,500		..	....	42												
7,000		..	....	48												
7,500		..	....	53												
8,000		..	....	58												
Total breaking weight in lbs.	5,500		8,150		6,500		3,200		5,180		3,000		2,920		3,000	
Break'g weight in lbs. per sq. in.	7,755		11,631		8,933		4,230		7,035		4,320		4,089		4,040	
Co-efficient of elasticity in lbs.	2,578,350		2,518,500		2,224,750		1,377,000		2,036,200		1,978,450		1,426,000		2,264,500	
Time of test in minutes.	27		18		23		14		13		23		18		15	

Results of tension tests on specimens 1 to 3 cut out of Douglas Fir Beam XIX. (Fig. 124.)

Readings taken by

Loads in lbs.	1 Extr.	Rule.	2 Extr.	Rule.	3 Extr.	Rule.	1 Extr.	Rule.	2 Extr.	3 Extr.	Rule.	1 Extr.	Rule.	3 Extr.	Rule.
100	0	..	0	..	0	..	0	..	0	0	...	0	..	0	..
200	50	..	50	..	58	..	57	..	50	61	....	58	..	78	..
400	190	..	183	..	212	..	190	..	153	190	....	179	..	235	..
600	320	..	290	..	315	..	296	..	279	339	....	276	..	358	..
800	448	..	384	..	425	..	422	..	400	478	....	327	..	490	.
1,000	589	..	520	..	550	..	544	..	520	620	....	479	..	620	..
1,200	741	..	619	..	661	..	665	..	642	760	....	584	..	758	..
1,400	864	..	730	..	790	..	782	..	766	898	....	695	..	881	.
1,600	1,008	..	869	..	902	..	901	..	890	1,041	....	795	..	1,013	..
1,800	1,128	0	970	..	1,034	0	1,033	..	1,008	1,142	....	898	..	1,148	..
2,000	....	3	1,090	0	....	3	1,151	0	1,140	1,240	0	1,000		1,276	0
2,200	....	..	....	..	....	..	....	..	In this test-piece the central portion was pulled through the head, so that its tensile strength exceeded the breaking weight.	In this test-piece the central portion was pulled through the head, so that its tensile strength exceeded the breaking weight.	....	1,104	..	....	..
2,500	....	9	....	8	....	9	....	5			10	....	..	....	5
2,600	....	..	....	..	....	..	....	..			....	1,308	0	....	..
3,000	....	14	....	12	....	13	....	10			16	....	4	....	10
3,500	....	19	....	16	....	19	....	14			23	....	11	....	16
4,000	....	25	....	21	....	24	....	18			29	....	16	....	22
4,500	....	29	....	26	....	30	....	23			36	....	21	....	28
5,000	....	35	....	31	....	35	....	29			41	....	27	....	33
5,500	....	42	....	36	....	39	....	34			51	....	32	....	39
6,000	....	49	....	40	....	43	....				59	....	35	....	48
6,500	....	54	....		....	..	....				70	....	41	....	52

7,000						76	45	58
7,500						89	50	68
8,000						93	53	73
8,500						107	56	
9,000						112	59	
9,500						121	65	
10,000							70	
10,500							75	
11,000							82	
11,500							85	
12,000	....	....	....	....	....	....	 90	
Total breaking weight in lbs.	11,140	12,600	10,700	11,520	12,480	9,500	12,300	8,200
Break'g wgt. in lbs. per sq. in.	15,543	17,199	14,581	16,960	18,856	14,210	16,805	11,725
Co-efficient of elasticity in lbs.	2,082,700	2,407,950	2,320,950	2,451,150	2,450,600	2,279,350	2,687,000	2,197,750
Time of test in minutes.	....	....	....	18	15	19	28	22

Results of tension tests on specimens cut out of Douglas Fir Beam XX (Fig. 125), and of the repeated loading of other specimens cut out of same Beam :—

Loads in lbs.	Readings taken by																		
	Extensometer. 2				3		3		6		8		9		1			6	
	Forward.	Return.	Forward.	Forward.	Extr.	Rule.	Extr.	Rule.	Extr.	Rule.	Extr.	Rule.	Extr.	Rule.	Extr. Forward.	Extr. Re-turn.	Extr. For-ward	Extr.	Rule.
100	0	70	70	..	0	..	0	..	0	..	0	..	0	..	......	....	...	0	..
200	220	252	243	..	58	..	71	..	98	..	69	..	90	..	......	....	....	70	..
400	619	530	641	..	180	..	240	..	232	..	218	..	235	..	.. .	....	0	194	..
600	987	991	1,005	..	263	..	375	..	364	..	249	..	376	..	......	....	118	324	..
800	1,366	1,366	1,385		429	..	510	..	516	..	485	..	521	..	......	....	233	455	..
1,000	....	....	1,773	0	563	..	650	..	680	..	623	..	672	..	0	22	361	589	..
1,200	....	....	....	5	688	..	791	..	827	..	759	..	840	..	......	......	478	721	..
1,400	....	....	....	10	835	..	931	..	970	..	900	..	1,011		......	....	603	854	..
1,500	....	....	....	..	....	..	....	..	....	..	....	..	.....	..	300	321	....	......	..
1,600	....	....	....	15	942	..	1,075	..	1,097	..	1,038	..	1,117	..	......	....	719	976	.
1,800	....	....	....	20	1,063	..	1,210	..	1,222	..	1,174	..	1,301	..	......	....	848	1,102	..
2,000	....	....	....	28	1,182	0	1,369	0	1,357	0	1,310	0	1,501	0	597	612	960	1,232	0
2,200	....	....	....	32	1,349	..	....	..	....	..	....	..	....	..	....	....	1,080	......	..
2,400	....	....	....	41	....	..	....	..	....	..	....	..	....	..	....	....	1,205	......	..
2,500	....	....	....	..	....	..	....	2	....	4	....	4	....	5	892	....	....	......	..
2,600	....	....	....	49	....	..	....	..	....	..	....	..	....	..	......	....	1,320	......	5
2,700	....	....	...	..	....	3	....	..	....	..	....	..	....	..	......	...	....	......	..
2,800	....	....	....	52	....	..	....	..	....	..	....	..	....	..	......	....	1,440	......	..
3,000	....	....	....	61	....	..	....	10	......	10	....	10	....	12	1,190	1,190	1,570	......	9
3,200	....	....	......	..	....	10	....	..	....	..	....	..	....	..	......	....	....	......	..

3,400						Failed at a pin knot.						
3,500					13	15	15	18				
3,700				15								15
3,800												
4,000					20	20	20	24				20
4,200				21								22
4,500					26	26	25	30				
4,700				27								30
5,000					32	30	30	36				
5,200				31								35
5,500					39	39	35	40				
5,700				38								40
6,000					44	45	42	50				
6.200				41								49
6,500					51	50	50	53				
6,700				50								52
7,000					60	60	53	58				
7.200				57								60
7,500					63	65	61	61				66
8,000					70	70	70					70
8,500					80	80						77
9,000					86							
Total breaking weight in lbs.	3,190	....	....	7,700	9,360	8,840	8,500	7,500	9,760	......	....	9,000
Break'g. wgt. in lbs. per sq. in.	4,631	....	....	10,783	13,265	12,133	12,024	10,610	14,171	......	....	12,710
Co-efficient of elasticity in lbs.	1,769,560	....	....	2,236,150	2,008,450	1,921,350	2,072,850	1,787,500	2,440,350	....	....	1,985,550
Time of test in minutes.	15	....	....	....	20	18	16	20	14	....	....	22

Results of tension tests on specimens cut out of Douglas Fir Beam XXI., and of the repeated loading of another specimen cut out of same Beam. (Fig. 126.)

Loads in lbs.	Readings taken by 1				2		3		4	
	Extensometer.			Rule	Extr.	Rule	Extr.	Rule	Extr.	Rule
	Forward.	Return.	Forward.	Forw'rd						
100	0	65	0	....	0	....	0	....	0	....
200	65	......	69	....	105	....	116	....	113	....
400	212	291	220	....	291	....	355	....	349	....
600	391	.....	360	....	455	....	630	....	600	....
800	529	571	498	....	620	....	918	....	880	....
1,000	663	......	626	....	810	....	1,244	0	1,229	....
1,200	806	853	775	....	1,011	....	......	....	1,539	0
1,400	948	......	918	....	1,234	....	......	....	....	....
1,500	......	......	...	....	......	....	......	10	....	5
1,600	1,090	1,118	1,050	....	1,428	....	Failed at a large knot.		Failed simultaneously at two sections.	....
1,800	1,239	......	1,199	....	1,593	....				....
2,000	1,385	1,385	1,340	0	1,731	0				14
2,500	In this test-piece the central portion was pulled through the head, so that its tensile strength exceeded the breaking weight.			7	......	6				25
3,000				12	......	11				37
3,500				17	......	18				50
4,000				22	......	24				60
4,500				30	......	32				
5,000				37	......	41				
5,500				42	......	50				
6,000				50	......	56				
6,500				56	......	63				
7,000				61	......	76				
7,500				70	......	82				
8,000				85	......	92				
Total breaking weight in lbs.	8,240				8,100	....	1,830		4,480	
Br'king wgt. in lbs. per sq. in.	11,565				11,095	....	2,485		6,157	
Coefficient of elasticity in lbs.	2,005,050				1,336,300	....	916,640		923,890	
Time of test in minutes.	44				35	....	14		27	

Results of tension tests on specimens cut of an old Douglas Fir stringer, Beam XXII., and of the repeated loading of another specimen cut out of the same Beam.

(Fig. 127.)	Readings taken by 1				2		3		4	
	Extensometer.			Rule.	Extr.	Rule.	Extr.	Rule.	Extr.	Rule.
Loads in lbs.	Forward.	Return.	Forward.							
100	0	......	......							
200	79	141	141	....	0	....	0	....	0	....
400	231	291	292	....	117	....	90	....	60	....
600	389	440	439	....	289	....	230	....	190	....
800	539	580	579	....	416	....	376	....	319	....
1,000	690	730	723	....	518	....	518	....	450	....
1,200	872	872	881	....	635	....	649	....	588	....
1,400			1,030	...	765	....	801	....	713	....
1,600			1,164	....	895	...	929	....	847	....
1,800			1,340	....	1,023	....	1,077	....	920	....
2,000				0	1,169	....	1,205	0	1,096	....
2,500				2	1,304	0		2	1,220	0
3,000				9		8		5		4
3,500				13		13		9		9
4,000				20		18		16		13
4,500				24		23		23		19
5,000				30		29		30		23
5,500				40		36		36		28
6,000				46		42		42		33
6,500				50		49		48		39
7,000				55		56		54		45
7,500				60		62		60		51
8,000				67		71		68		57
8,500				72		83		75		63
9,000				80		86				70
9,500						90				78
						98				
Total breaking weight in lbs.	8,800				10,000		8,320		9,340	
Br'king wgt. in lbs. per sq. in.	12,115				13,954		11,414		13,169	
Co-efficient of elasticity in lbs.	2,139,200				2,199,700		1,969,900		2,190,350	
Time of test in minutes.	17				18		14		14	

Results of tension tests on specimens cut out of Old Spruce stringer, Beam LVII. (Fig. 128.)

Loads in lbs.	Readings taken by												
	[illegible] Extr.	Rule.	Extr.	Rule.	1 Extr.	2 Extr.	Rule.	3 Extr.	Rule.	4 Extr.	Rule.	5 Extr.	Rule.
100	0	..	0	...	0	0	..	0	..	0	..	0	....
200	132	..	130	...	102	109	..	100	..	100	..	75	....
400	362	..	376	...	324	317	..	286	..	263	..	220	....
600	614	..	603	...	592	535	..	455	..	431	..	369	....
800	855	..	843	...	949	818	..	619	..	640	..	525	....
1,000	1,121	..	1,071	...	1,179	1,130	..	834	..	817	..	678	....
1,200	1,442	0	1,303	0	1,416	1,340	0	1,017	..	1,022	..	829	....
1,400	....	..	....	...	....	....	..	1,060	..	1,169	..	979	....
1,500	....	7	....	8	....	....	9	....	..	....	..	....	....
1,600	....	..	....	...	....	....	..	1,239	0	1,356	0	1,124	....
1,800	....	..	....	...	....	....	..	....	..	....	..	1,252	0
2,000	....	19	....	18	....	....	19	....	7	....	8	....	2
2,500	....	32	....	29			31	....	12	....	13	....	8
3,000	....	45	....	39		Failed at a small pin-knot.	46	....	20	....	20	....	13
3,500	....	57	....	50			56	....	29	....	29	....	20
4,000	....	69	....	62			68		39	....	38	....	27
4,500	....	82	....	75					49	....	46	....	32
5,000	....	99	....	89					60	....	..	....	39
5,500	....	..	....	105					71	....	..	....	48
6,000									81	....	..	....	56
6,500									92	....	..	....	64
7,000													72
7,500													80
8,000													88
8,500													96
Total breaking weight in lbs.	5,500		5,700		6,830	5,660		6,970		7,080		9,000	
Breaking weight in lbs. per sq. in.	7,662		7,941		9,564	7,739		10,069		10,175		12,626	
Co-efficient of elasticity in lbs.	1,032,050		1,202,350		1,025,850	1,069,350		1,818,950		1,577,900		1,903,200	
Time of test in minutes.	18		17		16	17		18		16		20	

Results of tension tests on specimens cut out of Old Spruce stringer, Beam LX. (Fig. 129.)

Loads in lbs.	Readings taken by					
	5 Extr.	Rule.	6 Extr.	Rule.	8 Extr.	Rule.
100	0	..	0	..	0	
200	54	..	127	..	90	
400	191	..	276	..	259	
600	344	..	468	..	445	
800	497	..	652	..	610	
1,000	657	..	870	..	780	
1,100	....	..	960	0		
1,200	811	..	....	..	950	
1,300	....	..	....	..	1,040	0
1,400	967					
1,500	1,040	0	....	5		
1,600	....	.	....	..	....	5
1,800	....	5	....	..	....	8
1,900	....	.	....	11		
2,000	....	9	....	..	....	11
2,300	....	..	....	18		
2,400	....	14	....	..	....	17
2,700	....	..	....	25		
2,800	....	20	....	..	....	23
3,100	....	..	....	31		
3,200	....	25	....	..	....	29
3,500	....	..	....	37		
3,600	....	31	....	..	....	35
3,900	....	..	....	45		
4,000	....	35	....	..	....	41
4,300	....	..	....	50		
4,400	....	40	....	..	....	49
4,700	....	..	....	57		
4,800	....	43	....	..	....	54
5,000	....	50	....	61	....	57
5,400	....		....	..	....	63
5,500	....		....	70		
6,000	....		....	80	....	
6,500	....		....	88		
Total breaking weight in lbs.	8,100		6,750		5,600	
Breaking weight in lbs. per sq. in.	11,445		10,206		8,004	
Co-efficient of elasticity in lbs.	1,830,650		1,547,350		1,647,150	
Time of test in minutes.	22		31		22	

cut out of same Beam. (Fig. 130.)

Readings taken by

Loads in lbs.	Extensometer. 1 For-ward.	Re-turn.	-ward	Re-turn.	For-ward	Re-turn.	For-ward	Rule	Extr. 2	Rule	Extr. 1	Rule	Extr. 3	Rule	Extr. 5	Rule
100	0	59	59	68	68	78	78	....	0	....	0	....	0	....	0	....
200	76	....	....	....	....	....	....	....	91	....	82	......	70	....	79	....
400	224	265	255	274	263	293	276	....	233	....	226	....	198	..	242	....
600	358	....	....	....	....	....	....	....	389	....	372	....	338	....	413	....
800	492	530	526	555	535	559	548	....	523	....	522	....	475	....	570	....
1,000	631	....	....	....	....	....	....	....	671	....	679	....	613	....	729	....
1,200	774	801	801	821	807	834	821	....	819	....	845	....	759	....	881	....
1,400	913	....	....	....	....	....	....	....	964	....	993	....	892	....	1050	....
1,500	1051	1051	1074	1074	1085	1085	1098	0	1100	0	1175	0	1028	0	1198	....
1,800								....		......		....		....	1321	....
2,000								7		5		7		4	1475	0
2,400								....		....		....		11		....
2,500								11		12		13		....		6
2,800								....		....		....		15		....
3,000								17		18		19		....		11
3,400								...		....		...		22		....
3,500								22		23		25		....		19
3,800								....		....		....		27		..
4,000								27		29		31		....		25
4,400								....		....		....		33		....
4,500								32		34		37		....		31
4,800								....		....		....		38		....
5,000								38		39		43		....		38
5,400								..		...		.		44		....
5,500								44		45		50		....		48
5,800								....		....		....		52		....
6,000								50		50		57		53		57
6,400								....				....				....
6,500								56				64				64
6,800								....								
7,000								61								
7,500								67								
8,000								73								
8,500								80								
Total breaking w'gt in lbs.	8,980								6,349		6,640		6,900		7,000	
Brk'g w'gt in lbs. per sq.in.	12,792								9,157		9,724		9,881		9,905	
Co-effi't of elast'cy in lbs.	2,066,950								1,999,050		1,851,850		2,070,600		1,836,300	
Time of test in minutes...	39								15		14		21			

Results of tension-tests on specimens cut out of a 2 in. x 4 in. Red Pine scantling, and also of the repeated loading of another specimen cut out of same scantling. (Fig. 131.)

Readings taken by

Loads in lbs.	Extr.	Rule	Extr. Forward.	Extr Return.	Extr Forward	Rule Forward	Extr.	Rule
100	0	....	0	23	0	...	00	....
200	60	....	58	....	55	....	56	....
400	190	....	179	187	173	....	182	....
600	311	....	286	....	279	....	306	....
800	432	....	391	401	396	....	433	....
1,000	553	....	495	....	492	....	559	....
1,200	678	....	600	614	599	....	682	....
1,400	804	....	708	....	712	....	812	....
1,600	929	....	816	837	816	....	942	....
1,800	1053	....	927	....	925	....	1074	....
2,000	1179	...	1035	1045	1039	....	1202	....
2,200	1306	....	1143	....	1142	....	1335	....
2,400	1429	0	1257	1257	1257	0	1461	0
3,000	....	5	....			5	....	6
3,500	....	12	....			10	....	12
4,000	....	18	....			14	....	18
4,500	....	21	......			19	....	22
5,000	......	28	....			23	....	28
5,500	....	30	....			29	....	33
6,000	....	35	....			33	....	40
6,500	....	41	....			39	....	45
7,000	....	49	....			43	....	50
7,500	....	52	....			50	....	55
8,000	....	57	....			52	....	60
8,500	....	62	....			60	....	69
9,000	....					62	....	74
9,500	....							
Total brk'g weight in lbs...........	9,000		9,280				9,500	
Breaking weight in lbs. per sq. in....	12,689		12,775				14,372	
Co-efficient in elasticity in lbs......	2,279,850		2,554,150				2,247,350	
Time of test in mins.	24		20				30	

Results of testing specimens cut out of White Pine Beam, and of repeatedly loading other specimens cut out of same Beam. (Fig. 131A.)

Specimen.

	Measurements taken by									
	3	7								
	Extr.	Extensometer.								
100	0	0	....	....	....	....	....	....	....	....
200	76	81	....	....	....	....	....	....	....	....
400	239	241	268	268	278	278	288	288	298	298
600	409	405	....	....	....	....	....	....	....	....
800	579	569	589	590	603	600	613	610	629	622
1,000	748	732	....	....	....	....	....	....	....	....
1,200	914	899	....	....	....	....	....	....	....	....
1,400	1,082	1,069	1,069	1,083	1,083	1,096	1,096	1,109	1,109	1,118
1,600	1,209									1,285
1,800										
Total breaking load in lbs.	8,260	7,4.0								
Break'g load in lbs. per sq. in.	12,252	11,128								
Co-efficient of elasticity in lbs.	1,835,100	1,799,100								

Results of testing specimens cut out of White Pine Beam, and of repeatedly loading other specimens cut out of same Beam. (Fig. 131A).—*Continued.*

Specimen.

Measurements taken by											
8	6					Rule.		4			5
Extr.	Extensometer.						Extr.	Extensomer.			Extr.
0	0	....	....	...	....	..	0	0	....	....	0
86	80	....	....	...	....	..	80	111	....	....	91
253	220	233	233	239	239	..	249	336	336	336	255
419	385	....	....	...	....	..	420	544	....	....	410
581	550	564	563	571	566	..	581	752	745	748	572
749	715	....	....	....	....	..	739	918	....	....	733
912	879	....	....	....	....	..	911	1,156	1,156	1,158	894
1,076	1,045	1,045	1,051	1,051	1,055	..	1,104	....	....	....	1,055
1,238					1,223	..	1,283	This specimen failed at two sections simultaneously.			
					1,390	0					
						2					
						10					
						17					
						24					
						30					
						39					
						48					
						53					
						60					
						66					
						73					
						81					
						90					
9,136	8,470						7,440	6,000			8,600
12,969	11,561						10,347	8,503			11,981
1,729,400	1,654,500						1,614,000	1,728,350			1,741,400

Results of repeatedly loading specimens 2, 8 and 9 cut out of White Pine Beam XLVIII. (Fig. 131A.)

Specimen.

	Measurements taken by Extensometer																		
	2					9					8								
100	0	....	....	....	....	0	....	....	....	....	0	....	....	....	....	....	....	....	....
200	92	....	....	....	....	78	....	....	....	....	92	...	....	....	...	....	...	....	....
400	265	274	274	274	274	232	251	251	256	256	254	285	285	* 292	324	340	340	340	340
600	420	....	....	....	....	392	....	....	....	....	423	....	....	....	....	....	....	...	....
800	583	603	591	605	593	548	565	564	570	569	591	613	613	619	663	668	671	672	677
1,000	749	....	....	....	....	705	....	....	....	....	760	....	....	....	....	....	...	....	....
1,200	912	....	....	....	....	865	....	....	....	....	929	....	...	...	....	....	..	....	....
1,400	1,078	1,078	1,079	1,079	1,083	1,027	1,027	1,030	1,030	1,034	1,098	1,098	1,102	1,102	1,150	1,150	1,158	1,158	1,162
1,600	....	....	....	....	....	....	....	....	....	1.192	....								
1,800	....	....	....	....	1,410	....	....	....	....	....	....								
Total breaking load in lbs.	6,840	....	....	.. .	....	8,316	....	....	....	....	9,624								
Break'g load in lbs per sq in.	9,321	...	....	....	....	11,624	....	....	....	....	14,273								
Co-efficient of elasticity in lbs	1,676,200	....	....	....	....	1,758,250	....	....	....	....	1,757,250								

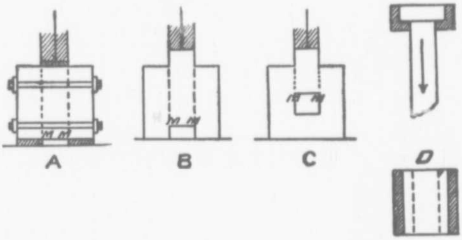
Specimens 2 and 8 failed at two sections simultaneously. Specimen 8, after the reading indicated by a *, was allowed to rest under the minimum load of 400 lbs. for an interval of 2¾ hours. When the loading was resumed the reading was .00324 in

Results of testing specimens 1 and 2 cut out of Red Pine Beam XXXI. and of repeatedly loading specimens 2 and 3 cut out of same Beam. (Fig. 121B.)

	Measurements taken by Extensometer.															
	1	2	2	1	2					3						
100	0	0	0	0	0	....	....	....	...	0	....	....	...	...	...	....
200	77	99	117	60	92	....	....	....	....	71	....	....	....	.	...	....
400	215	293	351	190	290	329	327	340	340	220	266	266	277	277	280	280
600	349	478	571	313	489	....	518	....	....	359	....	....	....	....	....	....
800	504	664	795	445	689	713	....	729	723	501	541	535	551	547	561	552
1,000	648	854	1,005	571	887	....	....	....	..	639	....	....	....	....	....	....
1,200	788	049	1,229	699	1,087	1.087	1,102	1,102	1,113	786	....	....	....	...	....	....
1,400	928	245	....	828	....	....	....	....	....	937	....	....	....		...	....
1,600	1,067	....	....	956	....	....	....	....	....	1,086	1.086	1,096	1,096	1,109	1.109	1.117
1,800	......	....	....	1,084	....	....	....	....	....	....	....	....	....	....	....	....
Total breaking weight in lbs.	8,460	6,928	4,620	7,910	5,592	....	....	....	....	6,790		....	....	...	....	....
Break'g wgt. in lbs. per sq. in.	11,825	9,378	6,274	10,889	8,090	....	....	....	....	9,508		....	....	...	....	....
Co-efficient of elasticity in lbs.	1,960,500	1,421,900	1.237,500	2.158,800	1,452,200	....	....	....	....	1,953,100		....	....	....	....	....

SHEARING STRENGTH.

In the experiments, to determine the shearing strength of timbers, considerable difficulty was found in preparing suitable test-pieces which would not at the same time be liable to a large bending action. Blocks were prepared as shown by sketches A, B and C; but unless the sides were sufficiently strongly clamped, as in Fig. A, the specimens almost invariably opened at M, under an effect chiefly due to bending. The clamping, again, introduced a compression, which rendered it impossible to obtain the true shearing stress.



After a number of experiments, more satisfactory and reliable results were obtained by preparing test-pieces as shown by Figs. E and D. The bending action is by no means eliminated, and, generally speaking, it is practically impossible to frame timber joints subjected to a pure shear only. The shearing strengths, which are of importance, are the resistances along planes tangential and radial to the annual rings. An examination of the test-pieces shows that the shears are invariably along these planes.

Thus it will be observed that in the tangential shears, the fibre, both hard and soft, is sheared radially, in the radial shears tangentially, and invariably through the soft fibre.

With test-pieces of the form shown by Fig. D, the shearing strengths along the tangential and radial planes are obtained, while the compound shearing strength, which may be considered as the resultant of the tangential and radial shears, is obtained with the test-pieces of the form shown by Fig. E.

The following tables give the results of experiments carried out with test-pieces and holders of the form described :—

TABLE OF THE TANGENTIAL, RADIAL AND COMPOUND SHEARING STRENGTHS OF DOUGLAS FIR SPECIMENS CUT OUT OF THE SAME BEAM.

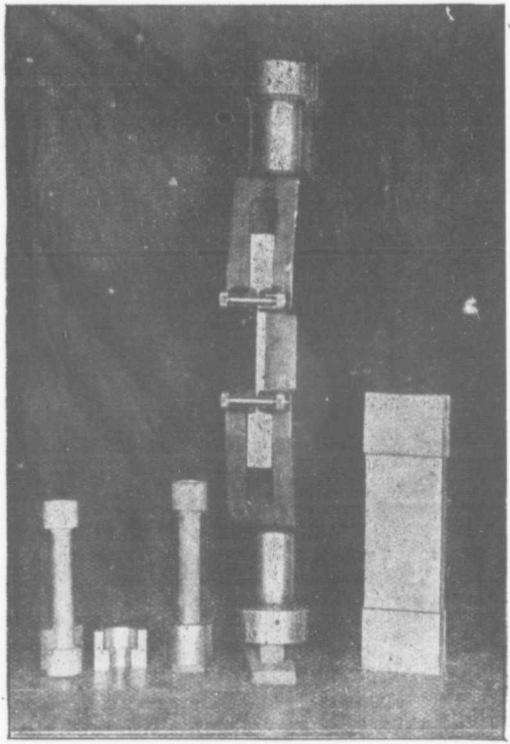
Specimen.	Shearing stress per sq. in. in a direction tangential to the annual rings.	Specimen	Shearing stress per sq. in. in a direction at right angles to the annual rings.	Specimen.	Compound shears.
No. 1	553	No. 3	560	*No. 13	471
No. 2	568	No. 5	484	*No. 14	536
No. 4	441	No. 7	544	No. 16	629
No. 6	555	No. 8	480	No. 16	657
No. 10	454	No. 9	436		
No. 11	415	No. 12	480		

TABLE OF THE COMPOUND SHEARING STRENGTHS OF DOUGLAS FIR AND RED PINE SPECIMENS.

Douglas Fir.		Red Pine.	
Specimen.	Shearing strength per square inch.	Specimen.	Shearing strength per square inch.
No. 1	802 lbs.	No. 1	648 lbs.
No. 2	727 "	No. 2	553 "
No. 3	886 "	No. 3	572 "
No. 4	795 "	No. 4	570 "
No. 5	706 "	No. 5	731 "
No. 6	649 "	No. 6	534 "
No. 7	746 "	No. 7	671 "
No. 8		No. 8	698 "
		No. 9	740 "
		No. 10	757 "

Not being altogether satisfied with these results, as the test-pieces did not seem to be of sufficient size to give results which could be considered of standard practical value, new holders, with spherical seats, were designed, and are shown in Fig. F.

With these holders, tests can now be made upon specimens in which the shearing surface has a width of 8 ins. and a depth limited by the tensile strength of the timber, the maximum shearing area being 96 sq. inches. The web of the specimens is usually about .7 in. in thickness, so that the depth should not exceed .35 t/s, t being the tensile and s the shearing strengths in lbs. per sq. in. The depth of the shoulder form-



E F G

ing the bearing for the pressure required to produce the shear is about ½ inch, and is made of only sufficient sectional area to resist failure by compression, as the deeper the shoulder the greater will be the bending action introduced.

From the tables giving the results of the shearing experiments, the following inferences may be drawn :

a. The shearing strength of the timbers is much less near the heart than at a distance from the heart.

b. Generally speaking, the shearing strength increases with the weight per cubic foot.

c. The shearing strength increases with the density of the annual rings, or rather with the proportion of hard to soft fibre.

d. A failure sometimes occurs, for which it is difficult to find a complete explanation.

For example, the two specimens from Beam X, and designated in the Table by a *, were precisely similar in dimensions and in weight, and also occupied precisely similar positions relatively to the heart in the stick from which they were cut. One of these specimens failed under a shear of 470.24 lbs. per sq. in., and the other under a shear of 301.84 lbs. per sq. in., so that the shearing strength of the latter was more than 35 per cent. less than that of the stronger specimen. A careful examination of the surfaces of fracture showed no visible difference in the specimens, and the only possible conclusion to be drawn seems to be either that one of the specimens might have been drier than the other, and was therefore deficient in the element of water, or that the shoulders of the weaker specimen, at the end at which the failure occurred, were not cut very parallel with each other, and thus the greater part of the load might have been concentrated on one side.

e. As a result of the experiments, the average shearing strength of Douglas Fir in lbs. per square inch is 411.61, 377.14 or 403.605 according as the plane of shear is tangential, at right angles, or oblique to the annual rings.

In practice, therefore, it will be safe to adopt as the average co-efficients of shearing strength for Douglas Fir, 400 lbs. per sq. inch for shears tangential and oblique to the annual rings, and 375 lbs. per sq. inch for shears at right angles to the annual rings.

Note.—The numbers in brackets at the end of the total shears in the following table correspond to the numbers in the diagrammatic sections, and indicate the position in the stick from which the specimens are taken. The letter H designates a specimen taken from the heart.


Fig 134

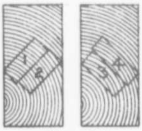


Fig 135

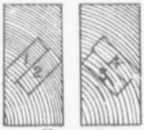


Fig 136

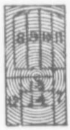


Fig. 137


Fig. 138

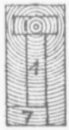


Fig. 139


Fig 140

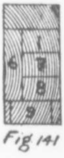


Fig 141


Fig. 142

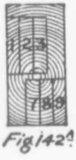


Fig 142a

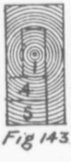
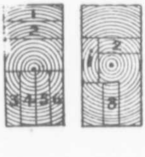


Fig 143


Fig. 144



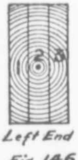

Left End
Fig. 146

Table of shearing strengths in lbs. of specimens cut out of various Beams.

DOUGLAS FIR.

Beam.	Tangential.		Radial.		Oblique.		Av. w'ght in lbs.
	Total.	Per sq. in.	Total.	Per sq. in.	Total.	Per sq. in.	Per cub ft.
IX	13,530 (1)	332·94	20,020 (4)	413·40	16,760 (2)	401·22	33·52
(Fig. 132.)	16,610 (1)	404·59	....	....	17,120 (2)	412·41	....
	16,170 (1)	375·47	....	....	14,720 (3)	393·41	....
	16,200 (5)	370·37	....	....	17,820 (3)	428·05	....
	17,210 (1)	412·48	....	....	15,820 (2)	372·01	....
	16,440 (1)	400·09	....	....	17,630 (3)	360·64	....
	....	....	....	....	19,570 (3)	367·29	....
	Average	= 382·65	Average	= 413·40	Average	= 455·94	
X	19,380 (2)	435·31	14,450 (1)	361·23	16,156 (3)	394·53	35·73
(Fig. 133.)	15,868 (2)	477·24	....	....	19,430* (1)	476·24	....
	16,660 (2)	406·14	....	....	12,424* (1)	301·84	....
	....	....	....	....	21,504 (4)	436·36	....
	....	....	....	....	24,880 (4)	511·41	....
	....	....	....	....	23,760 (4)	486·29	....
	Average	= 439·56	Average	= 361·23	Average	= 433·44	
XII	17,970 (1)	433·64	21,300 (2)	457·50	20,360 (1)	398·17	34·57
(Fig. 134.)	19,760 (1)	416·51	21,300 (2)	458·14	21,500 (1)	477·67	....
	....	....	16,160 (2)	377·81	....	....	....
	....	....	17,100 (2)	459·79	....	....	....
	Average	= 425·07	Average	= 438·31	Average	= 437·92	
XIII	16,984 (3)	462·15	17,886 (1)	464·60	....	....	31·81
(Fig. 135.)	14,552 (3)	395·22	16,980 (2)	441·04	....	....	....
	15,330 (4)	414·78	14,954 (2)	388·41	....	....	....
	15,210 (4)	409·97	....	....	....	....	....
	17,440 (3)	424·70	14,920 (1)	355·18	....	....	....
	12,940 (4)	443·79	15,350 (1)	367·07	....	....	....
	12,860 (4)	428·80	13,260 (2)	334·20	....	....	....
	19,600 (3)	478·37	14,610 (2)	350·55	....	....	....
	Average	= 432·22	Average	= 385·86			

DOUGLAS FIR—*Continued.*

Beam.	Tangential.		Radial.		Oblique.		Av. w'ght in lbs
	Total.	Per sq. in.	Total.	Per sq. in.	Total.	Per sq. in.	Per cub. ft.
XV	19,280 (3)	477·60	15,260 (1)	369·49	......	......	36·73
(Fig. 136.)	17,176 (3)	423·00	14,165 (1)	401·50	......	......	......
	16,170 (4)	420·00	17,914 (2)	431·56	......	......	......
	16,926 (4)	437·40	16,050 (2)	387·31	......	......	......
	Average	= 439·50	Average	= 397·46			
XVIII. ..	15,272 (14)	446·55	......	......	15,495 (7)	359·	......
(Fig. 137.)	......	......	......	......	15,600 (8)	411·9	......
	......	......	......	......	13,120 (9)	447·	......
	......	......	......	......	14,840 (12)	482·5	......
	......	......	......	......	12,595 (13)	402·	......
	......	......	......	......	17,180 (11)	380·	......
	......	......	......	......	12,500 (8)	389·7	......
	......	......	......	......	11,525 (9)	347·2	......
	......	......	......	......	19,420 (10)	382·1	......
	Average	= 446·55			Average	= 400·15	
XIX	16,040 (6)	409·1	14,430 (4)	375·7	14,470 (5)	393·2	38·4
(Fig. 138.)	20,390 (7)	422·6	14,220 (6)	388·9	20,830 (8)	442·	......
	18,470 (13)	395·3	14,590 (7)	411·8	17,200 (9)	371·	......
	14,650 (13)	340·	15,700 (4)	414·6	13,860 (5)	362·7	......
	19,580 (13)	416·5	15,200 (5)	418·5	15,500 (6)	437·6	......
	18,865 (7)	410·	......	......	......	......	......
	20,760 (13)	440·8	......	......	......	......	......
	Average	= 404·90	Average	= 401·90	Average	= 401·3	
XX	21,030 (7)	368·5	15,855 (4)	276·7	......	......	......
(Fig. 139.)	20,635 (7)	445·0	14,270 (1)	252·0	......	......	......
	21,190 (7)	360·4	17,630 (4)	378·2	......	......	......
	26,050 (7)	451·1	19,040 (4)	330·6	......	......	......
	Average	= 407·0	Average	= 309·37			
XXI......	18,700 (5)	350·	16,840 (1)	291·0	16,050 (1)	282·1	......
(Fig. 140.)	17,400 (2)	307·8	14,900 (3)	273·2	......	......	......
	17,800 (2)	394·	16,560 (3)	307·1	......	......	......
	Average	= 350·60	Average	= 290·43			

OLD DOUGLAS FIR.

Beam.	Tangential.		Radial.		Oblique.		Av. w'ght in lbs
	Total.	Per sq. in.	Total.	Per sq. in.	Total.	Per sq. in.	Per cub. ft.
XXII.....	14,220 (1)	31	12,175 (7)	287·0	17,150 (9)	371·	31·33
(Fig. 141)	13,370 (5)	29	14,630 (8)	333·0	....	....	....
	Average	= 302·00	Average	= 310·00	Average	= 371·	
RED PINE.							
XXXI....	20,780	430·22 (1)	....	....	13,020 (H)	379·59	33·71
	20,850	431·67 (1)	....	....	16,600	314·4	....
	20,700	386·9	....	....	18,680	347·2	....
	18,440	322·3	....	....	19,270	354·2	....
	Average	= 392·77			Averag	= 353·85	....
From 2 ins. x 4 ins. plank...	....	....	....	....	20,680 (H)	331·	....
	....	....	....	....	21,900 (H)	344·	....
	....	....	....	....	18,620 (H)	293·	....
	....	....	....	....	18,090 (H)	286·	....
					Average	= 313·5	
WHITE PINE.							
XLVIII ..	22,440 (1)	408·89	12,120 (7)	270·69	14,300 (3)	364·80	31·53
(Fig. 145 and	20,565 (2)	371·97	11,630 (7)	275·30	14,220 (5)	373·89	....
145A.)	16,160 (1)	430·67	....	....	18.505 (6)	352·35	....
	16,045 (2)	317. 96	....	....	....	....	....
	Average	= 382·37	Average	= 272·99	Average	= 363·68	
OLD SPRUCE.							
LVII.....	12,100 (6)	386·87	12,975 (3)	448·96	8,140 (4)	403·05	28·37
(Fig. 142A.)	....	....	11,390 (8)	408·88	9,280 (7)	447·85	....
	....	....	....	....	13,460	404·64	....
	....	....	....	....	16,075 (2)	457·84	....
	....	....	....	....	13,200 (9)	456·59	....
	....	....	....	....	12,480 (5)	322·00	....
	Average	386·87	Average	= 428·92	Average	= 415·33	
LX.......	16,650 (4)	302·7	....	....	17,130 (1)	292·	....
(Fig. 142.)	14,250 (5)	345·4	....	....	16,830 (3)	283·	....
	16,400 (4)	297·4	....	....	....	....	....
	Average	= 315·16			Average	= 287·5	
LXI......	13,100 (3)	329·1	14,800 (12)	460·73	14,000 (12)	436·78	28·6
(Fig. 144.)	....	....	14,840 (10)	314·6	12,820 (11)	299·1	....
	....	....	12,470 (9)	312·	13,460 (2)	404·64	....
	Average	= 329·1	Average	= 362·44	Average	= 380·17	

N. B.—I wish to express my acknowledgment of the help given to me by Mr. C. B. Smith, Ma.E., in carrying out many of the experiments and in checking the calculations. I have also been ably assisted by Mr. Withycombe, the foreman of the Laboratories, who has devised many mechanical devices which have greatly facilitated the work.

CORRESPONDENCE.

Prof. J. B. Johnson, M.Am.Soc. C.E., Professor of Civil Engineering Washington University, St. Louis, Mo., in charge U. S. Timber Tests, said:— Prof. J. B. Johnson.

While the writer desires to commend heartily the objects of the investigation here described, and to express his sense of the need of further studies of this kind, he is obliged to take exceptions to the methods and results herein reported in the following particulars.

1. The central load upon the beams was conveyed through a hard wood cylindrical bearing, having a ten inch radius. This offered so small a bearing surface to the timber, that in some instances it crushed bodily into the beam which was under test to a depth of two inches. Of course in practice no timber beam would ever be subject to so great a concentration of load as this, and it is therefore entirely unfair to so apply the load in making the test. In all tests of timber beams, the central bearing should be a saddle, preferably made of hard wood, being square at the bottom transversely, but curved longitudinally with a very long radius. When such a saddle is used, the distortion or compression of the upper fibres of the beam is insignificant, and can be neglected in the computation.

In the opinion of the writer, the abusive action of the central bearing used in these tests has to a large degree vitiated the results, and it is impossible now to determine what the normal strength of the beam was from the results obtained. It would seem that neither of the methods of computation offered by the writer of the paper has any great probability of being correct.

2. A very much more serious objection to these experiments is *the failure to make any determination of the percentage of moisture* in the beam at the time the tests were made. As a result of some fifty thousand tests on timber which the writer has made for the United States Government, reports of which are published by the Forestry Division of the U. S. Agricultural Department, it appears that the strength of wood in nearly all ways increases rapidly as the moisture is exhausted from the timber, and so great is this increase of strength, that on the average it may be said that *thoroughly seasoned timber is fully twice*

as strong as green timber. A test of any kind, therefore, on timber furnishes us no information as to the strength of that species of timber, unless we are also informed of the percentage of moisture found in the timber at the time the test was made. The absence of any such information as this in the work here under discussion would seem to make it of little value for comparative purposes.

In the work done by the writer for the U. S. Government, the percentage of moisture is found for every test made of every kind, by cutting from the stick an entire cross-section about one-fourth of an inch thick from the vicinity of failure, weighing this disk immediately, then drying it at a temperature of 212° F., and weighing again. The loss of weight, divided by the dry weight, gives the percentage of moisture in the stick, as compared with the dry weight. Before any comparisons can be instituted even between specimens of the same species, the law of the variation of strength with moisture for that species should be found, and then all the results of tests reduced by applying corrections to their equivalent values at some standard percentage of moisture. Hitherto this standard percentage of moisture has been arbitrarily taken in the U. S. tests as 15 per cent. of the dry weight. Thoroughly seasoned timber has about 10 per cent. moisture, whereas ordinary large timbers seasoned out of doors for several years will have a percentage of moisture of about 15. All these facts appear fully in the publications of the U. S. Agricultural Department on this subject, where the curves of variation of strength with moisture are all given. As these results have been before the American public since July, 1893, it would seem that no further tests of the strength of timber should ever be prosecuted without taking account of this greatest of all causes of variation in strength.

Aside from the two serious objections noted above, the work of Professor Bovey seems to have been well and carefully done. These two objections, however, have such great weight that I am persuaded his results have little or no scientific value, although they do give full information of the actual strength of the sticks tested. It is very unfortunate that we are unable to generalize from these tests and apply them to other known conditions. The objections here noted apply equally to nearly all tests made hitherto on the strength of timber, except those which are now being carried out by the U. S. Forestry Department.

Wherever in the results here described the specimen had been thoroughly seasoned, as is the case in several instances, it may be assumed that the amount of moisture in the stick was about 12 to 15 per cent. if the timber had remained out of doors, or about 10 per cent. if it had been in use for many years inside a building. Such timbers should be expected to have a strength nearly twice as much as they themselves would have had if they had been tested green.

Although the bulletins of the U. S. Forestry Division hitherto published on this subject have been entirely exhausted, another bulletin is about to appear, giving results of many thousands of tests on the four principal species of Southern Yellow pine, namely, Long-leaf (*Palustris*), Short leaf (*Echinata*), Loblolly (*Taeda*), and Cuban (*Cubensis*) Pine. Application for this publication should be made to the Secretary of Agriculture, Washington, D.C.

Mr. A. L. Johnson, of Washington, D.C. said: Mr. A. L. Johnson.

Of Mr. Bovey's excellent paper, the most novel and interesting portion, to the writer, is the chapter on

TENSION TESTS WITH REPEATED LOADINGS.

He here enters the unoccupied field of "Fatigue of Timber," and the experiments made are interesting and valuable. One of the first things to be noticed, on careful examination of these tests, is that the extensions obtained on the "Return" series are always greater than on the preceding "Forward" series.

This is natural, since the internal stress *follows* the external load, the action being a dynamic one.

Consequently the extensions are all too *small* for the recorded load on the "forward" series, and too *large* on the "return" series. As a result of this, and the method of making the test, the extension given for the minimum loads are all too *large*, while those for the maximum loads are all too *small*.

Hence, in a discussion of these results for the determination of either modulus of elasticity or "set," these values will have to be excluded. This consideration will also serve to explain why the minimum load left on for some hours sometimes gave less extension at the end of the time than at the beginning (see note on pp. 85, 87 and 88), in spite of the fact that timber has practically no elastic limit at all, any load left on for a sufficient time being able to produce a set.

The value of the extension at the minimum load after the period of rest is probably normal, while all the other values at the load are too large.

The first differential of extension, therefore, after the period of rest under minimum load, will be too large. The succeeding differentials, however, should not be affected, and on an examination of the tests they will be found perfectly normal. This is true only for light loads for a comparatively short time.

For heavy loads or long time, the modulus of elasticity seems to be injured.

Also, after a period of rest entirely free from load (see test on Beam XV, No. 4), the modulus of elasticity seems to be considerably increased.

Very similar, in fact, to muscular action.

To sum up, then, we may say:

1. That successive loadings, not exceeding 20 per cent. of ultimate strength, do in no wise permanently injure the material. Though some set is given it, this seems to disappear when left entirely free from load.

2. When a small load is left on for some time, the modulus of elasticity seems to be diminished.

3. That a period of rest, free from load, greatly assists in restoring the piece to its original strength and shape.

It now remains to try loads of varying amounts left on for different lengths of time, giving a complete discussion of the time element.

Also the effect of releasing the specimen from load. The next series, also, should include a discussion of the effect of these on the ultimate strength of the piece.

This, of course, will have to be done on separate pieces of comparable material, testing, for example, one end of a 3″ stick under repeated load, and the other end without such repetitions; making enough tests to eliminate lack of comparability due to peculiarities of the individual.

It will be noticed that the modulus of elasticity as determined from direct tension tests is 25 per cent. higher than indicated by the beams on the cross-bending tests.

This is interesting, if a correct relation, since, if the average modulus in direct compression and tension are equal to that in cross-breaking, the compression modulus must be about 25 per cent. weaker than the cross-bending modulus.

BEAM TESTS.

In the beam tests, the author is quite excusably at a loss to know what to do with his data after obtaining it. His use of a 20″ circular block for centre support has so deformed the original shape of his piece that he is in doubt as to what depth of beam to use in his fŏrmula. And, as he himself says, "a very small error in estimating the depth of a beam may lead to a considerable error in the calculated skin stress", citing spruce beam No. L as an example, in which case it made a difference of 22.8 per cent. He concludes to calculate this factor with the original height of beam, and of course gets results very low, but "on the safe side".

The beams are nearly all too deep for their length, and as a result many of them have sheared. In fact, on examination of table on page 18, taking only the "New Timber", we find that out of 19 beams tested, 9 sheared and 1 failed in a knot. That is to say, what is given as the maximum skin stress is for 50 per cent. of these beams, not the modulus of rupture, but much less. Therefore, considering that each individual value of this function is (due to crushing effect of centre support) from some per cent. to a maximum of 25 per cent. too small, and that of these values 50 per cent. are again considerably too small (since full skin strength was not developed), it is quite likely the *mean* given by Mr. Bovey is "on the safe side"! Besides, many of these beams had soaked in water—some in salt, and some in fresh—from 6 to 10 months. At least two of these laid on the beach and were alternately wet and dry, according to the tide, for a period of nine months.

To take means from such indiscriminate material is misleading. To classify and give means for each is impossible since too little data is left in each.

The above story is repeated in the

CRUSHING ENDWISE TESTS.

Out of 169 tests on New Douglas Fir, there were only 48 which were not manifestly defective before the test, and failed at these points.

That is to say, 72 per cent. of these tests are used to tell you that knots, cross-graining, and season checks are a source of weakness.

Of the remaining 28 per cent. of all sorts and conditions of pieces, having nothing in common but species, the mean tells nothing, except that, if you take the above number of pieces of the above number and kind of conditions, the mean thereof will approximate the mean here given.

The question may well be asked, "What is the object of these tests?"

Is it simply to determine factors of strength for *safe* design ?

If so, all that is necessary is to get a lot of poor, knotty, cross-grained stuff together and test it.

Or is it to determine factors of strength for *economic* design ?

If this is the object, it is altogether a different problem.

We must be able to say, not only that the material is strong enough, but that it is not too strong, or too good, for the purpose intended.

The uses, and requirements of these uses, must be classified.

The exact class to which the various kinds of material, under all the various kinds of treatment, belong must be determined.

The various effects of all defects are evaluated, and new rules of inspection determined.

Nor is this all. It should then be the endeavour, by new methods of treatment, to promote a material from a lower to a higher class.

This is the proper field of timber testing, or any kind of testing, and anything short of it—except to solve a specific problem for one specific purpose—is hardly worth while attempting.

SHEARING.

The classification of these tests into Radial and Tangential is a good one.

Mr. Bovey, however, gives his figures rather decisively to show that the tangential shearing strength is about 8 per cent. greater than the radial.

It is the opinion of the writer that this conclusion is not warranted.

In the first place, his results are not comparable. The pieces occupy different portions of the cross-section of log from which they are cut, and the variation of strength across the section is great.

The mean of 75 tests on Pinus Palustris made by this division on material as near comparable as possible give 6 per cent. greater strength to the radial than to the tangential shear.

The relative values of these two quantities depends upon the relative ratios of summer to spring wood sheared.

Mr. R. B. Fernow.

Mr. R. B. Fernow, of Division of Forestry, Washington, D. C.

I have just finished reading—nay, studying—your most interesting and valuable paper on the Strength of Douglas Fir, etc., having spent over five holy hours in acquainting myself with its contents and meaning. You may remember that I heard a part of it at the meeting of the Royal Society for the Advancement of Science last year, and being called upon to discuss it, refrained from doing so at length, only expressing my doubt whether the generalisations were justifiable on account of several deficiencies which appeared to me to exist as I heard the paper.

Now, after careful reading, I am confirmed in my doubts, although I fully appreciate this most acceptable addition to our knowledge of the behaviour of woods, and especially the painstaking work and presentation of the results, being thoroughly convinced that careful study of all the conditions surrounding any *one* test is worth more than the averaging of figures derived from many tests without knowledge or reference to the detail conditions. Yet I cannot help regretting that not more of the details of your test specimens was known or given, some of them most essential for a true interpretation of the results.

My criticisms then, if you care to have them, will take the form of a series of regrets. I regret then :

1. In general that so much empiricism still attaches to the series, that the tests are trials rather than experiments in which all the conditions that may have an influence on the result are taken cognisance of, or in part prepared or eliminated. The material under test, although an attempt is made to describe it, yet is only very partially described.

2. That no distinction of heart or sap, or the proportions of each in the test piece, is given.

3. That the relative moisture conditions of the test pieces is left to conjecture, although it is a well-established fact that small differences of moisture at certain stages of seasoning give differences in strength of thousands of pounds.

In some places, notably on p. 81 (beam XXI) and on p. 107 (tension pieces from beam X), it would appear as if a greater degree of seasoning was considered an element of weakness instead of the reverse. This *favourable* effect of seasoning seems also overlooked on p. 58, when comparing long and short columns.

The data given of loss of water in the laboratory indicate that much of the material was still green or wet, so that the weights given, which might otherwise be useful in relating strength to mass, lose this value.

4. That so many of the beams were designed so as not to develop their true transverse strength, failing in shearing. Of the Douglas Fir beams, more than 50 per cent. were thus at fault.

To evaluate transverse strength from such tests and use the figures in averaging with results from true transverse (tension or compression failures seems to me illogical and unwarranted. There may be value in such evaluations if they are kept separate, and are to refer only to beams designed to shear (designed for rigidity mainly), which seems in Douglas Spruce to take place invariably, when the ratio of height to length exceeds 1 in 15.

5. That the straight grained condition of some of the test pieces is asserted, presumably from the looks, without giving a basis for the assertion. Very frequently, as we have found, the grain *appears* straight and yet *is* spiral, and this can only be made sure of by splitting.

6. That so much of the material used for compression tests was defective (of the Douglas Spruce 72 per cent.), so that the compression value can hardly be said to have been established.

7. That the proof reader should persistently have allowed the recurrence of "annular" instead of "annual" rings, which jars upon one's eyes or mental ears.

Whether the method of loading at the ends, whereby span and angle of application of load are constantly varying, would appreciably influence results, and whether, on the other hand, with the changing of the effective depth due to the compression at the support the usual theory of flexure maintains, I leave to better mathematicians to discuss, although I am inclined to doubt the correctness of the latter assumption. The amount of compression taking place with the apparatus in use seems excessive.

The deduction that the wood farthest away from the heart is the strongest is in its generality decidedly erroneous. It may be correct with thrifty growing young trees of 60 to 100 years, because the proportion of the strong summerwood in the ring or rather per square inch is probably there at its maximum, but later in life this proportion skins again, and therefore in older trees the outer zone becomes again weaker, the best wood being, in conifers at least, found intermediate between heart and peripheral wood.

Most interesting to me, and without any flaw, as far as I can see, for general application, are the results from continued loading after first fracture, and of the repeated loadings and unloadings, although a great many more of the latter series will have to be done to clearly show the law of change in the set, due to "adjustment of parts" after repeated loadings.

Admitting the theory of flexure and the idea of skin or extreme fibre stress, there is nothing remarkable in the fact, that after first failure the same or even a greater strength is developed on second loading, provided the depth used in the calculation be reduced to that represented by the uninjured part. But the demonstration that this is really so is most useful.

Again let me congratulate you on this interesting contribution, which, although I have taken the liberty of pointing out its defects, is most suggestive and of much value and interest to me in our own work on similar lines.

Mr. J. H. Wicksteed, of Leeds, England : Mr. J. H. Wicksteed.

I am complimented and pleased by the advance proof you have sent me of your Paper on the Strength of Certain Timbers.

This paper will be a valuable standard for reference in the future on the strength of timber.

There are several points in the paper which I am very much struck with, and on which I should like to convey my remarks to you.

The striking tendency which the beams shew to shear longitudinally shews the great importance of testing long specimens instead of short ones, in order to arrive at the veritable strength of a beam in actual use, because as the long specimen has more length for cohesion of the fibres in the direction of longitudinal shear, it will be stronger in proportion than a short beam. Is this not your view?

I would therefore congratulate you on having made your experiments on such handsome sizes. I think this point is a rare proof of the superiority of a full sized test piece over a miniature sample.

While on this subject I should like very much to know whether you have found 9″ wide sufficient for your requirements, or whether you would not prefer if the machine had been able to admit a beam still wider.

Of course I recognize the further advantage that there is in using a long test piece for transverse straining, owing to the pressure on the central support being less intense, and in this connection I have pleasure in handing you herewith a tracing of a central support which I have recently designed. It consists of two swivelling plattens much on the principle of the thrust pieces you use for the ends of the beam, but arranged in a pair side by side so as to present a very wide surface to take the pressure on the centre of the beam. The point is that as these supports each swivel, they do not interfere with the deflection of the beam. They form a sort of articulated pressure foot, and by placing the end thrust pieces 3′ farther apart than the nominal span of the beam, you make an allowance for the 3″ distance that there is between the axes of the swivelling supports for the centre.

My friend Mr. Charnock of the Bradford Technical School has worked out this simple problem graphically, and I enclose you a tracing of the proof that the bending moment is the same with this broad foot as if the beam rested upon a theoretical edge.

I think with the use of this broad foot, you would be less bothered by the compression at the centre of the beam, and I shall feel very pleased if you approve this design so as to adopt it.

I notice from the photograph that you have improved the form of the brackets carrying the end thrust rams, doubtless in order to get open windows through them, so as to make the measuring gear accessible. I shall be greatly interested, if ever I find myself within reach of Montreal, to see the improvements you have made, and amongst other things to find exactly the means you have devised for ensuring absolute equality in the end loads.

Referring to your compression tests, I am greatly interested to see that you got the same resistance from a strut 20 diameters long as could be got from a short piece.—I suppose a simple cube.

Referring to the tension tests, I very much admire the smooth taper ends working within smooth wedge clips. This seems to me by far the best holding that has as yet been devised for wood.

I am also pleased to see the speed at which you made these tests; half an hour is not a long time for testing such a large piece and taking so many accurate observations. It implies that you have got the whole apparatus in first rate working order.

Mr. James E. Howard.

Mr. James E. Howard, Watertown Arsenal, Mass.:

Prof. Bovey has presented a very important paper on the strength of timber, and from its comprehensive character it possesses unusual interest.

In the case of timber, it is perhaps more difficult to judge of the strength of full sized members from the tests of smaller samples than with iron and steel, hence the transverse tests of the beams presented claim special attention.

The uniformity in strength found in small and carefully selected sticks can hardly represent the condition of beams of commercial sizes. It is believed furthermore that failures by longitudinal shearing occur more frequently with large sticks than with small ones.

The author invites attention to the fact that the ratio of deflection to load remains nearly constant almost up to the time of fracture, having previously stated "that timber, unlike iron and steel, may be "strained to a point near the breaking point without being seriously "injured," and further remarks, while referring to structures that have been heavily loaded, "whether it is advisable so to strain timber is "another question."

Questions of this nature are indeed very difficult to answer satisfactorily, and yet they seem to belong to that class of information most needed for practical use. It must be admitted that ordinary tests supply very little information concerning the probable endurance of the material under different conditions of loading.

The reverence which has attached to the elastic limit is disturbed by experimental demonstration that alternate stresses of tension and compression in rotating shafts eventually rupture the metal, notwithstanding the apparent maximum fibre stresses hardly exceed one-third the elastic limit of the metal, as that limit is commonly ascertained and defined by tensile tests.

Furthermore, material which, under direct tensile stress once applied, will develop 25 per cent. elongation before rupture may, under other conditions of loading, rupture with little or no measurable display of elongation.

These examples of iron and steel naturally awaken interest in the corresponding behaviour of timber.

The hygrometric character of wood, whereby in its unprotected condition it is continually changing its dimensions as it follows atmospheric changes, introduced an element of uncertainty, and might be supposed to assist the material in reaching its limit of rupture.

Owing to the absence of strict uniformity of timber in different parts of its cross sectional area, difficulty may often be experienced in securing the uniform distribution of the load on a post, and for the same reasons the disposition of stresses in a timber beam might exist in an equivocal state.

It appears that the compression tests submitted by the author consist of results obtained with small pieces, but illustrative of the strength of the material which comprised the beams. The influence of knots is well shown in the results.

Tests made at Watertown Arsenal have shewn that the presence even of sound knots is often more injurious than extensive seasoning cracks in the timber.

A somewhat extended series of tests was made at Watertown Arsenal during the fiscal year 1881-1882, in which single sticks of various sizes and lengths were tested and built up; posts of two, three and four sticks were also used. With four sticks tested together in a form resembling the compression members of a timber bridge, the sectional area aggregated 234 square inches.

These posts of white pine, which were 15 feet long each, showed a compressive strength in the vicinity of 2,000 lbs. per sq. in. At the time of making these tests, observations were made on the effect of load sustained for short intervals of time, and it was found that during the early stages of the tests the immediate effects of the loads were increased after a short time, and there was a sluggish recovery when the loads were released. And this behaviour became more pronounced as the test progressed.

There was a test made of a sample of white pine after it had been subjected in an hydrostatic cylinder to a pressure of about 90,000 lbs. per sq. in.

The water freely circulated through the wood, and the only visible effect of this enormous pressure was a slight swelling, which was apparently due to the absorption of water.

The compressive strength of this sample shewed no material change from the strength of a duplicate sample tested for comparison.

Tensile tests made at Watertown Arsenal have been upon specimens prepared with conical ends. The preparation of such turned specimens is expeditiously done, and no difficulty is experienced in shearing along the grain.

In making shearing tests, as pointed out by the author, difficulties are encountered in developing results uninfluenced by the form of the specimen employed.

It was thought that fairly reliable results were obtained with specimens prepared in the form of a Greek cross, shearing simultaneously two surfaces but rising surfaces of limited area.

Shearing along one surface would be preferred, other conditions being equal.

Prof. Bovey. Prof. Bovey, in making a brief reply to the various criticisms which have been passed upon his Paper, begs to thank those who have so kindly taken such an interest in the matter and have added valuable information to the subject matter of the Paper.

In the first place, a great deal of stress seems to be laid upon the very large compression which is supposed to have been occasioned at the bearing. Unfortunately the supposition is entirely due to a misprint in the Advance Proof, in which it is stated that the bearing block has a diameter of only 20 ins., whereas the diameter is in fact 44 ins. In the opinion of the author this diameter is certainly at least sufficiently large for the timber experiments, and the total compression was in every case, with two exceptions, extremely small. The exceptions are Beams LV and LVI, and these two beams were the two ends of Beam LIV from which the fractured portion had been cut out. The total compression of this Beam (LIV) was less than $\frac{1}{2}$ in. and the calculated maximum skin stress was 6,260 lbs. per square inch. Now, disregarding the compression, the skin stress in the case of Beam LV was 4,849 lbs. per square inch, and 4,614 lbs. per square inch in the case of Beam LVI, showing a very large difference between the skin stress of these two portions and the skin stress of the main beam. But the reason is not far to seek. The compression in Beam LV was 2 ins. and in Beam LVL was 1.9 in., and making due allowance for these compressions, the calculated skin stress becomes respectively 6,176 lbs. and 5,806 lbs. per square inch, showing a very small difference indeed from the stress of the main beam. These results sufficiently prove that when the amount of the compression is taken into account, the ordinary accepted formula for transverse strength gives results which are very approximately correct.

Again, it is stated that the beams were not properly designed, in other words, that the depth was too great as compared with the length, and that consequently some of the timbers sheared longitudinally so that the true transverse strength was not obtained. One of the objects of these tests was to determine the ratio of length to depth which would ensure the timber commencing to fail at the surface before shearing longitudinally. Certain results tending towards the solution of this problem have already been obtained, but further experiments on this point will be made. It must also be remembered that not only is it necessary that the timber should be sufficiently strong, but also that it should have sufficient stiffness, and this point seems to have been overlooked in the criticisms respecting the proper design of a beam.

In the next place, Mr. Fernow and Professor Johnson have set forth the great importance of determining the percentage of moisture present in a timber at the time of testing. The author quite agrees with these gentlemen as to the important effect of the presence of moisture upon the strength of the timber, and he has by no means neglected the investigation of this subject, but he is not at all prepared to accept the statements made respecting the comparative strengths of dry and moist timber. Further, the drying of a slab at 212° F. cut from the end of a timber will certainly not give the average weight of the whole timber, or the percentage of moisture present in the timber. Even in the same section the weight per cubic foot of the timber will be found to vary extremely with the distance from the heart. This is very forcibly illustrated in the case of Beam XIII. The section of this timber was divided into three equal parts, and they were thoroughly dried at 212° F. for 88 hours.

Before drying:

The portion nearest the heart weighed 30.208 lbs. per cubic foot.

The portion farthest from the heart weighed 36.492 lbs. per cubic foot.

And the intermediate portion weighed 28.512 lbs. per cubic foot.

After drying:

The portion nearest the heart weighed 29.123 lbs. per cubic foot.

The portion farthest from the heart weighed 35.096 lbs. per cubic foot.

The intermediate portion weighed 27.028 lbs. per cubic foot.

The average weight for the whole section was 31.445 lbs. per cubic foot before drying and 30.105 lbs. per cubic foot after drying. Besides, although it will be very important from a scientific point of view to be able to determine the percentage of moisture present, still it should be borne in mind that the structural work, as, for example, in bridges, the timber is taken to the site straight from the mill and is never kiln-dried. Thus the only strength upon which the engineer can depend is the strength as it leaves the mill, when there is usually a large amount of moisture present, and this strength of the timber, it is contended, is the strength which the engineer requires to know, as upon this strength he has to base his calculations.

Exception is taken to the fact that a large number of the compression pieces failed at knots, although the timbers were of first class quality. The auther is not aware that the finding of occasional knots in first class timber is at all unusual, and the results certainly justify his statements.

Thursday, 14th February.

THOMAS MONRO, President, in the Chair.

Paper No. 102.

CEMENT TESTING.

BY CECIL B. SMITH, MA. E., A.M. CAN. SOC.C.E.

This subject has so often been written on, and is being so continually and persistently investigated, that it forms, as it were, an inexhaustible mine.

But this very feature shows how very important and yet how little understood it is, for, when investigators continue to disagree, the presumption is, that there is either a lack of agreement as to the basis on which the investigations are made, or else a failure, up to the present, to solve all the intricate mazes of the problem, or indeed a combination of the two.

To illustrate the first point, a tabular synopsis (Table I) is presented, giving the present standard tests in use, in various countries, according to the latest obtainable information. The variations, in many cases, are too great to be reconciled, in others trifling; but it is evidently difficult to compare results obtained in different countries, and a hopeless task to ever bring them to a uniform standard. What it behooves us, as Canadian Engineers, to do is to take such sensible and immediate action on the subject as will commend itself to the good graces of all of us, if possible, or, if not, of a great majority of those who test the manufactured article.

However, before proposing a mode of conducting such tests as will (according to the author's experience) be of practical utility to practical men, the following Table (Table II) is presented to the Society, as embodying results which have been obtained during the last two sessions, in making ordinary commercial, private and student tests (chiefly commercial and private).

Many results have been discarded as being inaccurate, and only those are recorded here which are believed to be very close to the truth, much closer than is ordinarily obtained.

These results have been classified according to country of manufacture, and somewhat on a scale of increasing tensile strength.

Let us consider the various qualities given in their tabular order.

(*a*) *Specific Gravity.*

The average of Canadian Portlands = 3·11

The average of English Portlands = 3·10

The average of Belgian Portlands = 3·055

The average of all Portlands (16) = 3·09.

It would seem advisable, therefore, to specify a minimum for Portlands of 3·10.

The samples were not dried or prepared in any way; if they were dried for 15 minutes, according to English practice, it is probable they would go somewhat higher.

It will be noticed that the only two Portlands (?) whose specific gravities were low (Belgians Nos. 16 and 17) were both poor cements. One, No. 16, sets slowly, and the briquettes made for 4 week tests, and immersed in water after 24 hours, were found sloughed down in the tanks, and had evidently run and set over again! They would not give any test to speak of. Evidently the hydraul c property, in 24 hours, was not enough to hold them together, while the other one (No. 17) failed in the blowing test. Altogether, it is doubtful whether these cements are Portlands or naturals, although sold as the former, owing to their colour being gray.

It will be noticed, with satisfaction, that Canadian Portlands stand at the top in specific gravity, judging by the samples tested, which were, however, all received from manufacturers.

The specific gravity of natural cements might be placed at 2·95, although it is not so likely to be under-run, owing to the ease with which this can be obtained.

(*b*) *Water required for standard consistency.*

This is considered, by many, to be very important; but many tests have demonstrated to the writer that what is especially needed is that there shall be sufficient to make good briquettes; to err, say, 1 per cent. in adding water is fatal if too little, while if too much, it does not seem to affect the strength of briquettes at one week, certainly not at 4 weeks. This is contrary to statements often made regarding the increased strength given by a minimum amount of water; but probably what is referred to is an excess of water sufficient to make a thin batter or soup. Undoubtedly such an amount not only makes the briquettes shrink and crack in drying, but will seriously affect the early strength.

TABLE I.—STANDARD CEMENT TESTS.

Nationality.	Date of Standard.	Authority of Standard.	Weight per Bushel or C.F.	Specific Gravity.	Chemical Analysis.	Residues or Fineness.	P. c. of Water in Mixing.	Constancy of Vol. or Blowing Test.
Canadian.	1894	Recommended by Committee of C.S.C.E.	Considered to be indefinite and of little value.	3.12 to 3.25 } for Portland.	Not more than 2 % Sul. Ac. Not more than 5 % Mag. Portland: Lime 60.05, Silica 24.31, Al&FeO$_2$ 10.84, Mag. 3.00, Alkalies 1.60 Natural: Lime 37.18, Silica 28.11, Al. 27.62, Mag. 7.09	Less than 5 % on 50 sieve No. 35 stubbs gauge.	Standard consistency, rod .4″ diar. .66 lbs. to nearly penetrate mortar in a box 3″ diar. 1½″ high.	24 hours in water at 120° F. and 27 dys. in ordinary water or Final test, 24 hours after setting in boiling water.
English.	1893	Standard Practice no Regulations.	About 112 lbs. per bushel for Portland.	Not less than 3.10 for fresh or 3.07 for 3 months old (dried 15 min.)	Recommended to be made.	5 % on 80 siev 12 % on 100 " 25 % on 150 " 30 % on 180 " wire mesh.	Approximately 25 % Neat 12 % 3 to 1, Faija mixer.	Same as above, of which this is the original.
United States.	1885	Recommended by A.S.C.E. generally used and adopted.	(Authority of Clark.) French Port 69 per C F, English Port 78 to 87 p. C F American Port 95 per C F.	Not specified.	ditto	5 to 10 % on 50 sieve down to 3 to 10 % on 176 sieve.	Approximately 25 % Neat. Port., 30 % Neat, Natural, 15 % 1 to 1, 12 % 3 to 1 stiff mortar.	1 pat in air 1 month for signs of discoloring, 1 pat in air till set, then 1 month in water for checking.
German.	1893	Government regulations.	370 lbs per bbl net.	3.12 to 3.25 } for Portlands, increase with age.	ditto	10 % on 76 sieve wire ½ of mesh.	Same as the Canadian, which is a copy of this one.	23 hours in air, 1 hour boiling, or submerged 28 dys., no checking in either test.
French.	1884 or later.	Government regulations.	1 litre to weigh within 3½ oz of heav'st cement from same factory all sifted thro' No. 100 sieve.		Not more than 1 % Sul. Ac. " " " 4 % Fe O$_2$. When Si. and Al. are less than 44 % of lime.	None specified, argued that fine grinding gives high strength in periods of tests chiefly, which disappeared later on.	Sea water, standard consistency round ball dropped 20″ on slab to retain its general form without cracks.	Pat in sea water ($\frac{7}{1}$) days, no cracking or bulging.
Austrian.								

TABLE I.—STANDARD CEMENT TESTS.—*Continued.*

Tensile Strength.			Compressive Strength.			Setting Quality.	
Neat.	1 to 1.	3 to 1.	Neat.	1 to 1.	3 to 1.	How determined.	How defined.
	Not yet year later.	specified, to be	reported on 1			Gilmores' needles incipient to bear $\frac{1}{12}''$ diar. $\frac{1}{4}$ lb. full set to bear $\frac{1}{24}''$ diar. 1 lb.	
1 week 300-400 1 mo. 480-650		3 days 110 1 week 120-220 1 mo. 200-350				Vicat's needles incipient set to bear 66 lbs. to not quite penetrate full set to bear up same needle.	2 hours or more slow setting, less time quick setting.
Natural 1 day 40-80 1 week 60-100 1 mo. 100-150 1 year 300-400. Portland 1 day 100-140 1 week 250 550 1 mo. 350-700 1 year 450-800.	Natural 1 week 30-50 1 mo. 50-80. 1 year 200-300	Portland 1 week 80-125 1 mo. 100-200 1 year 200-350				Gilmore's needles.	
		1 mo. $227\frac{1}{2}$			1 mo. 2275	Vicat's needles.	Same as English.
Minimum. 1 week 285 1 mo. 498 3 months 640; to show 25 % increase 1 week to 1 month.		Minimum. 1 week 114 1 mo. 213 3 mos. 2[illegible]				Vicat's needles.	Incipient to be not less than 30′ full set to be not less than 3 hrs. or more than 12 hrs.
		1 week 114 1 mo. 171					

Kind of sand used.	How put in Moulds.	Rate of loading in tensile tests.	Time in air before immersions.	No. of tests used for Averages.	Time of Mixing.	Wearing Qualities.	Adhesive Qualities.
Standard crushed quartz to all pass No. 20 sieve all caught on No. 30 sieve.	10 lbs. per sq. in. steady pressure.	200 lbs. per minute.	24 hrs.	Not stated, probably 5	1 min. for quick setting, 2 minutes for slow setting, mechanical mixer.		
ditto	10 lbs. on briquette for5 min., or shaken in moulds or beaten with trowel for 1 min.	400 lbs. per minute.	24 hrs.	5	1 minute or more, mechanical mixer.		Mr. Mann 1 week 57, 1 mo. 78, 3 mos. 98 } ground glass. Fineness has great effect.
ditto	Pressed in with trowel without ramming.	ditto	24 hrs.	5 Smallest section only.	1 minute or more, hand or mechanical mixing.		
Standard crushed quartz, ½ to pass 20, caught on 30. ½ to pass 30, caught on 38 sieves.	Bohmes' apparatus, 150 blows with trip hammer weighing 4.4 lbs.	13 lbs. per minute.	24 hrs.	10	1 minute for quick setting, 3 min for slow setting cements.	1 to 1 and 2 to 1 give higher results than neat or 3 to 1 tough at 7 days as at 20 days.	Advised to be still reported on and investigat'd, to be made on ground glass.
Crushed Cherbourg quartz pass No. 20 caught on No. 30 sieves.	Filled in and tamped with rammer weighing 7 oz. till water stands on surface.	Not specified.	24 hrs. then in sea water of 59° to 64° F.	6 Mean of 3 highest taken.	5 minutes by hand on a slab, temp. of air 59° to 64° F.		
				10 Mean of 6 highest taken.			

A very peculiar effect was met with in two Canadian and one English Portlands. They were evidently fresh, and when mixed with a normal amount of water would work into a good plastic mass, but in about 1 to 2 minutes after the water was added, they would suddenly set, so hard that it was useless to attempt to put them in the moulds.

By increasing the per cent. of water to about 30, a thin batter was made, which could be got into the moulds before this action took place; of course this amount of water made the set very slow, and deadened the indurating action in 1 week tests.

When tests were made, several weeks later, on these cements, this effect had disappeared; perhaps someone connected with the industry can explain the cause of this action.

(*c*) *Residues or Fineness.*

The variation is enormous, as the following statement shows :—

	Residue on No. 50 Sieve %	Residue on No. 80 Sieve. %	Residue on No. 120 Sieve. %
Coarsest	31·4	52·2	61·2
Finest	0·25	2·7	6·7

The English Portlands are generally very coarse, as will be seen, and the selected Canadian ones fine.

It is not putting it too severely to say that specifying a certain residue on No. 50 sieve is a direct premium on coarse grinding, and so, in fact are neat tensile tests.

For instance, English brands No. 10, No. 11, No. 12, No. 13 and Nos. 14 A, 14 B, are all evidently ground to pass a specification of 5 per cent. residue on No. 50 sieve, and are all very coarse when sifted on finer ones, thus plainly showing the failure of the specification to obtain as good a product as possible.

The author would urge the severest requirements for fineness.

Various papers read and the statements of manufacturers themselves go to show that the increased cost is very slight, not more than 10c. per bbl. between ordinary and fine grinding,

$\left.\begin{array}{l}\text{10 per cent. residue on No. 80 sieve}\\ \text{20 per cent. residue on No. 120 sieve}\end{array}\right\}$ as maximums are not too high for present facilities for fine grinding; this would let in 3 out of 4 Canadian Portlands tested, 1 out of 10 English Portlands tested, 2 out of 4 Belgian Portlands tested, or in all 6 out of 18 brands. There are signs, however, that the English manufacturers are waking up to finer

O

n.			Transverse Strength.					No. of Tests.		
	3 to 1.		Neat 1″x1″ broken on 6″ centres.							
ns	1 wk.	wks.	1 wk.	2 weeks	3 weeks	4 weeks	1 year.	Tensile.	Comp.	Trans.
..	....	......	....	......	......	......	......	27	1	0
.	....	......	....	......	......	......	....	100	8	0
..	205	475	....	......	......	......	......	33	3	0
..	392	412($\frac{2}{1}$)	....	......	.....	.. ...	......	33	4	0
..	....	......	42	......	......	......	......	30	4	1
..	....	1025	99	......	......	......	......	35	7	2
..	....	......	....	......	......	.....	......	15	0	0
..	....	......	....	......	......	......	......	6	0	0
..	....	......	....	......	......	......	......	10	0	0
..	....	......	54	......	......	84	......	10	5	2
..	....	......	73	......	......	99	......	30	2	5
..	....	......	....	......	......	......	......	40	1	0
..	....	......	....	......	......	......	......	29	1	0
0	....	......	....	......	.....	112	......	105	9	2
.	..	......	....	......	......	......	......	44	0	0
..	....	......	....	......	......	......	......	14	0	0
..	....	......	....	.....	......	.. ..	......	30	0	0
.	....	......	....	....	......	......	....	30	0	0
..	....	......	....	.....	......	....	.....	93	0	0
.	...	......	..	......	.....	......	......	35	8	0
..	....	......	....	......	......	......	......	10	0	0
	....	......	....	......	......	......	......	13	0	0
..	....	......	....	.....	.....	......	......	50	0	0
.	718	900	. ..	......	......	......	......	24	6	0
.	...	......	....	.....	......	......	113	12	10	2
.	....	......	....	......	86	......	......	17	3	2
	....	......	....	......	......	59	......	13	9	2
.	....	......	....	......	......	......	60($\frac{3}{1}$)	6	2	1
.	....	......	....	......	.....	.. ...	.....	894	83	19

were lightly tamped with a small iron rammer.—C. B. S.

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TABLE II.

CONDENSED TABLE OF CEMENT TESTS. 1893–1894.

Designation of Origin.	No. in Table.	Obtained from.	Specific Gravity.	% water for standard consist'ncy	Residue % on Sieves. No. 20	No. 50	No. 80	No. 120	Blowing test result.	Time of setting in air. Incipient	Full.	Average Tensile Strength in lbs. per sq. in. Neat Cement. 3 dys.	1 wk.	2 wks	3 wks	4 wks	2 mos	3 mos	4 mos	6 mos	1 year
Canadian N	1	Dealer	3·01	33	0	7·2	12·5	18·3	very good	4°00′	7°45′	78	71	....	....	124	226	....	....	....	....
Canadian N	2	Maker	2·96	33½	0	2·9	11·7	21·4	good	0°45′	2°45′	99	150	....	....	268	377	448	478	492	....
Canadian P	3	Maker	3·12	25½	0	1·1	14·2	31·2	good	5°00′	20°00′	125	210	....	....	356	....	...	....	....	....
Canadian P	4	Maker	3·12	26	0	0·8	2·7	6·7	good	0°37′	3°10′	335	388	....	....	525	....	....	....	....	....
Canadian P	5	Maker	3·09	25	0	0·6	5·5	13·2	good	1°00′	5°00′	278	399	....	....	459	....	....	. .	....	....
Canadian P	6	Maker	3·12	24	0	0·9	6·4	13·2	good	4°30′	6°00′	438	588	....	....	671	....	....	....	....	....
Canadian P	6a	Dealer	....	24	0	3·0	13·6	20·7	very good	2°00′	6°30′	312	531	....	....	611	....	....	....	....	....
Canadian P	6b	Dealer	....	24	0	2·3	27·0	40·7	fair	.....	....	300	307	....	....	....	....	....	....	....	....
Canadian P	6c	Dealer	....	23	0	31·4	52·2	61·2	bad	....	....	253	264	....	....	....	....	....	....	....	....
English P	7	Dealer	....	33	0	10·5	21·6	....	good	....	....	....	230	....	....	331	....	....	....	....	....
English P	8	Dealer	3·09	25	0	14·0	28·4	39·5	good	3°20′	6°30′	160	414	..	....	528	....	....	....	....	....
English P	9	Dealer	3·09	26	0	1·4	12·8	22·9	good	13′	2°00′	390	420	....	....	453	....	....	....	....	....
English P	10	Dealer	3·10	23½	0	6·7	19·2	26·5	good	25′	50′	250	386	372	....	552	....	....	....	....	....
English P	11	Dealer	3·08	23	0	4·0	14·2	23·1	good	30′	1°00′	336	477	396	....	504	560	637	627	644	....
English P	12	Dealer	3·11	24	0	4·2	19·5	28·5	good	25′	3°00′	244	362	....	....	547	....	....	....	....	...
English P	13	Dealer	3·13	24½	0	4·5	17·3	26·3	bad	20′	4°00′	335	344	..	....	422	....	....	....	....	...
English P	14	Dealer	3·13	24	0	0·25	5·0	12·9	very good	20′	2°30′	....	387	444	....	495	....	....	....	....	....
English P	14a	Dealer	3·12	23	0	5·6	20·9	30·5	bad	27′	3°05′	...	343	350	...	469	....	....	....	....	....
English P	14b	Dealer	3·05	23	0	5·3	19·7	30·1	some bad	20′	2°30′	304	309	....	....	440	....	....	...	....	....
Belgian N	15	Dealer	2·97	30	0	7·7	15·4	19·4	very good	1°00′	2°30′	154	210	....	....	285	....	....	....	....	....
Belgian P	16	Dealer	3·03	26½	0	2·2	12·4	20·6	good	5°00′	12°00′	232	332	....	....	Fl'd	....	....	....	....	....
Belgian P	17	Dealer	3·02	26	0	3·1	12·9	20·1	bad	1°10′	5°00′	328	394	....	....	487	....	....	....	....	....
Belgian P	18	Dealer	3·09	27	0	0·3	2·8	9·4	good	1°20′	4°50′	255	360	....	....	492	....	....	....	....	....
Belgian P	19	Agent	3·08	25	0	0·9	6·2	15·8	good	2°40′	7°40′	452	536	...	......	525	...	...	....	....	....
	20	......	......	25					....	....	....	..	....	....	....	....	..	....	....	....	533
	21	......	......	25	0	4·0	10·8	...	....	....	....	....	....	....	463	....	..	....	....	....	648
	22	......	......	......					....	....	....	....	152	....	....	485	...	....	....	....	606
	23	......	......	......					....	....	....	....	345	450	....	545	....	....	....	....	593
Total..	....	......	......	......	....	....	....	....	....	....	....	....	....	....	....	....	....	....	...	....	....

No. in Table	Average Tensile Strength in lbs. per sq. in. 1 to 1. 1 wk.	2 wks	4 wks	2 mos	3 mos	4 mos	6 mos	1½ to 1. 1 month.	2 to 1. 1 wk.	4 wks	2 mos	3 mos	4 mos	6 mos	3 to 1. 1 wk.	2 wks	4 wks.	2 mos	3 mos.	4 mos.	6 mos.
1	68	....	102	125	....	....	....	....	....	....	....	....	..	....	......	....	......	....	......	......	......
2	76	....	115	158	162	137	163	132	...	....	....	....	....	....	......	....	......	....	......	......	......
3	....	....	....	....	....	....	....	....	....	....	....	..	....	....	30	...	72	....	......	......	......
4	....	....	....	....	....	....	....	....	....	....	....	....	....	....	49	....	75	....	......	......	......
5	....	....	....	....	....	....	....	......	....	....	....	....	....	....	43	....	92	....	......	......	......
6	....	....	....	....	....	....	....	....	....	....	....	....	....	....	106	....	126	....	......	......	...
6a	....	....	....	....	....	....	....	....	....	....	....	....	....	....	......	....	......	...	......	......	......
6b	....	....	....	....	....	....	....	....	....	....	....	....	....	....	......	....	......	....	......	......	......
6c	....	....	....	....	....	..	....	....	....	....	....	....	....	....	......	....	......	..	......	......	......
7	..	....	....	....	....	....	....	....	....	....	....	....	....	....	......	....	....	....	......	......	......
8	....	....	....	....	....	....	....	....	....	....	....	......	....	....	55	....	136	....	......	......	......
9	....	....	....	....	....	....	....	....	....	....	....	....	....	....	95	....	115	....	..	......	......
10	232	345	316	....	....	....	....	....	....	....	....	....	....	....	72	....	96	....	......	......	......
11	192	....	204	245	253	257	531	....	151	198	189	196	189	271	......	....	......	104	......	..	....
12	....	....	....	....	....	....	....	....	....	....	....	..	....	....	......	....	......	....	......	......	......
13	....	....	....	..	....	....	....	....	....	....	....	....	....	....	......	....	......	....	......	......	......
14	....	....	....	....	...	....	..	....	....	....	....	....	....	....	60	79	102	....	......	...	......
14a	....	....	....	....	...	....	....	....	....	....	....	....	....	....	54	64	90	....	......	.. ..	......
14b	..	....	....	....	....	....	....	....	....	...	....	..	....	....	34	....	......	....	......	......	......
15	134	....	195	....	....	....	....	....	48	77	....	....	....	....	......	....	......	....	......	......	......
16	....	....	....	....	......		....	....	....	....	....	....	....	....	......	....	......	....	......	......	......
17	....	....	....	....	....	....	....	....	....	....	....	....	....	....	......	....	......	....	..	......	......
18	....	....	....	....	....	....	....	....	126	207	....	....	......	....	......	....	..	....	......	......	......
19	....	....	....	....	....	....	....	....	....	....	....	..	....	....	133	....	154	....	......	......	......
20	....	....	....	....	....	....	....	....	....	....	....	....	....	....	......	....	......	....	......	......	......
21	....	....	....	....	....	....	....	....	....	....	....	....	....	....	....	....	....	....	......	......	......
22	....	....	....	....	....	....	....	....	....	....	....	....	....	....	......	......	......	....	......	......	......
23	....	....	...	....	....	....	....	......	....	....	....	....	...	....	......	....	......	....	......	......	......
Total	..	....	....	....	....	....	...	......	....	....	....	....	....	....	......	....	......	....	.. ..	......	......

No. in Table	Average Compressive Strength lbs. per sq. in. Neat Cement 1 wk.	2 wks	3 wks	4 wks	2 mos	1 year	1 to 1. 1 wk.	4 wks	2 mos	2 to 1. 1 wk.	4 wks	2 mos	3 to 1. 1 wk.	wks.	Transverse Strength. Neat 1″x1″ broken on 6″ centres. 1 wk.	2 weeks	3 weeks	4 weeks	1 year.	No. of Tests. Tensile.	Comp.	Trans.
1	....	....	...	....	1600	....	....	....	....	....	....	....	....	......	....	......	......	......	......	27	1	0
2	1285	....	..	2255	....	....	900	1350	....	....	....	....	....	......	....	......	......	......	......	100	8	0
3	....	....	..	....	...	....	....	...	....	....	....	....	205	475	....	......	......	......	......	33	3	0
4	....	....	....	....	....	....	....	....	....	....	....	....	392	412($\frac{2}{1}$)	....	......	......	......	......	33	4	0
5	1362	....	....	2492	..	....	....	....	....	....	....	....	....	......	42	......	......	......	......	30	4	1
6	3025	....	....	....	4975	....	....	....	...	....	....	....	....	1025	99	......	......	......	......	35	7	2
6a	..	....	....	....	....	....	....	....	....	....	....	....	....	......	....	......	......	......	......	15	0	0
6b	....	....	....	....	....	....	....	....	....	....	....	....	....	......	....	......	......	......	......	6	0	0
6c	....	....	...	....	....			....	....	....	....	....	....	......	....	......	......	......	......	10	0	0
7	1502	....	....	1720	....	....	....	....	...	....	....	....	....	......	54	......	......	84	......	10	5	2
8	1967	....	....	2650	....	....	....	....	....	....	....	....	....	......	73	......	......	99	......	30	2	5
9	....	....	....	....	4500	....	....	....	....	....	...	....	....	......	....	......	......	......	......	40	1	0
10	....	....	....	....	....	....	....	3325	....	....	....	....	....	......	....	......	......	......	......	29	1	0
11	....	....	....	....	..	....	....	2826	3200	....	....	800	....	......	....	......	......	112	......	105	9	2
12	....	....	....	....	....	....	....	....	....	....	....	....	..	......	....	......	......	......	......	44	0	0
13		....	....	....	....	....	....	....	....	....	....	....	....	......	....	......	......	......	......	14	0	0
14	...	..	....	....	....	....	....	...	....	....	....	....	....	......	....	......	......	.. ..	......	30	0	0
14a	....	....	....	....	....	....	....	....	....	....	....	....	....	......	....	....	......	......	....	30	0	0
14b	...	....	....	....	....	....	....	....	....	....	....	....	....	......	....	......	......	.....	....	93	0	0
15	1250	....	....	1950	....	....	....	....	....	743	953	....	...	......	..	......	......	......	......	35	8	0
16	....	....	....	....	....	....	....	....	....	....	....	....	....	......	....	......	......	......	......	10	0	0
17	...	....	....	....	....	....	....	....	....	..	....	..	....	......	....	......	......	......	......	13	0	0
18	....	....	....	....	....	....	....	....	....	....	....	....	....	......	....	....	......	......	......	50	0	0
19	2970	....	. .	4350	....	....	....	....	....	....	....	....	718	900	.. ..	......	......	......	......	24	6	0
20	....	....	..	....	....	6391	....	....	....	....	....	....	...	......	....	......	......	......	113	12	10	2
21	...	....	5310	....	......	4826	....	....	....	....	....	....	....	......	....	......	86	......	......	17	3	2
22	1200	1555	....	2566	....	5066	....	....	....	....	....		....	......	....	......	......	59	......	13	9	2
23	....	3000	....	4400	....	....	....	....	....	....	....	....	....	......	....	......	......	......	60($\frac{2}{1}$)	6	2	1
Total	....	....	....	....	....	....	....	....	....	....	....	....	....	......	....	......	....	.. ...	....	894	83	19

The sand briquettes were lightly tamped with a small iron rammer.—C. B. S.

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grinding, and will soon fall into line; there is no reason why educating influences should not bring grinding down much finer still for ordinary brands, but for the present, too much severity would defeat the object in view. (For tests on the effect of fine grinding, see Series I of Experiments.)

(*d*) The time of incipient and final set, as found by Gilmore's needles, does not seem to affect the strength, except for very short tests. When the slow settings are generally stronger, good cements may be either the one or the other; but ordinarily, unless for tidal work, a slow setting one has the desirable feature of allowing masons to mix and use good sized batches of mortar, without constant tempering, which is the practice with quick setting ones, much to their own hurt.

(*e*) The blowing test advised by Faija, has detected a "blowey" tendency in several instances; but much late evidence seem to throw some discredit on blowing tests, whether made with hot or boiling water, on the ground that manufacturers can, by the addition of sulphate of lime, cause the cement to be so slow setting and set so strongly as to resist the blowing tendency of so much as 3 per cent. of free lime added after the cement had been burnt. If this is a fact, chemical analysis will need to be resorted to more frequently, to detect this dangerous adulteration which is fatal in sea-water and bad in any case, as the great strength which it gives to cements at early dates is apt to decrease at longer periods. Belgian No. 19 cement tested gave higher results at 1 week than at 4 weeks; this looks a little suspicious.

Cements have been tested usually neat; the Germans have reached the stage of 3 to 1 mixtures as the deciding test, and this would seem to be the only rational way of testing a cement, *i.e.*, in the same condition as it is used.

The difficulty, however—and it is a very serious one—has been to get anything like uniform results in sand tests. The variation in putting the mortar in the moulds has been so much more than the variation in the cementing value of the cement that the tests were valueless, so that most testers have clung to neat tests as being simple and a fair index of cementing qualities. That this view is in fault, and misleading, every tester will admit, and it is only partly avoiding the difficulty to specify a certain fineness, strength and specific gravity in combination, and even then the results are not definite, as each cement is different in value. However, for those who have facilities for testing cement neat only,—and these will probably be in the majority for some time to come—it would seem that 350 lbs. at 1 week neat and 450 lbs. at 4 weeks neat are easily obtained, and quite enough to specify. 11 brands tested would give this much strengh and stand the blowing test, and of these there are 6 brands fine enough for 10 p.c. residue on 80 sieve and 20 p.c. residue on 120 sieve, with a specific gravity varying from 308 to 313, while the six brands which are not strong enough are also too coarse.

The tests on natural cements are not extensive enough to form a good basis, but it would seem easy to get 100 lbs. neat at 1 week and 200 lbs. neat at 4 weeks, and a fineness the same as for Portlands.

The tests on No. 2 natural and No. 11 Portland were carried on for 6 months, and show the natural to be gaining on the Portland, although each has evidently nearly reached a maximum. This would seem to bear out the idea which many people yet have, that, in time, a natural cement not being so brittle will catch up to a Portland. Long time tests are very much needed on this subject.

Natural cements being underburnt (usually) have very much less combining power with sand; the 1 to 1 natural is not as strong as 2 to 1 Portland, according to tests made last year as per Table II, in which the mixtures were made with 15 p.c. of water for 1 to 1, and 12 p.c. of water for 3 to 1 mixtures, the mortars being lightly tamped into the mould with an iron rammer; the tests made this year, however, by means of a uniform pressure, give much higher results for 1 to 1 naturals, when 20 p.c. of water is used, which would seem to be nearer to the amount used in practice, making a soft plastic mortar. (See pressure tests.)

Natural cement has many uses. It is being passed aside in many quarters,—why ? because *if immersed in water* for 1 week or 4 weeks, it will give low tensile tests. That terror of the present day, the testing machine, condemns it.

Now there are many occasions where it would not be wise to use anything but the best Portlands—such as laying mortar in extreme frost, or where great immediate strength is required, or for subaqueous work generally, but, on the other hand, no one doubts the *durability* of good natural cement. Works in Europe hundreds of years old, and all the work done in the United States and Canada previous to 30 years ago, are built with such mortars, and stand as witnesses of their lasting qualities.

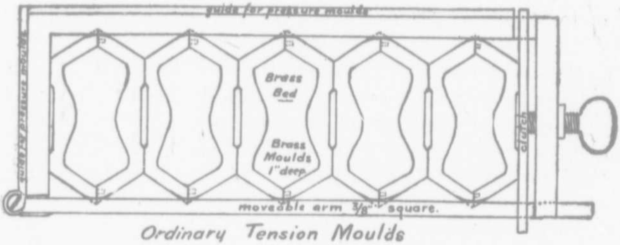
Moreover, tests made on No. 1 natural cement (see Series III frost tests) show that while it cannot be immediately exposed to extreme cold, yet when it is exposed, after it has set, it will resist frost thoroughly, and become stronger than if immersed in water at an ordinary temperature. There are thousands of situations, where natural cement mortar, 1 cement, 2 sand, will be found amply strong for the purposes required, in which case it will be found cheaper than Portland mortar, 1 cement, 3 sand. Referring ahead to Series III (frost), it will be seen that if mortars are tested in open air, the Portlands are weaker and naturals stronger than if the briquettes had been under water. This is a point of much importance, because if work is to be done which will not usually be submerged, as in damp foundations, abutments on land, culverts, etc., then tests made in open air will give results more favourable to naturals. In so many words our standard tests say: "Let us test all hydraulic cements under water; whether the mortar as used will be so or not, we will be on the safe side." This, as a generality, is doubtless best; but if we consider what a large proportion of cement is used in situations usually not submerged, it would seem more rational to test cements under conditions similar to those under which they are to be used in each case, be it in water *or air*.

As before mentioned, all the sand tests given in the Table (Table II) were made by tamping the mortar lightly into the moulds with an iron rammer weighing about ½ lb. and ½ inch square section.

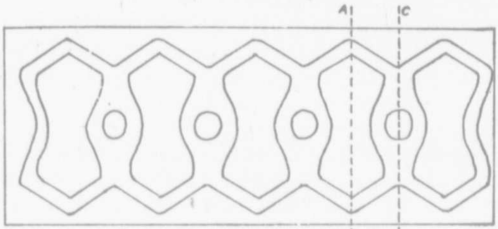
This has been done in as nearly a uniform manner as possible. About 3 layers were tamped, and then a 4th layer smoothed off with a spatula. Every effort was directed toward uniformity in method, and, doubtless, some degree of accuracy was obtained; but it was felt that the best possible would only enable comparisons to be made in this laboratory, it would not enable any to be made with results obtained elsewhere.

The Cement Committee of the Society (of which the writer was made a member, by invitation) advised that tests should be made under a pressure of 10 lbs. per sq. inch. It was not defined at the time whether this applied to sand tests only or to neat tests also; but the necessity for pressure is not so great in neat tests, because anyone with ordinary skill and practice can make a good neat briquette, and a light pressure will not affect the result much, as will be shown farther on.

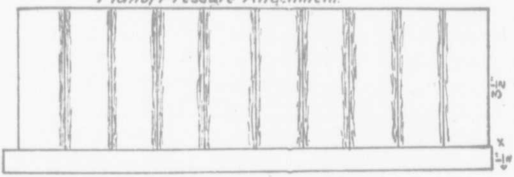
In November last the moulds for applying pressure (see drawings), which were from a design of the writer's, modified by Mr. Withycombe,



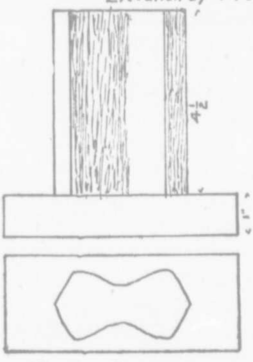
Ordinary Tension Moulds



Plan of Pressure Attachment



Elevation of Pressure Attach't



Plan and Elev of Plunger

section on C.D

were completed, and since then several hundred briquettes have been made with them. It would seem a simple matter to mix up mortar, put it under a plunger, and by putting on 10 lbs. per sq. inch, make briquettes; but theory and practice must be fellow-labourers. Now, 12 p.c. of water is considered the correct thing in 3 to 1 mixtures, but with this amount, the mortar would not pack at all in a closed mould under so light a dead pressure, and it is light dead pressure that is wanted; even 20 lbs. per sq. inch was of no greater effect, then 15 p.c. of water was tried, with very little better results.

It was finally concluded to try several series with different percentages of water, and thereby determine the best per cent. for making a good briquette.

These series (see Table III) ran from 15 p.c. to 25 p.c. of water, and were for 10 lbs. and 20 lbs. pressure per sq. in. for 1 week and 4 weeks, and each result tabulated is the average of 5 briquettes, and the whole table the result of 77 experiments, or 385 briquettes.

The result, to the author's mind, is definite, 20 p.c. of water is just sufficient to make a plastic mortar, so that a good briquette can be formed while more water tends to drown the cement and make it weaker at both the 1 week and 4 week tests, although longer tests would probably show a recovery in this respect.

This 20 p.c. applies to 1 to 1 and 3 to 1 mixtures, and will probably be about right for 2 to 1 also, if it is desired to make such tests. It is conclusive from the table that if any standard test under light pressure is to be adopted for sand tests, 20 p.c. of water must be prescribed as a definite part of the test, and in this way perfect uniformity obtained. It is understood that the sand used is standard sand dry and sharp, a finer or rounder sand would allow less water to be used. This amount of water, while greater than that usually given by authorities whose method of making sand briquettes is by some severe hammering process (*e.g.* German) is still close to the amount used in practice.

What we want, it seems, is, first of all, a uniform method capable of application in any part of the Dominion; after that we want it to approach as nearly as possible to actual usage, and fortunately the two conditions are in harmony with each other. Even at the risk of repetition, it is worth saying again, that plastic mortar made with 20 p.c. of water is close to practice, and will give regular and accurate tests if put into moulds under light pressure. The amount of this pressure does not seem to be of such great importance, but 20 lbs. per sq. inch gives

TABLE III.

TESTED IN TENSION. PRESSURE SAND TESTS.

Brand	Mixture.	% of water.	Pressure per sq. in.	1 week tests, 1 air, 6 water. lbs. per sq. in. Highest.	Lowest.	Average.	W't when tested in oz.	Weight after 2 days evapr'n.	% of evaporation.	Product col. 3 x col. 6.	4 week tests, 1 air, 27 water. lbs. per sq. in. Highest.	Lowest.	Average.	Weig't when tested in oz.	W't after 2 days evapor'n.	% of evaporation.	Product col. 3 x col. 6.
No. 2	1 to 1	15	10	45	23	**32½**	**4·56**	3·98	12·63	410·4	71	39	**59**	**4·64**	4·01	13·49	795·9
		17½	10	165	106	**136**	**5·26**	4·84	7·98	1085·3	282	205	**233½**	**5·32**	4·95	7·03	1683·7
		20	10	130	94	**117**	**5·56**	5·08	8·62	1008·6	292	239	**265½**	**5·52**	5·17	6 34	1683·2
		22½	10	123	106	**113½**	**5·54**	4·99	9·88	1124·4	258	200	**[illegible]35**	**5·49**	5 12	6·74	1583·9
No. 2	1 to 1	15	20	47	42	**43½**	**4·79**	4·05	15·52	675·1	95	70	**84**	**4·99**	4·22	15·40	1293·6
		17½	20	144	111	**126½**	**5·37**	4·92	8·38	1060·0	2[illegible]8	160	**176½**	**5·27**	4·83	8·35	1473·7
		20	20	157	90	**114**	**5·67**	5·13	9·63	1097·8	297	212	**264**	**5·62**	5·28	6·12	1615·6
		22½	20	126	110	**119**	**5·54**	5·03	9·28	1104·[illegible]	295	234	**262**	**5·56**	5·21	6·29	1648·0
No. 15	1 to 1	15	10	86	40	**62½**	**4·92**	4·42	10·46	653·7	112	98	**104**	**5·04**	4·41	12·50	1300·0
		17½	10	60	37	**52**	**5·14**	4·60	10·50	546·0	......	......	......	......	......	......	......
		20	10	149	108	**133**	**5·60**	5·12	8·46	1125·1	......	......	......	......	......	......	......
		22½	10	129	120	**125**	**5·68**	5·19	8·76	1095·1	......	......	......	......	......	......	......
No. 15	1 to 1	15	20	49	42	**45**	**4·94**	4·18	15·46	695·7	......	......	......	......	......	......	......
		17½	20	184	145	**166½**	**5·62**	5·28	6·61	1100·5	......	......	......	......	......	......	......
		20	20	146	114	**135½**	**5·63**	5·17	8·20	1111·1	......	......	......	......	......	......	......
		22½	20	130	108	**118**	**5·72**	5 25	8·22	970·0	......	......	......	......	......	......	......

Brand	Mixture.	% of water.	Pressure per sq. in.	1 week tests, 1 air, 6 water. lbs. per sq. in. Highest.	Lowest.	Average.	W't when tested in oz.	W'ght after 2 days evapor'n.	% of evaporation.	Product col. 3 x col. 6.	4 week tests, 1 air, 27 water. lbs. per sq. in. Highest.	Lowest.	Average.	Weig't when tested in oz.	W't. after 2 days evapor'n.	% of evaporation.	Product col. 3 x col. 6.
No. 15	3 to 1	15	10	20	14	**16½**	**4·75**	4·03	15.21	251·0	35	19	**28**	**4·61**	3·88	15·88	444·6
		17½	10	12	5	**7**	**4·59**	3·92	14·66	102·6	48	32	**40**	**4·66**	4·15	11·03	441·2
		20	10	13	7	**11**	**4·73**	4·17	11·79	129·7	23	5	**15**	**4·86**	4.24	12·75	191·2
No. 15	3 to 1	15	20	23	9	**16**	**4·64**	3 97	14·48	231·7	55	28	**38**	**4·56**	4·01	12·15	461·7
		17½	20	7	2	5	......	......	......	......	40	25	**33½**	**4·74**	4·23	10·80	361·8
		20	20	17	8	**12½**	**4·85**	4·28	11·75	146·9	28	19	**24**	**4·89**	4·36	10·80	259·2
No. 9	3 to 1	15	10	25	14	**19**	**4·37**	3·81	12·77	242·6	71	58	**63**	**4·54**	3·89	14·24	897·1
		17½	10	35	18	**27**	**4·49**	4·07	9·35	252·4	106	92	**96**	**4·72**	4·24	10·17	976·3
		20	10	27	20	**23½**	**4·68**	4·08	12·91	303·4	134	101	**120**	**4·65**	4·18	10·14	1218·8
		22½	10	27	22	**24½**	**4·85**	4·23	12·86	315·1	88	74	**79**	**4·70**	4·16	11·49	907·7
		25	10	11	8	**10**	**4·81**	4·13	14·13	141·3	53	33	**46½**	**4·73**	4·11	13·18	612 9
No. 9	3 to 1	15	20	37	33	**34**	**4·66**	4·05	13·22	459·5	86	62	**71½**	**4·69**	4·15	12·22	873·7
		17½	20	33	20	**27½**	**4·53**	4·10	9·54	262·3	124	103	**114½**	**4·75**	4·27	10·15	1162·1
		20	20	29	25	**26½**	**4·8**	4 19	12·78	338 7	143	109	**127**	**4·69**	4·26	9·17	1164·5
		22½	20	25	22	**23**	**4·86**	4 27	12·06	277·4	103	87	**95½**	**4·81**	4·28	11·02	1052·4
		25	20	27	22	**25**	**4·80**	4·18	12·89	324·4	53	44	**49**	**4·70**	4·09	12·94	634·1
No. 10	3 to 1	15	10	37	30	**34½**	**4·70**	4·18	11·07	381·9	59	51	**55½**	**4·72**	4·18	12·27	681·0
		17½	10	43	22	**31½**	**4·67**	4·12	11·69	368·2	87	63	**70**	**4·84**	4·35	10·05	703·5
		20	10	48	32	**37½**	**4·79**	4·24	11·41	427·8	65	62	**6 ½**	**4·89**	4·32	11·68	741·6
		22½	10	34	27	**30**	**4·95**	4·33	12·45	373·5	50	38	**44½**	**4·88**	4·22	13·48	600·0
		25	10	33	15	**23½**	**4·92**	4 27	13·14	308·7	34	23	**28½**	**4·86**	4·15	12·94	368·8
No. 10	3 to 1	15	20	41	27	**33½**	**4·68**	4·11	12·18	408·0	67	52	**61**	**4·95**	4·40	11·04	673·4
		17½	20	37	16	**27**	**4·65**	4·08	12·13	327·5	88	47	**68**	**4·84**	4·31	10·96	745·3
		20	20	42	31	**35**	**4·82**	4·24	11·96	424·5	84	56	**71**	**4·97**	4·42	11·03	783·1
		22½	20	36	23	**29½**	**4·90**	4·28	12·65	373·1	85	70	**75**	**4·90**	4·35	11·23	842·2
		25	20	33	27	**31**	**5·00**	4·35	13·06	403·0	58	34	**48**	**4·85**	4.27	11·92	572·2

rather sharper-edged briquettes, with about the same variation in uniformity and the same tensile strength per sq. inch. This is equivalent to 20 feet of masonry, which, of course, is more than practice would give; but the tests do not vary to any extent when compared with those made with 10 lbs. per sq. inch. Therefore it is not deemed of sufficient importance to sacrifice good manual results. Therefore, 20 lbs. per sq. inch pressure and 20 p.c. water was adopted about 1 month ago, and the following results obtained (Table IV); this table wil be completed in a few months, when it is intended to complete this paper by additional results on pressure, frost and pier tests.

Whether the future will bring sand tests to greater uniformity than this remains to be seen; but it is believed that, in this way, the sand combining qualities of cements can be compared with accuracy with one another, and in future such will be the method adopted in the cement laboratory at McGill, subject to the modifications of our cement committee.

It is earnestly to be desired that a code of tests be formulated at once, and all members urged to test under this code. Let all cements stand or fall under it. In the contest it is believed that Canadian cements can be as good as the best; but to do this, there must be reform on some sides, so that tests made from outputs will show a greater regularity, and cause the cement to commend itself to the consumers of the article.

COMPRESSIVE TESTS.

These are doubtless more valuable than tensile ones, in the sense that we use mortar usually in compression. There are several reasons, however, why such tests are not really needed:—

(1) Because the strong machinery needed would not be generally available;

(2) Because the compressive strength, after all, varies quite regularly with the tensile, being 5 to 6 times as great at 1 week or 4 weeks and gradually increasing to 9 to 10 times as great at a year, because by this time the cement is becoming brittle and has attained its maximum tensile strength. This is more particularly true of Portland cements, as naturals do not get so brittle;

(3) Because the compressive strength of cement mortar is so great that we need seldom concern ourselves with it, but should rather know, the adhesive and tensile strengths should they ever be called into play and, moreover, the strength of mortar in thin joints is much greater than

TABLE IV.

CONDENSED SUMMARY OF PRESSURE SAND TESTS.

Put in Moulds with 20 % water, 20 lbs. per sq. in.

Brand	Mixture.	1 week tests, 1 air, 6 water.							4 week tests, 1 air, 27 dys. water.							REMARKS.
		lbs. per sq. in.			Weight when tested.	Weight after 2 days evap'n.	% of evaporation.	Product, col. 3 x col. 6.	lbs. per sq. in.			Weight when tested.	Weight after 2 days evap'n.	% of evaporation.	Product col. 3 x col. 6.	
		Highest.	Lowest.	Average.					Highest.	Lowest.	Average.					
No. 1	1 to 1	75	46	**58**	**5·25**	4·55	13·33	773·1	102	80	**93**	**5·32**	4·70	11·73	1090·9	Temp. of air, 60° F.
No. 2	1 to 1	157	90	**114**	**5 67**	5·13	9·63	1097·8	297	212	**264**	**5·62**	5·28	6·12	1615·6	" " 60° F.
No. 15	1 to 1	146	114	**135½**	**5·63**	5·17	8·20	1111·1	......	......	......	....	....	......	......	" " 61° F.
No. 15	3 to 1	17	8	**12½**	**4·85**	4·28	11·75	146·9	28	19	**24**	**4·89**	4·36	10·80	259·2	" " { 60° F. (1) 69° F. (2)
No. 3	3 to 1	19	8	**13**	**4·74**	4·17	12·06	156·8	52	37	**47**	**4 48**	3·89	13·20	620·0	" " 63° F.
No. 9	3 to 1	29	25	**26½**	**4·80**	4·19	12·78	338·7	143	109	**127**	**4·69**	4·26	9·17	1164·5	" " { 65° F. (1) 58° F. (2)
No. 10	3 to 1	42	31	**35**	**4·82**	4·24	11·96	424·5	84	56	**71**	**4·97**	4·42	11·03	783·2	" " { 68° F. (1) 59° F. (2)
No. 8	3 to 1	34	25	**30½**	....	....	......	......	85	75	**80**	**4·99**	4·41	11·55	924·0	" " 54° F.
No. 5	3 to 1	15	12	**14**	**4·78**	4·12	13·70	191·8	58	43	**50**	**5·13**	4·36	15·01	750·0	" " 74° F.
No. 4	3 to 1	52	30	**39½**	**4·94**	4·37	11·58	457·4	118	83	**103**	**5·02**	4·49	10·56	1087·7	" " 61° F.
No. 19	3 to 1	77	58	**69½**	**4·79**	4·09	14·61	1015·3	143	101	**129**	**4·88**	....	......	......	" " 65° F.
No. 6	3 to 1	83	74	**78**	**4·77**	3·97	16·84	1313·5	139	118	**128**	**4·90**	4·28	12·65	1619·2	" " 64° F.
No. 11	3 to 1	25	15	**19**	**4·56**	4·13	9·51	180·7	46	37	**41½**	**4·85**	4·18	13·90	576·8	" " 48° F.
No. 14A	3 to 1	15	8	**10½**	**4·69**	....	......	......	36	24	**30**	**4·88**	4·16	14·76	442·8	" " 53° F.

in cubes. Tests on cubes always go higher for small cubes than for large ones. (See also Series (IV*a*) tests of mortar joints in brick piers.)

TRANSVERSE TESTS

Have often been advocated, and the machinery needed may be quite simple; but there are two objections which would preclude there being any great value in such tests :—

(1) Because the co-efficients of rupture in transverse testing are known to be at fault in not really indicating the tensile strength of the outer layer or fibre ; this could possibly be avoided by determining certain corrections as a thesis paper to the *Engineering News* pointed out ;

(2) The main objection is that a flaw of a very slight amount may be objectionable in such tests if situated near the tension face. Any cement tester knows that bubbles will occur. They may be very minute, or if of any size may be deducted in tensile tests, while in transverse tests, who could determine the correction to be made ? Also tests made show that if tested upside down from position moulded, the results are higher than when tested as moulded. Altogether, this method of testing does not seem to commend itself to general use.

To conclude the subject of ordinary testing for commercial purposes, and with the addition of chemical analysis where available for scientific ones also, the following seems to be a good basis to work on, that 4 tests should be made in combination :—

(1) Specific gravity 3.10 for Portlands, 2.95 for Naturals.

(2) Blowing test. In the absence of really final knowledge on the subject to continue to specify pats in steam at 115°F. for four hours, in water at 115°F. for twenty hours, at which time if the pats are stuck tight to the ground glass, the cement may be considered safe, while if it has loosened from the plate but has not yet cracked or warped, it may be immersed again for 24 hours at 115°F., or else placed in water of ordinary temperature for 4 weeks, after which, if no further signs have developed the cement may be considered safe.

(3) Fineness :—

10 p.c. residue on No. 80 sieve } as maximum.
and 20 p.c. " " " 120 " }

(4) Tensile strength :—

		Portland.	Naturals
Minimum neat	3 days	250	75
" "	1 week	350	100
" "	4 weeks	450	200

1 to 1 and 3 to 1 sand tests with 20 p.c. water, and 20 lbs. per sq. inch pressure to be determined by tests made and results furnished within the next year.

SERIES I.

SPECIAL TESTS.

On the effect of fine grinding :—

(*a*) 2 oz. cement passing No. 120 sieve.........Cement

2 oz. " caught on No. 120 sieve }
2 oz. " " " No. 80 sieve } ...... Sand
2 oz. sand }

tested at 4 weeks gave 165 lbs., while

2 oz. cement passing No. 120 Sieve.........Cement
6 oz. sand.. Sand

gave 121 lbs. tested at the same age.

Thus, if in the first instance we consider all but the finest as sand, then our result is only 35 per cent. higher than the 2nd mixture, showing of how little value the coarser particles were.

(*b*) No. 8 English Portland (very coarse) gave in ordinary test 414 lbs. 1 week neat, 528 lbs. 4 weeks neat ; but when all the particles caught on No. 80 sieve were rejected, the results were 393 lbs. in 1 week, 484 lbs. in 4 weeks, demonstrating the well-known fact that neat tests of Portlands operate against fine grinding, and therefore should be considered only in connection with fineness and specific gravity.

(*c*) Equal portions (same brand) of residues on No. 50 and No. 80 sieve were mixed with $22\frac{1}{2}$ per cent. water, and gave 262 lbs. in 1 week and 324 lbs. in 4 weeks, which is very surprising, and can only be accounted for on the ground that the dust of cement clinging on to the coarse particles was sufficient to hold them together, or else that the mechanical action of mixing the mortar broke up many coarse particles into finer ones.

(*d*) To show the superior value of fine cement in sand mixtures, the following results have been obtained :—

	1 to 1.		2 to 1.		3 to 1.	
	Ordinary.	Fine on 120 Sieve.	Ordinary.	Fine on 120 Sieve.	Ordinary.	Fine on 120 Sieve
No. 2 Natural 1 week 20 % water 20 lbs. pressure.	114	190	....	....	....	....
No. 2 " 4 " 15 % " tamped........	98	65	....	....	....	....
No. 2 " 4 " 15 % " "	145	123	....	....	....	....
No. 15 " 1 " 20 % " 20 lbs. pressure.	166	229	....	....	....	....
No. 15 " 4 " 14 % " tamped........	....	....	77	125	....	....
Brand A " 4 " 20 % " "	31	39	....	....	....	....
No. 3 Portl'd 4 " 12 % " "	....	....	....	....	72	121
No. 3 " 4 " 20 % " 20 lbs. pressure.	....	....	....	....	47	100
No. 9 " 4 " 20 % " 20 " "	....	....	....	....	49	109
No. 5 " 4 " 12 % " tamped........	....	....	....	....	82	102
No. 6 " 4 " 12 % " "	....	....	....	....	126	188

These results should be a convincing argument to users of Portland cement, that fine grinding is worth paying for, because the finer the same cement the greater its sand-carrying value is.

The only partial exception in the above results is No. 2 natural. This is either erratic, being, however, duplicated, or if not, is easily accounted for. An underburnt cement is easily ground, and therefore is not apt to be *well* ground; very easy grinding will make it fine enough, and the better burnt particles being a little *better* burnt are, therefore, harder and escape grinding; but these particles, not being very hard, are probably bruised up in mixing, and form the best part of the cementing substance; therefore, when these are sifted out, the underburnt fine particle has not as great a cementing value as the mixture would have unsifted. On the other hand, the coarse particles in Portland cement are much harder, and are always a detriment in a sand mixture.

SERIES II.

HOT WATER TESTS.

(*a*) No. 1 Natural cement neat, 2 months old, gave when tested the following results:—

(1) Water at temperature 52°F., 226 lbs. average.
(2) " " " 122°F., 250 lbs. average.

(*b*) No. 1 Natural cement 1 to 1, 2 months old, gave when tested the following results:—

(1) Water at temperature 47°F., 125 lbs. average.
(2) " " " 118°F., 129 lbs. average.

(*c*) No. 4 Portland, neat, 1 month old, gave when tested the following results:—

(1) Water at temperature 65°F., 533 lbs. average.
(2) " " " 118°F., 616 lbs. average.
(3) " " " 186°F., 556 lbs. average.

(*d*) No. 4 Portland, 3 to 1, 1 month old, gave when tested the foling results:—

(1) Water at temperature 66°F., 81 lbs. average.
(2) " " " 183°F., 81 lbs. average.

These tests, which are very uniform, indicate that for either natural or Portland cements tested neat or with sand, there is a slight gain in strength, by using hot water in mixing.

The *advantage* being that for exposure to frost the cement will set quicker and resist the frost action better. By referring ahead to frost tests, it will be seen that cements exposed at about same temperature (natural cement only tested with hot water in frost) gave much higher results when mixed with hot water, being in ratio, 94 to 0 for neat cement No. 1 Natural, and 117 to 44 for 1 to 1 cement No. 1 Natural.

SERIES III.

FROST OR EXPOSURE TESTS.

This series consisted of various investigations into the strength of mortars when mixed with different conditions of water and under different exposures, reference being particularly made to frost. All tests were made in quadruplicate.

The 1st set was submerged, after 24 hours, in water of laboratory tanks;

The 2nd set was kept on damp boards in a closed tank for the whole period, and never allowed to dry out;

The 3rd set was allowed to set in the laboratory, and then exposed to the severe frost and left in open air for the whole period;

The 4th set were exposed in from 8 to 10 minutes to the severe frost, and left there for the whole period, except to take them out of the moulds when they were set or frozen.

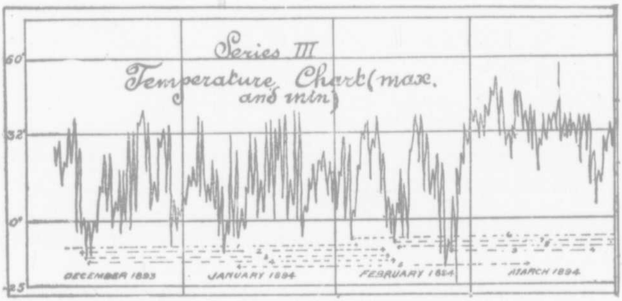


Table V is here given, showing the results obtained, and accompanying it is a temperature chart showing the weather to which these mixtures were exposed during their whole period.

It will be noticed that these tests were purposely made in cold snaps so as to make the tests as severe as possible.

It would appear improbable that mortar immediately exposed to severe frost would become stronger than that allowed to set in a warm atmosphere, but the results of all the Portland cement tests, both in tension and compression (with one exception) assert it; and also that those allowed to set in the laboratory, and then exposed continually, are the weakest of all the 4 conditions treated of. This would go far to dispute the advisability of covering up mortar laid in frosty weather.

TABLE V.

FROST OR EXPOSURE TESTS.

SERIES III.

Mixture.	Age.	Tensile Strength. Water test. (1)	Tensile Strength. Damp air test. (2)	Tensile Strength. Exposure after sett'g. (3)	Tensile Strength. Exp're before sett'g. (4)	Compressive Strength. 1	Compressive Strength. 2	Compressive Strength. 3	Compressive Strength. 4	Dates of Exposure.	Temp. of Exposure for 3.	Temp. of Exposure for 4.	Time from mixing till exposure.	Natural time of set.	No. of tests.	Remarks.
No. 11. Portland Neat.	2 mos.	602	471	282	334					Dec. 6th to Feb. 6th.	+23°F.	+22°F.	30′ (3) 12′ (4)	25′	16	
1 to 1.	"	377	276	194	233	3200	1780	1600	1900	Dec. 11th to Feb. 11th.	+5.°F.	$+3\frac{1}{2}$°F.	40′ (3) 8′ (4)	35′	20	
2 to 1.	"	168	150	105	111	800	720	660	440	Dec. 12th to Feb. 12th.	$-\frac{1}{2}$°F.	0°F.	40′ (3) 10′ (4)	37′	24	
3 to 1.	"	104	86	92	97	300	520	230	300	Dec. 13th to Feb. 13th.	−5°F.	−6°F.	1°27′(3) 10′ (4)	1° 25′	24	Nos. 3 and 4 showed irregular and injured fractures.
No. 1. Natural Neat.	"	226	221	349	0	1600	1500	2300	1390	Jan. 12th to Mar. 12th.	+2° F.	+5° F.	4° 15′ (3) 11′ (4)	4° 15′	24	No. 4 tension completely blown in fragments.
1 to 1.	"	125	229	187	44			0	800	Feb. 5th to April 5th.	+8° F.	$+\frac{6}{10}$°F.	8° 0′ (3) 10′ (4)	8° 00′	22	Some of No. 4 tension injured and No. 3 compression.
Neat.	"	250	281	159	94	2800	2000	3300	1300	Feb. 13th to April 13th.	+13° F.	+ 5° F.	6° 0′ (3) 10′ (4)	6° 0′	24	Mixed with water at temp. 122° F.
1 to 1.	"	129	170	80	·117					Feb. 14th to April 14th.	+ 9° F.	0° F.	3° 0′ (3) 8′ (4)	2° 50′	20	Mixed with water at temp. 118° F.
Neat.	1 m	155	278	217	249					Feb. 26th to Mar. 26th.	+17°F.	$+7\frac{1}{2}$°F.	7° 0′ (3) 9′ (4)	7° 0′	20	Mixed with 2 % brine.

The next deduction from the Portland cement tests is that laboratory tests made with briquettes submerged give higher results than can be expected in open air work, and therefore that engineers should add this to the various other degenerating contingencies, such as bad mixing, dirty sand, etc. A deduction not much evidenced in the Table is that it is not safe to lay Portland cement mortar below 0° F. because the 3rd and 4th series of 3 to 1 Portland exposed at —6° F. gave ocular evidence that their structure was injured, and the test-pieces broke most irregularly, while the other exposures at about 0° F. gave no evidence of any injury at all. Coming to the natural cement mortar in the 5th and 6th lines, we find much different results. The first one is decisive, and is that this particular cement mortar cannot be laid in zero weather. The first set were all blown to pieces (except the cube), which surprisingly stood 1390 lbs., while the 2nd set, although not quite blown to pieces, all showed extreme injury.

The most peculiar result is that this same cement, neat, if given a few hours to set in the temperate air, will on exposure to the frost attain a strength highest of the 4 conditions; this is quite remarkable, that while the Portland cement was strongest when submerged, the natural cement was stronger in damp air and strongest in frost.

Indeed, the Portland cement, in air, for 1 to 1 mixtures, was very little stronger than the 1 to 1 natural.

All of the natural cement specimens exposed to frost showed a disintegrated layer on the outside about $\frac{1}{8}''$ thick; underneath this the structure was quite sound, and doubtless much of the variations in tests is due not so much to a weakening through the whole mass as to a reduced sectional area.

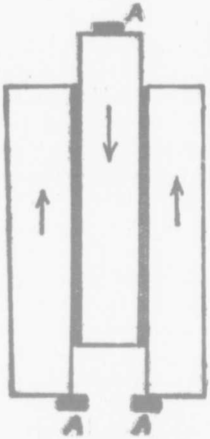
The last series made with 2 per cent. brine in mild weather for 1 month (exposed at $+7\frac{1}{2}°$ F) showed that salt increased the strength, making them as strong as others were at 2 months when mixed with fresh water, and also again emphasised the advantage to this natural cement of open air tests.

It would seem that either hot water or salt are therefore very strengthening in their effect. Much additional data on this subject is hoped for in Part II of this paper.

SERIES IV.

SHEARING TESTS.

This series of experiments was carried out with a view of obtaining more information on the shearing strength of mortar. The method adopted was as follows:—



Three bricks placed, as shown in sketch, were cemented together, and tested at the end of one month. It was found that by placing pieces of soft wood at *A.A.A.*, an action as nearly as possible a shear was obtained, and gave very satisfactory results, the pressure being practically concentrated along the two mortar joints. No side pressure was applied, because the desire was to obtain minimum results where friction was not assisting.

The combined effect of adhesion and friction can easily be computed if the adhesion and super-imposed load are known.

The results are divided into lime mortar, natural cement mortar and Portland cement mortar, also into $\frac{1}{4}''$ and $\frac{1}{2}''$ joints, also into flat common unkeyed bricks and pressed Laprairie brick keyed on one side. (1) The lime mortar was mixed 1 lime to 3 of standard quartz sand, by weight; (2) natural cement mortar was mixed, 1 of No. 2 natural cement to $1\frac{1}{2}$ standard sand; (3) Portland cement mortar was mixed, 1 of No. 5 Portland cement to 3 standard sand. (See exhibits of bricks with mortar attached.) The test-pieces were chiefly allowed to stand in the laboratory at a temperature of 55° to 65° F., but one set of natural cement mortar and two of Portland cement mortar were duplicated by immersing in water for 29 days, after setting in air 24 hours before submersion.

These results point out many interesting facts: (*a*) the first fact noticeable is that the results are independent of the *thickness* of joint; this is true of lime and cement mortars. (*b*) The next one is not evidenced to any extent in the table, but was quite apparent in the testing, viz., that the adhesion of the mortar to the brick was greatest when the mortar was put on very soft, and least when the mortar was dry. This will largely uphold the use of soft mortars by masons, albeit their reason is a purely selfish one, the mortar being easy to handle. The tensile tests of cements made *very* soft are lower than when the mixture has the minimum amount of water for standard consistency.

But for adhesive tests the case is evidently the reverse. It may be here mentioned that in these tests all bricks were thoroughly soaked with water before the joints were laid. (*c*) Coming now to the tests on lime mortar, the shears were through the mortar, except in the 4th experiment, and therefore they are quite independent of the key of the pressed brick on the surface of adhesion. This would point out the fact that keyed brick are superfluous in lime mortar joints, and the shearing strength per sq. inch averages about $10\frac{1}{2}$ lbs. per sq. inch. The tensile strength of the same mixture at the same age was 30 lbs. per sq. inch, and the compressive strength 102 lbs. per sq. inch. (*d*) The natural cement mortar showed distinctly that its adhesive strength was not as great as its shearing strength, which is the reverse of the lime mortar tests. It also showed that the keyed brick aided in some unknown way, for the results on them are 3 times as great as with the common flat brick. Of course this may have been, and probably was partly due to the different surface of adhesion. In 5 tests out of 21 made on the natural cement mortar, the mortar sheared through, and the average of these 5 was 97 lbs. per sq. inch, which gives the shearing strength proper, while the average adhesive strength of the 13 tests in air which came loose from the bricks was 26 lbs. per sq. inch in common brick, 48 lbs. per sq. inch on Laprairie pressed brick, and 38 lbs. per sq. inch on Laprairie pressed brick for three tests submerged in water for the whole period.

This would show that the adhesive strength is nearly twice as great on pressed brick as common brick, and that submersion in water had a rather harmful effect than otherwise on the adhesive strength, and was certainly of no benefit.

The tensile strength of the same mortar at the same age was 132 lbs. per sq. inch; the compressive strength was not obtained, but would have been about 1000 lbs. per sq. inch. The hints to be taken from these tests are that pressed brick keyed on both sides will give much higher results than flat common bricks, and would probably place the shearing strength of such joints at 100 lbs. per sq. in., and make it largely independent of the consistency of the mortar. Also that the shearing strength is very much higher in proportion to the tensile strength than was the lime mortar shearing strength to its tensile strength, but about the same proportion to its compressive strength, *i.e.*, 10 to 1.

It becoming evident that the thickness of joint had no appreciable effect, the Portland cement mortar tests were made all $\frac{1}{4}''$ thick. The results are surprisingly low. The adhesion on the common brick is about the same for air drying or submersion in water, and is slightly less than ½ that of natural cement mortar tests of 1½ to 1. This is a significant fact, for while a neat tensile test of No. 2 natural cement 4 weeks old is 268 lbs., the No. 5 Portland is 459 lbs. for the same age, and a 3 to 1 No. 5 Portland is 82 lbs. for same age. (See table of general laboratory results.) Thus while any test of this cement would show that a 3 to 1 mixture of the latter would be nearly equal to a 1½ to 1 test on the former, yet in their adhesive properties to common brick the heavily dosed sand mixture was only half as strong as the natural cement mortar with a smaller dose of sand. We might easily have expected this; but the main point is: is it taken account of, in considering the comparative values of these mixtures, that the adhesive strength of a Portland cement mortar heavily dosed with sand is low as compared with a weaker but richer mixture of natural cement mortar? The shearing of Portland mortar shows that the adhesion to pressed brick is greater than to common brick, but not in such proportion as in natural cements, being 1½ or 2 to 1 in place of 3 to 1 in the latter. But here again comes out the advantage given to Portland cements by testing them under water; the submerged specimens are stronger than open air ones, while in natural cements the reverse is the case.

Table VI is given on next page summarising the results obtained.

SERIES IV. (*A*)

THE STRENGTH OF MORTAR IN COMPRESSION IN BRICK MASONRY.

All engineers realise that the strength of mortar is much less, tested in cubes than in thin layers, but just what proportion they bear to one another is not very well known. The following experiments have been made with a view of obtaining this information. (See table VII, p. 263).

At the same time that these tests were made, mortar was also made into test-pieces, and tested at the same age. We are thus enabled to form an idea of the relative strengths of mortar in thin joints and in cubes, and also to form an intelligent opinion of the comparative strengths of lime mortar, natural cement mortar and Portland cement mortar. The mortars of the 4th, 5th and 6th tests are identical with the mortars of the *shearing* tests, and show the same clear superiority of the natural cement 1½ to 1 over the Portland cement 3 to 1 when used in this manner. Table VIII, p. 265, summarises the results obtained.

TABLE VI.

TABLE OF SHEARING TESTS, OR MORTAR ADHESION TO BRICK SURFACES (in shear.)

SERIES IV.

Kind of Mortar.		Joint.	Brick.	No. of tests.	How indurated.	Shear in lbs. per sq. in.			Remarks.
						Average.	Least.	Greatest	
Lime 1.	Sand 3.	1/4"	A	5	in air	9·7	8·4	11·9	All sheared through the mortar.
"	"	1/2"	A	4	"	12·1	6·1	19·8	" " "
"	"	1/4"	B	5	"	12·0	9·1	15·5	" " "
"	"	1/2"	B	5	"	8·0	5·5	11·0	All came away from brick (mortar dry).
No. 2. Natural Cement	1 Sand 1½	1/4"	A	5	"	22·3	8·0	32·1	" " "
"	"	1/2"	A	5	"	29·0	24·0	33·0	" " "
"	"	1/4"	B	5	"	75·0	25·0	118·0	2 came away from brick, 3 sheared.
"	"	1/2"	B	3	"	85·0	43·0	118·0	1 " " " 2 "
"	"	1/2"	B	3	in water	38·0	34·0	42·0	All came away from brick.
No. 5. Portl'd Cement	1 Sand 3	1/4"	A	3	in air	10·6	10·2	11·6	" " " The brick which was on top in the original laying, always sheared 1st and at a less load than that of lower one, which, of course, set under twice as much load or pressure.
"	"	1/4"	A	5	in water	13·0	10·2	16·4	" " "
"	"	1/4"	B	3	in air	16·5	9·2	24·2	" " "
"	"	1/4"	B	3	in water	27·1	20·2	36·9	" " "

A. common, flat, unkeyed, salmon brick.
B. Laprairie pressed brick, key on one side.

TABLE VII.

MORTAR JOINTS IN COMMON BUILDING-BRICK PIERS.

Composition of Mortar.	Age of Test.	Thickness of Joints.	Dimensions of Brick Pier.	% of water in mortar.	Loads in lbs per sq. inch.				Compression per foot under a total load of		
					1st signs of failure in mortar.	1st signs of failure in brick.	Bricks failing rapidly.	Maximum load	5,000	20,000	35,000
No. 1. 1 Lime. 5 Building sand.	1 week.	$\frac{3}{16}''$	7.80" x 7.85". 16.57" high. 6 bricks. 61.2 sq. in. area.	37 ($\frac{2}{1}$)	245	327	980	1,143	.015"	.08"	.13"
No. 2. 1 Lime. 5 Building sand.	3 weeks.	$\frac{3}{16}''$	8.0" x 8.0". 11.16" high. 4 bricks. 64.0 sq. inches.	37	469	563	1,406	1,553	.007"	.043"	.075"
No. 3. 1 Lime. 5 Building sand.	3 weeks.	$\frac{3}{16}''$	7.9" x 7.9". 24.50" high. 9 bricks. 62.4 sq. inches.	37	400	689	897	1,282	.005"	.053"	.094"
No. 4. 1 Lime. 3 Lab'tory sand.	1 week.	$\frac{1}{4}''$	7.75" x 7.85". 11.42" high. 4 bricks. 60.84 sq. inches.	34	287	575	....	1,117	.032"	.133"	.158"
No 5. 1 of No. 2 Natural cement. 1½ Lab'tory sand	1 week.	$\frac{1}{5}''$	7.80" x 7.90". 11.15" high. 4 bricks. 62.01 sq. inches.	22½	968	1,190	1,403	1,984	009"	.027"	.054"
No. 6. 1 of No. 5 Portland cement. 3 Lab'tory sand.	1 week.	$\frac{1}{4}''$	8.00 x 7.95". 1130". high 4 bricks. 63.60 sq. inches area.	20	755	959	1,305	1,564	.007"	.007"	.019"

CONTINUATION OF TABLE VII.

MORTAR JOINTS IN BRICK PIERS.

Composition of Mortar and Piers.	Age of Test.	Thickness of Joints.	Dimensions of Brick Pier.	% of water in mortar.	Load in lbs. per sq. inch.				Compression per foot under a total load of		
					1st signs of failure in mortar.	1st signs of failure in bricks.	Bricks failing rapidly.	Maximum load	5,000	20,000	35,000
No. 7. 1 No. 5 Portland. 1½ Laboratory sand. Common bldg. bricks.	1 week.	¼"	8.00" x 8.00". 11.5" high. 4 bricks. 64.0 sq. in. area.	20	1125	1563	....	1734	.000	.0045	.011
No. 8. 1 No. 11 Portland. 1 Laboratory sand. Laprairie pressed brick.	12 days.	¼"	8.3" x 8.3". 11.8" high. 4 bricks. 68.9 sq. in. area.	....	1679	1800	1930	1960	.001	.006	.011
No. 9. 1 Lime. 3 Laboratory sand. Laprairie pressed brick	4 weeks.	¼"	8.2" x 8.2". 11.5" high. 4 bricks. 67.2 sq. in. area.	35	260	853	....	1263	.048	.115	.156
No. 10. 1 No. 2 Natural. 1½ Laboratory sand. Laprairie pressed brick.	4 weeks.	¼"	8.4" x 8.4". 11.0" high. 4 bricks. 70.6 sq. in. area.	22½	1345	1629	1746	1983	.000	.0027	.005
No. 11. 1 No. 5 Portland. 3 Laboratory sand. Laprairie pressed brick.	4 weeks.	¼"	8.4" x 8.4". 11.1" high. 4 bricks. 70.6 sq. in. area.	20	1204	1600	1629	1785	.002	.011	.016

NOTE:—These results were obtained after the publication of the paper, and are the additional pier tests promised in the text.

TABLE VIII.

	Strength of Mortar per sq. in.			Loads released at 17,500 lbs., set observed per lineal foot.	
	In joints.	In cubes.	In tens'n.		
(1)	245	40	17	....	1 week old, mortar, 1 lime, 5 sand.
(2)	469	57	20	.01″	3 " " " 1 " 5 "
(3)	400	57	20	.03″	3 " " " 1 " 5 "
(4)	287	21	....	.08″	1 " " " 1 " 3 "
(5)	968	250	....	....	1 " " " 1 Natural Cement 1½ sand.
(6)	755	341	43	.00	1 " " " 1 Portland " 3 "

Roughly speaking, the lime mortar at 1 week 5 to 1 is 6 times as strong; the lime mortar at 1 week 3 to 1 is 14 times as strong; the natural cement mortar at 1 week 1½ to 1 is 4 times as strong; the Portland cement mortar at 1 week 3 to 1 is twice as strong, as the same mortar tested in cubes, at the same age.

Referring to the amount of compression in Table VII, it will be seen that the amount of compression per foot is much less according as this ratio is less—*i.e.*, the less yielding the mortar, the nearer does the strength in cubes approach to the strength in joints. This is to be expected, because the more yielding substances will be at a much greater disadvantage when unsupported at the sides than if enclosed in a thin masonry joint.

In the 2nd, 3rd, 4th and 6th tests at 17,500 lbs., the load was released, and the permanent set observed was as given in the 5th column of the preceding table.

It seems probable from this, therefore, that the lime mortars must have yielded to an injurious extent before there were any external signs. But whether this was the case or not, it is impossible to say, because the compression was quite uniform up to and in many cases much past the points of evident failure.

It seems fair to suppose that 1 week and 3 weeks are about the minimum and average times which would elapse before the maximum load might be put on a brick wall, and when it is remembered that these joints were less than ¼″ thick, the amount of compression in a high brick wall under a load of 80 or 90 lbs. per sq. inch is seen to be very great, and under a load of 300 to 400 lbs. per sq. inch, a brick wall 50 ft. high in lime mortar would not only fail, but compress from 2 to 6 inches in doing so—the compression practically all taking place in the mortar, as in the unyielding Portland cement mortar the compression is seen to be very small.

The second part of this paper will contain tests made on piers built with pressed brick, in which the mortar has had longer time to harden, and interesting results are looked for.

The brick in this case was, as mentioned in Table VII, common building brick. The photograph given illustrates the method of testing and the interesting manner of failure of 5th test, in which the lines of least resistance are clearly defined.

SERIES V.

EVAPORATION AND CRUSHING TESTS AND EVAPORATION AND TENSILE TESTS.

(*a*) *Evaporation and crushing tests.*

This series had for its first intention, information on the comparative and actual amount of evaporation of moisture from different mortars made with different cements, but it soon developed into an endeavour to obtain some relation between crushing strength and evaporation. Any law on the matter, if there is any general law, will of course take years to demonstrate; but enough has been done to show that any investigations on this subject will be fruitful of results. The method of procedure was as follows:—Mixtures were kept in damp air 30 days, then immersed 2 days in water of ordinary temperature, then taken out and

TABLE IX.

EVAPORATION AND CRUSHING TESTS.

No. 11—PORTLAND.

SERIES V.

Mixture.	Evap. % in 2 days.	C ushing strength per sq. in.	Product.	Max. wt. of 2″ Cube.	$\left(\frac{2}{\sqrt[3]{wt.}}\right)$	Column 4 divided by col. 6.
				oz.		
Neat.	1.48	3925	5809	10.43	22.16	262.1
1 to 1	3.41	2211	7539	10.12	21.71	347.3
2 to 1	6.20	1031	6492	9.39	20.66	314.2
3 to 1	10.39	544	5652	9.14	20.30	278.4
4 to 1	11.49	431	4952	8.92	19.97	247.9

No. 10—Portland.

Mixture.	Evap. % in 2 days.	Crushing strength per sq. in.	Product.	wt.	$\left(\sqrt[3]{wt.^2}\right)$	Column 4 divided by col. 6.
Neat.	0.97	4367	4231	9.84	21.31	199.0
1 to 1	2.20	3062	6736	10.23	21.87	308.0
2 to 1	5.59	1079	6032	9.43	20.72	291.1
3 to 1	8.61	*940	8093	9.15	20.31	398.4
4 to 1	11.68	504	5886	8.86	19.87	296.2

* One day older than others.

No. 3—Portland.

Mixture.	Evap. % in 2 days.	Crushing strength per sq. in.	Product.	wt.		
Neat.	4.65	1863	8662	10.00	21.62	400.7
1 to 1	4.10	1875	7687	10.12	21.71	354.1
2 to 1	5.67	1417	8034	9.60	20.97	383.1
3 to 1	8.11	687	5572	8.95	20.01	276.2
4 to 1	12.56	412	5176	8.88	19.90	260.0

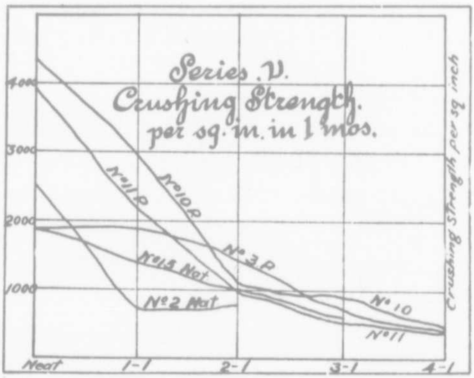
No. 15—Natural.

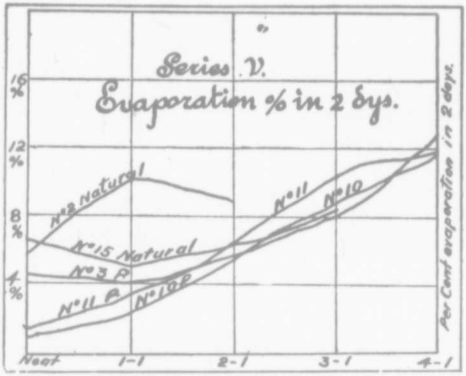
Mixture.	Evap. % in 2 days	Crushing strength per sq. in.	Product	wt.		
Neat.	6.76	1888	12762	9.40	20.67	617.4
1 to 1	5.08	1437	7300	9.65	21.02	347.3
2 to 1	6.12	988	6046	9.32	20.57	293.9
3 to 1	8.34	575	4796	9.05	20.16	237.9

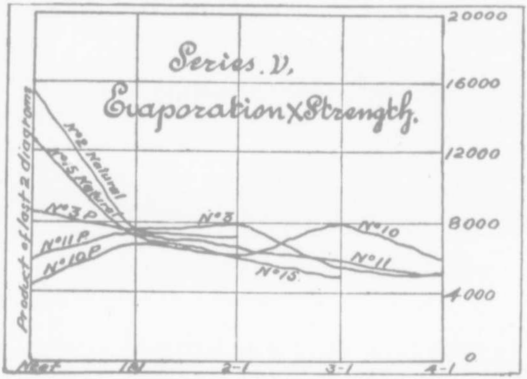
No. 2—Natural.

Mixture.	Evap % in 2 days.	Crushing strength per sq. in.	Product.	wt.		
Neat.	5.93	2575	15720	9.43	2072	758.
1 to 1	10.32	703	7254	9.06	2016	359.9
2 to 1	8.93	810	7233	9.28	2057	352.6

weighed ; they were then kept in the warm dry air of the laboratory at a temperature of about 65° F. exactly 2 days, when they were again weighed and immediately crushed. The experiments recorded in Table IX were all made on 2″ cubes, and 2 days was established, because it was found that at that time the evaporation was practically complete. Other experiments (not recorded) made on 3″ cubes gave less evaporation per cent. and also less strength. Attached to this are 3 diagrams : the first two show strength and evaporation in different mixtures and with 5 brands of cement. The third diagram is the product of the other two, and is quite worthy of inspection, because it would appear from it that it would be possible to estimate fairly and accurately, without actually crushing a specimen, what load it would bear.




Series. V.
Evaporation % in 2 dys.
16 %
12 %
8 %
4 %
Nº2 Natural
Nº15 Natural
Nº3 P
Nº11 P
Nº10 P
Nº11
Nº10
Per Cent evaporation in 2 days.
Neat
1-1
2-1
3-1
4-1


Series. V.
Evaporation × Strength.
20000
16000
12000
8000
4000
0
Product of last 2 diagrams
Nº2 Natural
Nº15 Natural
Nº3 P
Nº11 P
Nº10 P
Nº3
Nº10
Nº11
Nº15
Neat
1-1
2-1
3-1
4-1

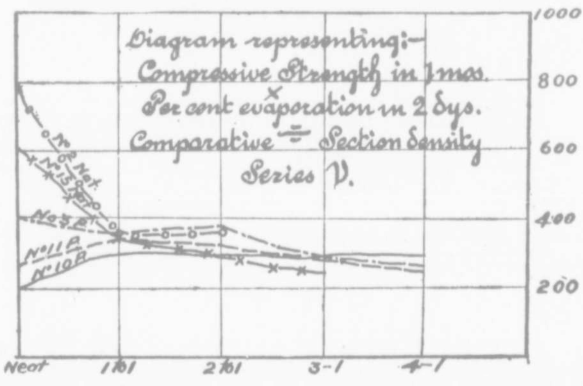


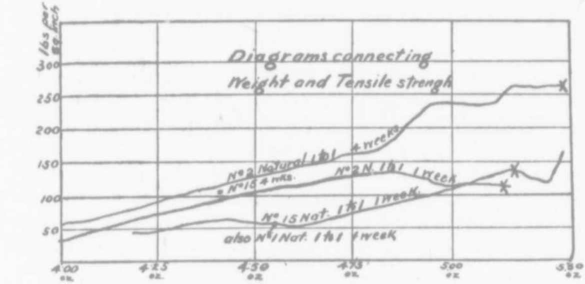
Diagram representing:—
Compressive Strength in 3 mos.
×
Per cent evaporation in 2 dys.
Comparative ÷ Section density
Series V.
1000
800
600
400
200
Nº2 Nat.
Nº15 Nat.
Nº3 P
Nº11 P
Nº10 P
Neat
1 to 1
2 to 1
3-1
4-1

Reference to the table and diagrams will show that the evaporation increases and the strength diminishes with the increase of sand in the mixture. This is, of course, almost self-evident, but the striking difference in the amount of evaporation for different cements neat is unaccountable. This difference disappears as the admixture of sand increases, and we are led, therefore, to conclude that there is something inherent in the cement itself, which aids it more or less in holding particles of water in suspension. The natural cements show high evaporation neat, so also does the No. 3 Portland, which has a high specific gravity (see general tables), and the cubes of which weighed more than those of the No. 10, which evaporated least. We cannot account for it on the ground of Portland and natural, but one thing is evident, that that same quality which enables it to hold water in suspension also aids it in holding particles of sand together, but not particles of itself. The third diagram showing the convergence of lines on the 1 to 1 mixture is very striking. The *product* of the *crushing strength of a* 1 *to* 1 *mixture* and the *evaporation per cent.* under conditions named is practically CONSTANT. This is for one condition only, namely, 32 days, with access of water and 2 days drying. This means in plain words that we may possibly be able to test with a balance instead of a crushing machine.

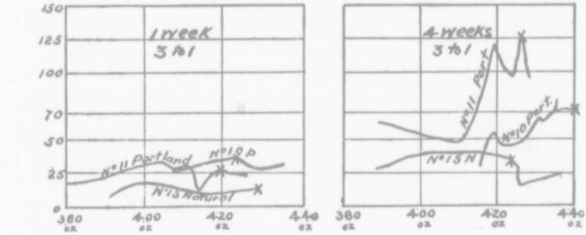
It is probable that the microscope would reveal a decided difference of structure in various cements. It is, of course, well known that the underburnt natural cements have softer, rounder and more easily pulverised grains than that produced by the highly burnt clinker of the Portland. It is possible, therefore, that the evaporation qualities of a neat cement would indicate more closely than anything else the degree of burning practised, independent of the fineness. It will be noticed by Table II, that the residues on sieves afford no clue to the density of the mixture, and no guide to determine beforehand the evaporation. Neither does the weight of the specimens vary at all regularly either with the crushing strength or evaporation.

It would seem that the coarse, angular laboratory sand had its interstices just about filled up with a 1 to 1 mixture, and the strength of the mixture depended directly on the amount of evaporation, in an inverse ratio. The Evaporation diagram No. 4 is the same as No. 3, except that this product is referred to a uniform section density (*i. e.*) $(\sqrt[3]{weight})^2$; the diagram is practically the same, showing that the variation in weight of test pieces made practically no difference in the results, *i.e.*, the per cent. of evaporation determines the strength in 1 to 1 mixtures, but is no criterion in neat ones.

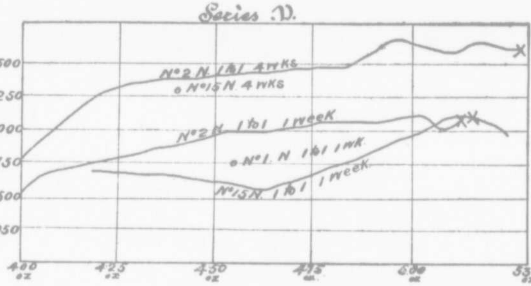
(*b*) *Evaporation and tension tests.*



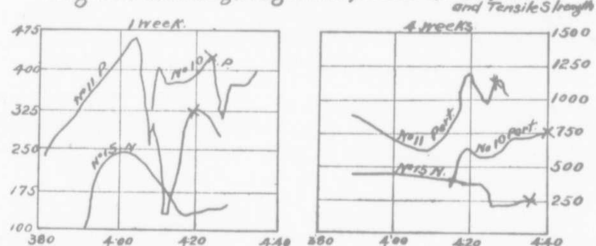
Pressure Tests.

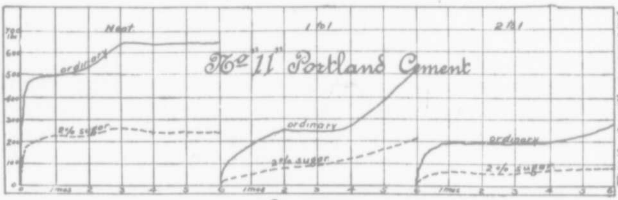


Series V.

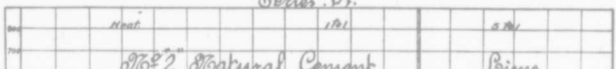


Diagrams connecting. Weight with product of:- Evaporation and Tensile Strength





Series VI.



In Table III. and Table IV. the per cent. of evaporation in 2 days is again given, and diagrams are plotted showing the relation between the tensile strength and the weight of the dried briquettes in the pressure tests, and also other diagrams showing the product of tensile strength and evaporation plotted on a base of weights of briquettes.

The × marks in the diagrams show the positions of tests made with 20 lbs. pressure and 20 p. c. of water, and they are seen to stand at prominent and usually maximum points on the diagrams, proving that this is the best point to select of all the tests made.

It will be seen in these diagrams as in those of crushing tests, that in 1 to 1 mixtures the variation of evaporation and strength combined is not very great, but not so close as in the former tests.

The 3 to 1 tests are very erratic, as might have been expected with different per cents. of water and different amounts of pressure. It is evident that each cement has distinctive qualities of its own, because with the same weight of briquette the strengths vary, and this brings up the important point that in sand tests the strength ought to be referred to some basis of weight of briquette, because a slight variation in weight seems, from Table IV, to affect the strength very much. It would not take much evidence to determine the average weight, and all tests could be reduced to this by multiplying by $(\sqrt[3]{weight})^2$ which would change the section density to a standard.

SERIES VI.

SUGAR TESTS.

Sucrate of lime is soluble in water, and it was chiefly a matter of interest to see the effect of sugar on cements in weakening them, because it has been asserted by several writers that the reverse is the case; one investigator several years ago showed by tests that from $\frac{1}{2}$ to 1 p. c. of sugar would in 4 to 6 months give a gain in strength.

Sugar, in these tests, 2 p. c. of the amount of cement (by weight), was used, and the diagrams attached sufficiently indicate the results. In the Portland cement the strength ranges closely at 50 p. c. of the ordinary strength as far as 6 months, while with the natural cements, the sugar effect was overpowering. After one week's immersion the briquettes showed signs of cracking, and as time went on became completely checked, and expanded so much as to give practically no tests. This is further evidenced (see exhibit of briquettes) by the upper surface, which was protected by a coating of iron deposited from Montreal water, being intact, while the checking was greatest on the bottom where the water had free access.

The lime mixtures, kept in open air, showed encouraging results for 2 months, and seemed to prove that the use of sugar, in lime, as practised in India, was beneficial; but the 3, 4 and 6 months' tests disprove it. Altogether, it seems evident that this much or more sugar would be damaging in its effects on any kind of mortar in any situation, and it is extremely doubtful whether any sugar whatever would have other than a weakening effect.

In concluding this paper, the author cannot but help feeling that he is, as it were, dipping just on the surface of a vast subject, and that the more one finds out, the larger the unknown fields beyond appear.

In any efforts that have been made, the frequent manual aid and more frequent sound practical advice of Mr. J. G. Kerry have been of much service, and here is the place to acknowledge it.

The endeavour has been to find out anything of practical use to the Engineering profession; and if any points raised here will fulfill this desire, the object of this paper will be, in the main, accomplished.

In conclusion, the author cannot but acknowledge the opportunity given by the Engineering Equipment of McGill University. In carrying out the various tests recorded, every facility has been offered not only for student instruction but for private research, and whenever

anything is needed that is not possessed, Professor Bovey, the Dean of Engineering, is always ready to have the want filled, if possible. In this way many things not feasible in ordinary cases are practicable, and it is hoped that, in due time, other results of value to the profession may be determined and presented to the Society.

CORRESPONDENCE.

Mr. H. F. Perley.

Mr. H. F. Perley said :—Relative to the subject of cement testing, I would state that there has been, and no doubt still will be, a large amount of information furnished by writers in different countries, for the subject is one in which the mechanical part possesses an amount of interest more or less fascinating, whilst the scientific part can only be indulged in by those whose training and education have fitted them to pursue that line of study. But in spite of all that has been written, and all that we have been told, experimenters and scientific persons have not yet devised a scheme, a test, or an analysis, which will enable a contractor, or a user of Portland cement, to quickly and accurately determine the value of the article he has procured, or which has been furnished for his use. The tests in vogue are numerous, each giving a different result, but they all require time, and plenty of it, which can ill be spared during the prosecution of a work where "time is the essence of the contract"; for tests and trials at any other time can only be carried on in the laboratory where a "handful of minutes" is not of much importance, and often by those who may be *au fait* as regards the tests, but whose knowledge of the practical use of cement is but small.

Relative to this matter, the late Henry Fairja, in a letter in the *Engineer* of 2nd of March, 1894, stated that, "if a cement is unsound and does not comply with the mechanical test specified, let it be rejected, and leave it to the manufacturer to find out where he is wrong; but let the quality of the cement be decided before it is used in the work, otherwise, in the event of failure, complications may arise as to whether such is due to the cement, to the aggregate, or the manner of use. If users could only come to this conclusion, we should hear no more of magnesia or anything else, which would be unspoken secrets known to manufacturers only, and we should hear only of cement being either sound or unsound, which for all practical purposes is sufficient."

In Canada, contractors are often obliged to use imported cements, because those who prepare the specifications under which they work labour under the impression that cements of foreign make give the best results, and they base their opinion on results obtained in the country where such cements are manufactured, forgetting the fact that there, manufacturers are numerous, competition is keen, and vast quantities are required for home consumption, and therefore the quality of the manufactured article must be good, to ensure which the German cement manufacturers have established standards to which all must attain; and it is to be regretted that standards do not exist in England as well. When cement is manufactured for exportation, it is doubtful if the article is as good as it ought to be. Much of the cement imported into Canada is not obtained under a direct order, but arrives as ballast, and is sold on almost any terms; and therefore it cannot be expected that the vessel owner would purchase other than the cheapest grade for his purpose. This article is, of course, rightly termed Portland cement, but it is a cement of the poorer class, as evinced by its price in the market. We have a few Portland cements made in Canada, which are suitable for any class of work, but they have not an extended use, simply because they cannot compete with the foreign article brought to our ports, and perhaps distributed on through freight rates by being hampered with (1) high *local* freight rates, and (2) the cost of barrels and packing, which alone amounts to more than the freight of a barrel of cement across the Atlantic. If our engineers would only specify that cement should be bought by weight—with a limit on the weight per cubic foot filled under imposed conditions,—and delivered in bags, then our cement makers would have a greater radius of action, and be better able to compete with the imported article; and contractors would only have to pay for the use of the bags, which would be returned for further use, for every empty barrel represents loss and absolute waste, which ought to be avoided.

If the committee on cements appointed by the Society can form a set of tests for cements, which will be simple in their application and quick and accurate in their results, they will confer much favour and benefit on the users of cement.

Mr. Fred. P. Spalding.

Mr. Fred. P. Spalding, M. Am. Soc. C.E., of Cornell University, said:—The paper of Mr. Smith is a very interesting one, and raises some rather difficult questions, the final settlement of which will require a much more extended knowledge of the nature and action of hydraulic cement than we now possess. As most of the questions discussed in this paper have been subjects of inquiry by the writer during the period that he has been in charge of the Cement Laboratory at the College of Civil Engineering of Cornell University, a brief statement of those points upon which he has been led to conclusions differing from those of the author of the paper may be of interest.

There can be no doubt of the wisdom of using sand tests for the tensile strength of cements in so far as they can be made to give uniform and reliable results. The chief difficulty in prescribing a sand test for the quality of cement lies in the variable nature of the sand. Even with the artificial sand now used for standard tests, the quality will not always be found the same, and the results of tests may often be considerably affected in consequence. Tests of the quality of the mortar to be used in any work, by tensile tests with the sand in use for the work, would be of much value, but the advisability of dispensing with neat tests for determining the quality of the cement is questionable. It is true that various cements differ somewhat in their power to "take sand;" it likewise seems to be true that any cement which gives good results, neat, and is finely ground, will show good strength when tested with sand; while no short time test, either neat or with sand, can determine the actual relative values of samples of cement of different brands.

The desirability of using a method in preparing briquettes which shall eliminate the personal error of the operator is unquestionable. This is the most serious difficulty met in testing tensile strength. A single operator may readily obtain even results by any of the ordinary methods, but the results of different men with the same material are likely to differ widely. The problem in devising a specification is to secure uniformity in the work of different experimenters. The results of experiments in the Cornell Laboratory indicate that if a direct pressure be used sufficient to compress the material into a compact and homogeneous briquette, the average results obtained by different men agree quite closely with each other. This requires a pressure of about 100 lbs. per sq. inch over the surface of the briquette. With small pressures (20 to 30 lbs.) the results were found to differ as much as in ordinary hand work. The heavier pressure therefore seemed preferable. It requires no expensive apparatus, and is easily applied anywhere.

With reference to the quantity of water to be used in mixing, the experience of the writer is that no definite quantity can be fixed upon as applicable to all material; what is right for one cement is too much or too little for another. It is better to determine the water in each case by a standard of consistency.

The proposition of the author of the paper, that bubbles be deducted from the sectional area in tensile tests, is quite inadmissible. The tensile strength of cement briquettes is not proportional to the area of the section.

In a recent discussion before this Society, the present writer was quoted by Mr. H. R. Lordly as expressing an opinion adverse to the use of hot water for mixing mortar. This was disposed of by the author of the paper then under discussion, by the reply that the writer "must have been unfortunate in his cements." This was a very easy way to settle the question, but unfortunately does not seem a conclusive argument. Mr. W. W. Maclay, from an extended series of experiments upon this matter (see Trans. Am. Soc. C.E., Vol. VI, p. 311), concludes that the use of hot water in mixing is detrimental to the strength of the mortar. The loss of strength when the mortar was mixed with hot water in a cold atmosphere was found by Mr. Maclay to be very serious, and when the briquettes were placed in cold water they lost coherence.

Experiments made in the Laboratory at Cornell University have shown that different brands of cement are affected very differently by the use of hot water in mixing. The writer has experimented upon about a dozen brands of cement in this particular, of which four were but slightly affected by the temperature of the water, giving much the result found by the author of the paper now under discussion. The others were all materially weakened by hot water, and three of them were rendered entirely worthless when the temperature of the water reached 120° to 150°F., the mortar never setting sufficiently to resist crushing under the pressure of the fingers. All of these cements were of good quality, and satisfactorily resisted the hot bath tests for permanence of volume.

Mixing the mortar with hot water and subjecting it to a cold atmosphere is by no means the same in effect as mixing with cold water and then subjecting it to heat.

The effect upon the rate of setting produced by mixing with hot water varies as much as that upon the strength. The rule seems to be that those cements which are accelerated in action are injured in strength. With some of the uninjured cements, no acceleration of the rate of setting took place, and in one case there was a decided lessening of the rate of setting as the temperature of the water was increased.

A single example may serve as an illustration of the action of the cements most affected. One brand mixed with water at 40°F. set in 70 min.; with water at 70°, set in 30 min.; with water at 100°, set in 10 min.; with water at 120°, set in 3 min.; with water at 140°, set in 2 hrs. 20 min., but did not get firm. Tensile strength at 100°, about half that at 40° in one week. Temperature of air 65°.

It has occurred to the writer that there may be a point for some cements at which the process of setting occurs so quickly as that the individual particles fail to combine with each other as in a slower crystallisation.

In giving this brief statement of what seemed to be shown by the few experiments at Cornell University, it is not intended that any conclusion should be drawn from them other than the one already stated, that hot water affects different cements very differently, that it is unsafe to draw general conclusions in the matter from a few tests on a particular material, and that the whole subject has yet to be investigated.

Mr. J. G. G. Kerry.

Mr. J. G. G. Kerry said he had read with much pleasure Mr. C. B. Smith's paper on cement testing; and having had the good fortune to be with Mr. Smith while he was making many of his experiments, wishes to bear evidence to the careful and conscientious manner in which these elaborate series of tests were carried out; and, knowing perhaps better than anyone else the amount of painstaking labour and self-sacrifice that these tests have necessitated, is anxious to voice the thanks which are due to Mr. Smith, both for these and for the clear and explicit form in which he has presented his results to the Society.

The greater part of the paper, dealing as it does with the history and results of the experiments, is of course beyond discussion; but as Mr. Smith has confined his own comments to those that can be made from a laboratory point of view, he has left a field open for discussion in the practical significance of some of his results.

As the quality most required in cements is durability under ordinary exposures, it is disheartening to read the remarks in paragraph (*e*), p. 6, on the probable inefficiency of the blowing test. Present evidence seems to point more and more to the necessity of chemical analysis as a part of cement testing, if we are to escape from the often costly appeal to trial and time, which is often quoted as the only authoritative test of a cement. A cement may fail in any one of the prescribed tests, and yet prove satisfactory in service; but if it have a dangerous constitution, it must prove unsatisfactory, and while a good chemical analysis is not proof of the excellence of the cement, a bad chemical analysis is certain evidence of its worthlessness; and in this respect chemical analysis is perhaps more sharply determinative of value than any other single test. The writer regrets that the chemical and manufacturing side of the question has not been more fully taken up in the various papers read before the Society on this subject. The particular facts which would prove useful in practice would be a knowledge of the dangerous ingredients that might possibly be present, and their probable distribution throughout any batch of cement. It is of course well known that in any burning, clinker of varying qualities results, but this is usually attributed to better or worse burning, and not to chemical combination; and there does not seem to be much information obtainable concerning the chemical variation of the output. This variation is mainly a manufacturing question, but a knowledge of its extent would furnish a measure of the number of analyses necessary to establish the purity of the cement.

The absolute importance of specific gravity tests is open to question. The other specified tests determine some necessary quality in the cement, but specific gravity is taken as an indication of sufficient burning in the manufacture; and as it is an indirect test, it is doubtful whether an engineer would be justified in condemning a cement on specific gravity alone. The testing sheet attached to the paper indicates that high specific gravity is not an indication of a good cement, nor low specific gravity of a very poor one. The writer would like to know from Mr. Smith whether a cement tested at different finenesses shows any variation in its specific gravity; theoretically, the grinding should have no effect on this quality, but the imperfections of the methods usually employed for this test might cause a discrepancy.

The advisability of sand testing, as discussed on p. 6, may prove doubtful by reason of the present rapid improvement of cement manufacture. The strength of a briquette depends upon three main features,—the cementitious activity of the cement, the fineness of its grinding, and the sand used. The results of many experiments make it certain that it is only the cement in form of an impalpable powder that has any cementitious value, and as this fact becomes more generally known the grinding clause in cement specifications will be made much stiffer. It is probable, though not established by experiment, that there would be a definite relation between the strengths developed by neat and by sand-testing, if only that portion of the cement were used which is known to possess cementitious value. If such prove the case, the sand test is of use only as an indirect test of fineness, and this quality can be more simply tested with sieves; and as sand testing necessarily introduces a third variable and is more difficult to carry out uniformly, it can be discarded. The value of sand-testing, as a demonstration of the inefficiency of a coarse ground cement, is beyond question; but the argument that a cement should always be tested with sand, because it is always so used, is not of very great weight in view of the tremendous variations between laboratory sands and the sands of practice, and the further fact that the mortar is not tested for the strengths that are required of it in practice.

The attack on p. 6, on the importance attached to the results obtained by "that terror of the present day, the testing machine," is well grounded. There are few structures to day built or building that have or will develop a pressure of 100 lbs. per square inch on a month old mortar, and the attached testing sheet shows that any cement that is at all good will develop strengths vastly in excess of this. There seems to be no value whatever attachable to the numerical results obtained by tension testing, and it is yet to be proven that a cement that will give materially higher results in tension testing than another is really the better of the two. Undoubtedly all cements should show a certain minimum strength; but it seems from paragraph (*e*), p. 6, that attaching any value to the fact that the strength of a cement proves materially greater than that minimum is simply putting a premium upon the introduction of certain dangerous adulterants. If a careful series of tests were made to ascertain the maximum and minimum strengths of unadulterated Portland cements, it would be possible in some degree to guard against such adulterants by introducing both the maximum and the minimum strengths into the specifications; this idea has been carried into practice occasionally.

Averages of Table III.

Percentage of water.	One Week.				Four Weeks.			
	Tensile strength lbs.	Extremes of strength, lbs.	Weight oz.	Evaporation pr'ct'ge	Tensile strength, lbs.	Extremes of strength, lbs.	Weight oz.	Evap'ration p'ct'ge
Natural 1 to 1								
15	46	23–86	4.80	13.52	82	39–112	4.89	13.80
$17\frac{1}{2}$	120	37–184	5.35	8.37	208	160–282	5.30	7.69
20	125	90–157	5.61	8.73	225	......	......	.....
$22\frac{1}{2}$	119	106–130	5.62	9.04	....	......	......	.....
Natural 3 to 1								
15	16	9–23	4.70	14.85	33	19–55	4.59	14.02
$17\frac{1}{2}$	7	2–12	4.59	14.66	37	25–48	4.70	10.92
20	12	7–17	4.79	11.77	20	5–28	4.88	11.78
Portland 3 to 1								
15	30	14–41	4.60	12.31	63	51–86	4.73	12.44
$17\frac{1}{2}$	28	16–43	4.59	10.68	87	47–124	4.79	10.33
20	31	20–48	4.77	12.27	96	56–143	4.80	10.50
$22\frac{1}{2}$	27	22–36	4.89	12.50	73	38–103	4.82	11.80
25	$22\frac{1}{2}$	8–33	4.88	13.30	43	23–58	4.78	12.75

In order to test the conclusion on p. 9 with regard to the percentage of water necessary for pressure testing, the writer averaged the results of Table III, disregarding, in so doing, the fact that the tests were made under two different pressures; these averages are given in the accompanying table, and indicate that percentages varying from $17\frac{1}{2}$ to $22\frac{1}{2}$ will give good results with the least variation in results with $22\frac{1}{2}$ p.c., and show that Mr.

Smith's choice of the mean value 20 p.c. is well made. Disregarding the 15 p.c. results as the briquettes with this percentage were evidently not perfect, the table points out two facts of practical importance, for it will be noted:

(1) That the percentage of evaporation steadily increases with the percentage of water used;

(2) That the percentage of evaporation diminishes with the duration of set.

If the percentage of evaporation be taken as a measure of the cement's imperviousness, it is evident that when it is necessary to construct practically water-tight works with cement mortars, that, up to some unascertained limit, the drier the mortar is, the better the result will be, a mortar being thus preferable to a grout, and that the longer the mortar can be allowed to set before being exposed to water pressure, the less will the liability to leakage be. In this connection Table IX is of great interest, although it must remain an open question for some time to come, whether the fact that Mr. Smith is endeavouring to establish is merely a strange coincidence, or whether it has some physical basis. The percentages of evaporation in the table show that Portland is much superior to natural cement for water-tight work, and that a 1 to 1 natural is about equal in this respect to a 3 to 1 Portland. In the test of No. 15 natural, the percentage for a neat cement is given as considerably larger than for a 1 to 1 mortar. This peculiar fact is more than confirmed by a series of direct percolation experiments made by Mr. F. C. Coffin of Boston, and published in *Engineering News*, January 3 and 10, 1895. Mr. Coffin found that while neat and 1 to 1 Portland and 1 to 1 natural made good water-tight joints, the leakage through neat natural was enormous. Mr. Smith's results do not show as great a discrepancy, but this is probably due to the fact that the Belgian natural used by him approaches Portland cement much more closely in constitution than do the Rosendales used by Mr. Coffin. It is to be remarked, however, that the results given for No. 2 natural are in flat contradiction in this respect to the results of the other experiments, and indeed appear erratic in themselves.

Though agreeing with the general conclusion to be drawn from the remarks on compression testing on p. 11, it does not appear to the writer that statement (2) is borne out by the testing sheet, and he has prepared the accompanying Table from the results thereon, to indicate the lack of any definite relation between the tensile and compressive strengths of cement mortars. Authoritative deductions cannot be drawn from a table based on so few experiments;

CO-EFFICIENTS CONNECTING TENSION AND COMPRESSION STRENGTHS OF CEMENT MORTARS CALCULATED FROM THE FIGURES ON THE TESTING SHEET.

Number of cement.	NEAT.						1 TO 1.			2 TO 1.			3 TO 1.	
	1 wk.	2 wks	3 wks	1 mo.	2 mos	1 year	1 wk.	1 mo.	2 mos	1 wk.	1 mo.	2 mos	1 wk.	4 wks
1	..	..	..	..	7.1	..	..	..	..	..	..	..	..	..
2	8.5	..	..	8.4	..	..	11.8	11.7	..	..	..	..	..	..
3	..	..	..	..	..	..	..	..	..	..	..	..	6.8	6.6
4	..	..	..	..	..	..	..	..	..	..	..	..	8.0	..
5	3.4	..	..	5.4	..	..	..	..	..	..	..	..	..	..
6	5.1	..	..	..	..	..	..	..	..	..	..	..	..	8.1
7	6.5	..	..	5.2	..	..	..	..	..	..	..	..	..	..
8	4.8	..	..	5.0	..	..	..	..	..	..	..	..	..	..
10	..	..	..		..	..	..	10.5	..	..		..	..	..
11	..	..	..	..	..	..	..	13.9	13.0	..	..	4.3	..	.
15	6.0	..	..	6.8	..	..	..	..	..	15.5	12.4	..	..	..
19	5.5	..	..	8.3	..	..	..	..	..	..	..	..	5.4	5.8
20	..	..	..	..	..	12.0	..	..	..	..	..	..	..	..
21	..	..	11.5	..	..	7.5	..	..	..	..	..	..	..	..
22	8.0	..	..	5.3	..	8.3	..	..	..	..	..	..	..	..
23	..	6.7	..	8.1	..	..	..	..	..	..	..	..	..	..

but it indicates:—

(1) That the compressive strength cannot be closely predicted from the tension tests;

(2) That sand mixtures show a higher co-efficient than neat cements;

(3) That the co-efficient increases with the age of the cement.

The writer has never seen the fact of the greater comparative strength developed by the sand mixtures commented upon; but this result would naturally have been expected. The record of the tension tests shows that cement mortars have reached very closely to their ultimate strength at the age of three months, and the increase of the co-efficient with age indicates that this does not apply to the compression strength of the mortar, and that we do not yet know when it ceases growing harder, stronger in compression and more brittle.

The value of the results in Series II, p. 13, will be greatly increased, if Mr. Smith will add to them the times of setting at the varying temperatures. This is a point of considerable practical importance, as it has been found, particularly in pneumatic work, that in the high temperature and heavy pressure of a caisson chamber, some cements will set almost instantaneously, so rapidly, indeed, as to prevent their use, because they are set before they can be deposited in place. The results given in Table V, p. 14, indicating as they do both shortening and lengthening of time of set with higher temperatures, would seem to prove that heat acted differently under almost similar circumstances.

The results of the freezing tests fully confirm the ideas of current practice; but there is one field of this part of the subject which does not seem to have been touched by investigators. Many engineers to-day are of the opinion that the most dangerous treatment that a cement can be exposed to is to be mixed at very low temperatures. There is no question that a cement mixed with heated sand and heated water will set perfectly in sharp freezing weather; but will the same cement mixed with water at ordinary winter temperature, which is always within a very small fraction of 32° Fahr., in large streams and with cold sand, set at all, or will the low temperature of the mass more or less completely kill the activity of the cement? There seems to be no definite information extant upon this point, although it is one of great importance.

The ordinary objection to the use of salt is the same as that to the use of sugar, namely, that it remains in the mortar after it is set in a soluble form, and will sooner or later weather out. The writer would like to know whether Mr. Smith exposed any of his specimens to percolation or running water in order to develop such a tendency before testing them.

The results of the pier testing as embodied in Table VII, p. 18, are very interesting and of great practical value; it is a pity, however, that the strength of the brick itself is not given, that being one of the principal factors in the practical problems; and the effect of using a stronger building material is as yet unknown, at least in amount, and can only be determined by a series of similar experiments. The comparative strengths developed by the Portland and natural cement mortar in the cubes and in the piers are discordant, and this fact indicates that the duty of cement mortar in joints is not only to transmit compressive strains, but also to resist a tendency to flow sideways out of the joint under pressure. Its power to resist this latter tendency seems to be the actual measure of the practical utility of the cement, and must be closely related to its adhesive strength; and it is noticeable in the adhesion tests of the same mortars that the stronger in the pier tests proved the stronger in adhesion. The distribution and transmission of stress in the joints of a heterogeneous mass like brickwork is not clearly understood; and as Mr. Smith has at various times kindly considered and developed crude suggestions of the writers, he would now suggest that the knowledge of this action might be increased by an experiment on piers built with a non-adhesive mortar, such as dry sand, the sand being held in place by a pointing of strong cement mortar. In discussing a paper on the Masonry of the Cheat River Bridge, read by Mr. Smith before the Society in 1893, the writer described some instances of masonry abutments founded on rock, showing some considerable settlement after the banks had been built against them, and asked an explanation of this action from the members then present. None was offered, but as pressures developed in an abutment by a green earth bank are enormous, Mr. Smith's demonstration of the compressibility of natural cement mortars is a satisfactory answer, as the abutments in question were built with natural cement mortar.

Throughout the paper, Mr. Smith comments on the many circumstances under which natural cements are fully as good as Portlands, and protests against the seemingly growing tendency to specify Portlands for everything; and his opinions are well grounded upon the results of his experiments. There is no question but that it is better to use Portlands under the special circumstances mentioned at the head of p. 7; but in the writer's personal practice, fully 75 p.c. of the cement he has used has been laid under the precise circumstances shown in the paper to be most favourable to natural cements, *i.e.*, laid in summer weather and only liable to occasional submergence, and this piece of experience is probably the rule and not the exception. The variations in the price of cements in different localities are entirely due to transportation charges, and comparing a 1 to 1 natural with a 3 to 1 Portland mortar the following note of costs per cub. yard of mortar shows that in the score of cheapness the choice is entirely a matter of location, the prices for cements used being the extremes that have come under the writer's notice, sand being taken at 75 cts. per cub. yard.

	Cement.	Sand.	Labour.	Total.
Portland @$1.75 per bbl...........	3.24	.65	.50	4.39
" @ 3.00 " "	5.56	.65	.50	6.71
Natural @ 0.62½ " "	2.56	.45	.50	3.51
" @ 1.50 " "	6.14	.45	.50	7.09

If the two mortars be compared by their results throughout the tests, the natural shows a marked superiority in everything, and it is further claimed that the naturals give promise of being much the more durable of the two. Canadian natural cement is usually condemned offhand because of the uncertainty of the product, and in view of the many trials it has been given, and the frequency of the condemnation pronounced by eminent engineers, it is evident that it will never come into general use until it is manufactured in much better grades; but it is also certain that there are many firms to-day manufacturing natural cements in the United States that are every bit as reliable as the best Portlands, and Canadian manufacturers should be competent to produce a like result. With regard to this comparison of Portland and natural cements, the results of the table on p. 12 show that the Portlands promise to make a better comparative showing when the standard of fineness is raised. The writer has prepared the accompanying comparative table from the tests in the paper, to show the relative values of the two mortars in so far as they can be shown by the testing machines, and in concluding would draw attention to the remarkable variation in the comparative tension strengths, when tested in the ordinary manner and when tested with Mr. Smith's pressure apparatus. This was first pointed out to the writer by Mr. Smith, in conversation, and has yet to be explained.

COMPARATIVE TABLE—NATURAL AND PORTLAND CEMENTS.

TEST.	LBS. PER SQ. IN.		MIXTURE.	REFERENCE.
	Natural	Portland		
Tension 1 wk.	72	57	N 1 to 1	Canadian cements on testing sheet.
" 1 mo.	109	91	P 3 to 1	
" 1 wk.	125	31	N 1 to 1	Averages of Table III.
" 1 mo.	225	96	P 3 to 1	
" 1 mo.	122	75	N 1 to 1	Table to Series 1. Ordinary
" 1 mo.	94	124	P 3 to 1	" " " sifted thro' 120 sieve.
Compn. 1 wk.	900	298	N 1 to 1	Canadian Cements on testing sheet.
" 1 mo.	1350	750	P 3 to 1	
Adhesn. A.	26	12	N 1½ to 1	Averages of Table VI.
" B.	66	22	P 3 to 1	
Pier Compn.	968	755	N 1½ to 1 P 3 to 1	Table VII.
Evaporn	7.70	9.04	N 1 to 1 P 3 to 1	Table IX.

Mr. J. L. Allison, Mem. Can. Soc. C.E., said:—The testing of cements for the Soulanges canal was commenced on the 9th October, 1891, and has been continued up to the present date. Mr. J. L. Allison.

Thirty-nine brands of Portlands have been tested, twenty-one English, eleven Belgian, five Canadian, and two German. Three brands of natural cement have also been tested. Over 17,000 briquettes have been made, all by the same man.

Cements have generally been purchased on the Montreal market through a commission merchant. A few barrels have, however, been sent by manufacturers or their agents, for the purpose of being tested. As a rule, two barrels of each brand have been used, in order to make it moderately certain that the cement was of normal composition.

When received, the barrels are stored in a dry room connected with the office. On the opening of a barrel the contents are removed to a depth of about five inches, and the quantity necessary for all the tests is taken from the central portion of the barrel, which is then set aside and never again used for testing purposes. From time to time these opened barrels are removed from the store room. When necessary, cements are air-slacked in this room.

All cements are subjected to the same tests, namely: (1) rate of setting, (2) specific gravity, (3) tensile strength, neat, and with sand, (4) fineness of grinding, and (5) soundness. The proportion of water required for gauging is also carefully determined.

Immediately on opening a barrel, the time required for setting is found by mixing a sample to a paste with water, and noting the penetration of Vicat's and Gilmore's needles.

The paste is placed in a mould 40 m.m. in depth, and the time of initial setting is taken as being the time at which the Vicat needle ceases to penetrate to a greater depth than 20 m.m. If initial setting does not take place in less than ten minutes, the full quantity required for making all the tests is placed in glass jars which seal air-tight. These jars are at once labelled and placed on shelves in the testing room. If, however, initial setting is found to have taken place in less than ten minutes, the quantity necessary for all the tests is exposed to the air in the store room in shallow pans, and turned over every day until the time required for setting has reached ten minutes, when it is put in the jars, as stated before. This limit of ten minutes is taken because that length of time is required to properly gauge a paste and fill a dozen moulds, and all work on the paste should be finished before setting has commenced.

The time of setting noted in the accompanying table is that found on opening the barrel.

The density is found by determining the specific gravity. No value is attached to the weight per bushel, or per cubic foot, as the range, within wide limits, depends on the method of filling the measure. Thus, with one brand the weight per cubic foot varied from 81 lbs. unpacked, to 121 lbs. packed, or nearly 50 per cent. The packing was done by jarring the measure, but no pressure was applied to the cement. The weight per cubic foot (both packed and unpacked) of all cements tested has been determined carefully, and the results shew that no reliance whatever should be placed on the weight as a measure of the density. The extreme range in the specific gravity of the cements tested is so small (about 6 per cent.) as to be neutralised by the greater effects due to differences of grinding, etc. Thus, the Johnson (coarsely ground), sp. gr. 3.023, weighed 84 and 111 lbs. per c. foot, unpacked and packed, while the Josson (finely ground), sp. gr. 3.174, weighed only 80 and 105 lbs. per cubic foot.

The specific gravity is determined by the volumetric method, the volumeter used being of 200 c.c. capacity. The quantity of liquid used in each experiment is about 125 c.c., and the weight of cement used is always 200 grammes. In order to prevent the setting of the cement in the volumeter, turpentine is used instead of water, and to prevent changes in the volume of the turpentine, the volumeter and its contents are kept at a constant temperature during the test by being kept standing in a jar of water at the temperature of the room. A thermometer is used to insure both readings being taken at the same temperature. Two tests are made with each cement, and the mean taken as the true result. This test is made, in all cases, on the cement as received—that is, without exposure to the air.

All cements, when tested neat, are mixed with water in such proportion as to give pastes of the same consistency. The apparatus used for determining this proportion consists of a brass cylinder of 80 m.m. diameter and 40 m.m. in depth, and a round brass rod 10 m.m. in diameter weighed to 300 grammes. The method of using them is as follows: the cylinder is filled flush with a paste made up with a known percentage of water (by weight). The rod is then placed vertically on the surface and allowed to sink under its own weight. A penetration of 34 m.m. is taken as indicating the proper consistency, and tests are made with different proportions of water until the proper penetration is secured. This test is made on the cement when ready to be tested,—that is after exposure to the air when necessary.

The water used is the same as that used for making the briquettes for tensile tests. It is taken from the St. Lawrence River, and is without visible impurity. The temperature at which it is used is always between the limits of 60° and 70° Fahr.

Tests of the tensile strength of all cements are made on briquettes of neat cement, and also of cement mixed with sand.

All gauging has been done with Faija's cement gauger. This machine consists essentially of a circular, flat-bottomed, cylindrical vessel, 10 diam. and 5 deep, in which a four-bladed mixer is rotated by a vertical shaft. The blades are in length about one-half the diameter of the vessel, and the vertical shaft carrying them is, by means of a crank centered over the centre of the vessel, made to travel in a circular path midway between the centre of the vessel and its circumference. An additional rotary motion in the opposite direction is given to the blades by means of a pinion on the head of the shaft, which engages with the teeth of a fixed annular gear on the under side of the frame carrying the crank. The blades rotate about 2.6 times while making one revolution about the fixed centre, and this relation insures the whole area of the vessel being worked over by the blades. The frame carrying all the working parts can be quickly removed and the vessel left unobstructed for the removal of the paste.

This machine is illustrated and described in Faija's "Portland Cement for Users," and also in *Engineering News* of 1st March, 1894.

The moulds used have a minimum section of one inch square, and are of the usual shape, furnished by the makers of testing machines. They were procured from the Fairbanks Company.

A sufficient quantity of cement to make 12 briquettes (about 1800 grammes) is put into the gauger, and has added to it the proportion of water previously determined; the gauger is then turned quickly until the mixture is complete, after which the paste is immediately filled into the moulds which have been previously given a film of oil and arranged on thick glass plates. In filling, no pressure is applied to the paste, but it is worked into the moulds with the point of a small trowel moved edgewise. The surplus paste is then removed with the trowel, and the numbers from a prepared list are stamped on the soft briquettes with dies, after which the moulds, on the glass plates on which they were filled, are placed in covered pans containing a little water, the plates being supported above its surface. The time at which they were gauged is entered with the numbers in the record book. The moulds are not removed from the briquettes until the cement is hard set; and, after the removal of the moulds, the briquettes are kept in the moist air until the expiration of twenty-four hours from the time of gauging, when they are immersed in water in pans about 30″ square, arranged on wide shelves occupying one side of the testing room. The briquettes are arranged in the pans according to the dates on which they are to be broken, and the pans are labeled to shew these dates. The arrangement of the numbers in the record book is decided on before the briquettes are made, and a list is posted on the testing room each morning, shewing the numbers to be given to the briquettes made during the day, and the tests for which they are intended.

The twelve briquettes in one gauging are distributed in the record forms over six tests; consequently, the twelve briquettes broken for any one test are made up of two from each of six gaugings. This reduces the effects of differences in the gauging and filling of different batches. The temperature of the room is kept within the limits of 60° and 75° Fahr. The water covering the briquettes in the pans is drawn off at intervals by means of a syphon, and replaced by fresh water.

The neat briquettes are broken at the end of three, seven, fourteen and twenty-eight days, and two, three, six and twelve months, the time in all cases being counted from the date of gauging.

When mortar briquettes are to be made, the proportions of sand and cement, determined by weight, are placed in the gauger and thoroughly mixed dry. The water is then added and the mixing continued until the mass is uniformly moistened. Only enough water is used to moisten the mixture sufficiently to form a stiff paste. This paste cannot be properly filled into the moulds without being slightly compacted or compressed. A certain degree of compression is effected by heaping the mortar about one inch higher than the mould and beating it down with a paddle-shaped tool of iron, about one foot long, weighing about 12 ounces. This method of filling was adopted, in order to avoid lack of uniformity in the strength due to applying pressure with a trowel. The filling has always been done by the same man, and the tests show that the compression is practically uniform, since all the briquettes in a set (twelve) made up from six gaugings break with nearly the same load. The extreme range is quite often within seven pounds, and is in some cases as small as three pounds.

The procedure after filling the moulds is the same as with neat briquettes.

The mortar briquettes made previously to June 13th, 1892, were mixed in the proportions, by weight, of one of cement to two of sand. Washed and screened pit sand was used; only that portion which passed the 20 and was retained on the 30 sieve being made use of in the tests. The briquettes were broken after seven, fourteen and twenty-eight days.

Since the above date the proportions used have been (by weight): one of cement to three crushed quartz and 33-100 water. The quartz used is known on the market as No. 5. It all passes the 20^2 sieve, and about 60 per cent. is retained on the 30^2 sieve. The part passing the 30^2 sieve contains no dust or very small particles.

The briquettes are broken after seven, fourteen and twenty-eight days, and two, three and six months.

From the date of the commencement of the tests, until January, 1893, all briquettes were broken on a Fairbanks machine of 1000 lbs. capacity.

In this machine the leverage is constant, while the load is variable, and is applied through a system of compound levers. The load at the beginning of each test is zero, and is increased gradually, by the addition of small shot, until breakage takes place. The breaking stress is found by weighing the breaking load on the same machine, the scale and weights being marked so as to give the breaking stress in pounds. With briquettes of high strength many of the breaks did not occur at the minimum section, but on a line between the points of contact with one of the grips.

Since the above date a Riehlé machine of 2000 lbs. capacity has been used. In this machine the weight is constant while the leverage is variable. The test piece is strained by moving the constant weight out on a simple lever until breakage takes place, when the breaking stress is read directly from a graduated scale on the lever, at the point indicated by a pointer attached to the weight. The grips are provided with renewable rubber tips which insure the proper application of the force, as shown by the fact that no briquettes have broken at the points of contact with the grips.

The load has, in all cases, been applied approximately at the rate of 400 lbs. per minute.

A list of briquettes to be broken each day is prepared from the records and posted in the testing room. For the short tests (3, 7, 14 and 28 days) the briquettes are broken at the same time of the day as they were gauged.

The fineness to which cements have been ground has been tested by sifting samples, taken from the centres of the barrels, through sieves of 625, 900, 2,500, 6,400 and 10,000 meshes per sq. inch.

The accompanying table shows the percentages retained on the 625 (25^2), 2,500 (50^2), and 10,000 (100^2) sieves, the results given being means of two tests for each cement. The same set of sieves has been used for all the tests. They are nine inches in diameter and two inches deep, with meshes formed of woven brass wire.

The test for soundness, to which most importance is attached, is Faija's hot water test. Two pats (about 4″ x $1\frac{1}{2}$″ x $\frac{1}{2}$″) of neat cement paste on small glass plates are, immediately after gauging, supported above the surface of water in a closed vessel. The water is kept at the temperature of 114° Fahr.; consequently the pats are subject to the action of a hot moist atmosphere. At the end of $4\frac{1}{2}$ hours they are immersed in the water, which is kept at the same temperature, for an additional 14 hours. Separation of pats from the glass, cracking, and the presence of blow holes are indications of unsoundness.

In addition to this test, two test tubes (6″ x $\frac{5}{8}$″) are filled with cement paste; one is treated in the same way as the pats, and the other left in the air. Swelling of the cement causes cracking of the tubes. In many cases a slight contraction could be noticed after a considerable time, and this could be made more apparent by putting water in the tube above the cement, when, if contraction had taken place, the water could be seen passing between the glass and the cement.

The colour of the pats, after exposure to the air, has also been noted, as well as the weathering qualities of the broken briquettes on exposure out of doors.

These tests have generally corroborated one another. Any considerable changing of colour to yellow has almost invariably been accompanied by the cracking of test tubes or of pats in the hot water test.

All the above tests have been taken into consideration when deciding as to the soundness of a cement.

The degree of fineness to which a cement is ground has, after a certain stage, little effect on the strength of neat briquettes; but with mortar briquettes, finer grinding is found to noticeably increase the strength.

In the accompanying table it will be seen that the cements giving the strongest mortars are the most finely ground. The effect of fine grinding is most clearly shown in the case of the Hunter, Taylor & Spoor (No. 15 *b*), and the Hunter, Taylor & Spoor fine (No. 16). These brands have their specific gravity, rate of setting, and tensile strength neat, practically identical, while their fineness of grinding and strength of mortar briquettes are different. A true measure of the effect is not, however, shown, because the two brands were tested with different proportions of sand, but the fine ground with three times its weight of sand has nearly twice the strength, at twenty-eight days, of the ordinary grinding with only twice its weight of sand. The same tendency is shown in the case of J. B. White & Bros. (No. 5) and J. B. White & Bros. fine (No. 10).

As an example of the degree to which fineness of grinding may be carried, attention is drawn to the Addison Potter & Son extra fine (No. 18), of which the 100^2 sieve retained only 4.10ths of one per cent. This cement, although unsound, and one of the poorest brands of English Portland tested, gives a mortar much stronger up to six months than any other cement tested with the same proportion of sand.

Tests have been made on sand delivered on the works from two localities, with a view to ascertaining the action of the finer particles in affecting the strength of mortar. The following table gives a summary of the results obtained. It will be seen that in all cases a loss of strength accompanies the inclusion of the extremely fine particles.

TABLE SHOWING RESULTS OF SAND TESTS.

Sand tests with "Clover" cement.	7 days	14 days	28 days
Sand as taken from the pit.	76	120	239
Sand retained on the 20^2 sieve.	128	256	311
Sand retained on the 30^2 sieve.	153	253	260
Sand retained btween the 20^2 & 30^2 sieves.	160	245	247
Sand that passed the 30^2 sieve.	61	114	188
Sand tests with "Burham" cement.			
Sand as taken from the pit.	125	116	131
Sand retained on the 30^2 sieve.	143	155	197
Sand retained between the 20^2 & 30^2 sieves.	151	129	172
Sand that passed the 30^2 sieve.	43	85	72
Sand tests with "Schifferdecker" cement.			
Sand as taken from the pit.	135	151	162
Sand retained on the 30^2 sieve.	169	196	214
Sand retained between the 20^2 & 30_2 sieves.	153	184	188
Sand that passed the 30^2 sieve.	148	153	156

All the above tests were made with sand from Grand Coteau.

ENGLISH PORTLANDS.

5	3·157	0·2	1·4	7 4	0-1 1½	20-30	126	186	281	277	356	403	430	451	4	....	....	....	....	....	....	44	69	83

CANADIAN NATURAL.

1	3·008	0·8	, 4·9	10·9	0-12½	0-16½	63	62	74	109	222	293	347	476	1	....	....	....	...	....	....	19	20	35
2	2·930	0·3	2·6	7·8	2-00	5-00	23	32	55	82	163	229	310	406	2	....	....	....	....	....	....	10	12	17

FRENCH NATURAL.

1	2·810	0·0	3·8	12·2	4–00	***	78	107	147	201	278	318	356	536	1	....	....	....	....	....	....	72	85	114

J. L. ALLISON,

M. Can. Soc. C.E.

ENGLISH PORTLANDS.

Numbers.	Specific Gravity.	Fineness.			Setting.		Neat Briquetts 1″ × 1″. Tensile Strength.								Numbers.	Mortar Briquetts 1″ × 1″. Sand, 3; Cem., 1 (by weight)						Sand, 2, Cem. 1		
		Per ct. Residue.			Initial.	Hard.	Days.				Months.					Days.			Months.			Days.		
		25²	50²	100²	H. M.	H. M.	3	7	14	28	2	3	6	12		7	14	28	2	3	6	7	14	28
1	3·071	0·0	7·8	21·5	***	***	455	516	584	639	675	713	746	745	18	148	181	217	267	265	266	....	....	....
2	3·100	0·0	5·7	21·2	0-16	0-29	454	506	551	581	657	727	834	758	19	126	154	184	193	230	236	....	....	....
3	3·023	0·0	9·9	27·2	0-20	0-25	356	495	566	625	663	745	806	791	16	108	128	141	178	192	223	....	....	....
4	3·076	0·5	10·2	21·7	0-16	0-30	348	497	577	665	665	659	743	725	20	96	130	147	159	210	214	....	....	....
5	3·078	0·0	11·0	26·8	0-06	0-08	408	501	569	602	622	696	729	761	9	126	144	190	203	212	206	....	....	....
6	3·068	0·6	7·6	18·2	0-17	0-27	376	502	553	657	675	629	705	710	4	153	133	147	198	188	205	....	....	....
7	3·055	0·0	16 5	33·6	0-10	0-12	275	439	539	594	687	754	795	846	2	98	118	150	143	172	199	....	....	....
8	3·090	0·3	8·6	23·0	0-15	0-18	297	450	500	618	703	656	834	797	10	90	120	137	164	186	195	....	....	....
9	3·149	0·0	3·7	18·1	3-05	***	427	427	567	595	617	645	694	704	15A	76	98	119	138	177	193	....	....	....
10	3·125	0·0	5·5	23·0	0-07	0-22	312	418	485	597	646	692	715	764	6	103	122	140	164	192	185	....	....	....
11	3·090	0·1	7·1	21·8	0-10	0-12	374	460	532	564	604	643	666	777	14	87	103	125	148	200	180	....	....	....
12	3·050	0·7	9·9	22·6	***	0-05	261	434	477	555	661	634	736	811	13	80	103	144	143	180	176	....	....	....
13	3·112	0·0	11·7	27·6	0-07	0-18	388	452	448	538	629	623	672	742	12	....	....	....	....	....	....	112	142	165
14	3·149	0·0	7·4	21·3	0-22	0-45	385	443	495	529	569	626	671	713	3	....	....	....	....	....	....	129	136	163
15A	3·133	0 0	9·1	27·7	4-00	5-25	350	414	460	518	659	644	669	747	11	....	....	....	....	....	....	116	127	151
15B	3·125	0·6	11·5	28·8	0-11	0-15	378	418	472	535	569	631	711	681	1	....	....	....	....	....	....	114	111	135
16	3·137	0·0	2·3	12·1	0 11	0-36	310	411	487	538	602	653	700	715	8	....	....	....	....	....	....	113	107	128
17	3·077	0·2	11·7	27·4	0-05	0-06	312	413	469	544	592	626	724	765	5	....	....	....	....	....	....	103	105	120
18	3·053	0·0	0·0	0·4	2-00	4-12	404	463	506	530	480	568	551	589	7	....	....	....	....	....	....	77	88	108
19	3·053	0·0	0·3	5·8	4-20	8-05	282	432	516	537	554	596	568	639	17	....	....	....	....	....	....	89	81	90
20	3·069	0·0	1·7	11·9	4-40	7-30	263	402	507	487	543	580	621	692	15B	....	....	....	....	....	....	67	59	83

BELGIAN PORTLANDS.

1	3·100	0·0	0·8	10·2	1-00	* * *	309	542	611	694	791	761	859	888	4	134	167	185	226	241	233	....	....	....
2	3·130	0·0	1·7	15·1	5-00	* * *	380	546	625	678	674	753	762	837	10	84	101	172	178	158	220	....	....	....
3	3 131	0·0	0·5	13·6	3-20	3-50	213	445	615	670	747	817	883	876	1	131	190	176	204	183	218	....	....	....
4	3·137	0·0	1·2	10·5	2-17	4-30	329	530	612	661	669	695	758	900	11	69	119	139	154	177	201	....	....	....
5	3·174	0·0	2·3	12·8	0-34	0-39	411	495	551	624	649	721	730	752	3	97	136	148	171	188	199	....	....	....
6	3·050	0·4	2·8	18·3	0-30	2-00	372	484	574	625	617	567	708	763	7	105	124	157	164	158	190	....	....	....
7	3·100	0·3	2·3	12·2	0-06¼	* * *	206	425	542	560	633	615	681	703	8	69	93	119	134	167	176	....	....	....
8	3·100	0·0	0·3	11·4	1-02	2-00	297	393	441	511	609	653	680	678	2	....	....	....	....	....	....	202	218	242
9	* * *	0·0	5·9	17·8	* * *	* * *	260	377	464	518	511	521	604	591	5	....	....	....	....	....	....	176	199	220
10	3·022	0·0	1·2	9·8	3-00	6-15	276	328	385	427	527	511	558	584	6	....	....	....	....	....	....	156	154	171
11	3·030	0·0	1·3	8·8	4-05	7-35	203	233	318	335	482	512	485	649	9	....	....	....	....	....	....	103	101	117

GERMAN PORTLANDS.

1	2·980	0·1	0·9	4·8	4-00	7-30	398	520	562	565	589	629	669	699	1	...	....	....	....	...	....	229	248	278
2	3·077	0·0	0·5	9·2	0-06	0-08	398	452	508	558	584	568	627	642	2	....	....	....	....	....	....	197	207	224

CANADIAN PORTLANDS.

1	3·076	0·1	7·5	28·5	6-17	* * *	352	495	547	646	700	750	834	848	3	87	104	108	153	151	195	....	....	....
2	3·125	0·1	5·9	21·1	0-15	1-30	335	436	497	598	617	654	684	731	1	75	104	131	145	152	178	....	....	....
3	3·125	0·1	2·4	11·5	0-14	0-35	210	347	458	387	527	530	585	660	5	34	55	48	78	100	97	....	....	....
4	3·113	0·2	4·0	12·4	1-15	2-00	212	280	330	383	436	546	595	705	2	....	....	....	....	....	....	136	133	151
5	3·157	0·2	1·4	7 4	0-11½	20-30	126	186	261	277	356	403	430	451	4	....	....	....	....	....	....	77	69	83

CANADIAN NATURAL.

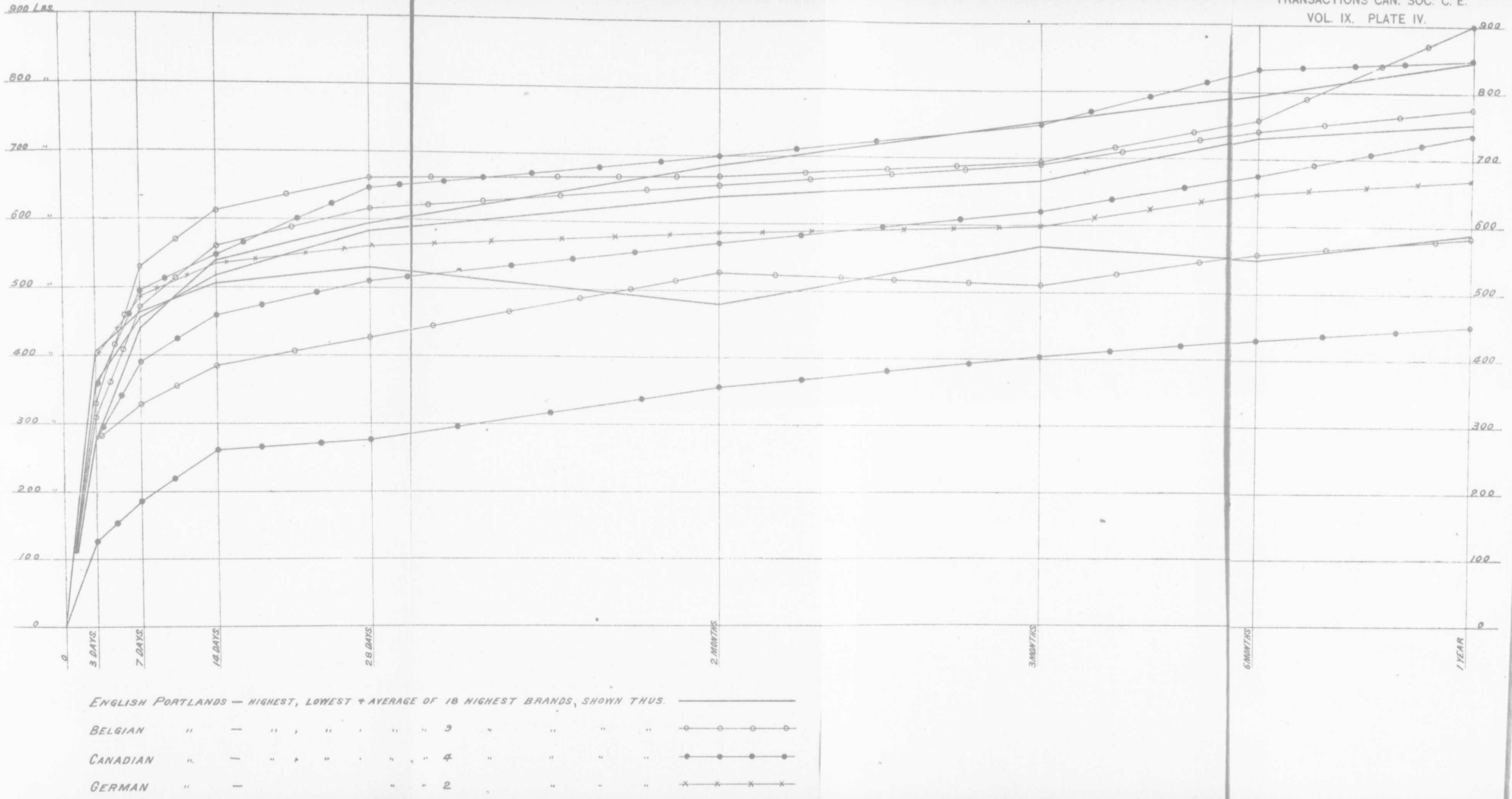
1	3·008	0·8	4·9	10·9	0-12½	0-16½	63	62	74	109	222	293	347	476	1	....	....	....	...	....	....	19	20	35
2	2·930	0·3	2·6	7·8	2-00	5-00	23	32	55	82	163	229	310	406	2	....	....	....	....	....	....	10	12	17

FRENCH NATURAL.

1	2·810	0·0	3·8	12·2	4–00	* * *	78	107	147	201	278	318	356	536	1	....	....	....	....	....	....	72	85	114

J. L. ALLISON,

M. Can. Soc. C.E.


900 LBS.
800 "
700 "
600 "
500 "
400 "
300 "
200 "
100
0
900
800
700
600
500
400
300
200
100
0
0
3 DAYS.
7 DAYS.
14 DAYS.
28 DAYS
2 MONTHS
3 MONTHS
6 MONTHS
1 YEAR
ENGLISH PORTLANDS — HIGHEST, LOWEST + AVERAGE OF 18 HIGHEST BRANDS, SHOWN THUS.
BELGIAN " — " , " " " 9 " " " "
CANADIAN " — " , " " " 4 " " " "
GERMAN " — " " 2 " " " "

Sand tests with "Schifferdecker" cement.			
St. Regis sand (not screened), 2 to 1.	181	197	232
Grand Coteau sand (not screened), 2 to 1.	149	163	172
St. Regis sand (not screened), 3 to 1.	110	135	141
Grand Coteau sand (not screened), 3 to 1.	79	98	99

FINENESS OF SAND.

Sand	On 20^2 sieve.	Bet. 20^2 & 30^2.	Bet 30^2 & 50^2.	Passed 50^2.
St. Regis.	12.2 per cent.	25.8 per cent.	51.3 per cent.	10.7 per cent.
Grand Coteau.	13.8 per cent.	29.6 per cent.	26.6 per cent.	30.0 per cent.

The accompanying table shows the results obtained from all tests which have been completed, or are well advanced, to date.

The curves show the maximum, minimum, and average strengths of the cements grouped according to the countries in which they are manufactured. In the case of the German cements, however, only two brands have been tested, and, as all three curves would fall very close together, only that for the average has been shown.

DISCUSSION.

The President. Mr. Monro, President, observed that the author deserved, in his opinion, great credit for his paper, which showed a large amount of careful and intelligent experiment and research on the subject of which it treats; and pointed out the fact that owing to the munificence of a gentleman in the city of Montreal, the University of McGill was supplied with the best means obtainable for making elaborate and continuous experiments on this and many other engineering subjects—means which were not within the reach of ordinary persons.

With reference to Thorold cement, he said that it had been exclusively used in the building of some twenty-five locks and weirs, and numerous other structures on the Welland Canal.

There were over one and one-quarter millions of bushels put in the works, all taken from a stratum of the Niagara group, about five feet thick and extending along the face of the "mountain" for several miles. This stratum was traversed by the line of the canal, and formed part of its excavations. During the progress of the works no instrumental tests were made; but the stone quarried was examined before being put into the kilns, to see that it was of the proper quality and broken to cubes of about six inches. The burning was determined by the colour, and the grinding by passing the cement through the fingers, by which method, with some practice, a very good idea was formed as to its fineness. The locks and structures were generally finished some years before being brought into use, and as the cement was very slow setting even in the open air, it had ample time to acquire full strength from age—the locks being high and dry along a side hill. He recollected that in removing a farm bridge on section 15 near Thorold, which had been built in connection with the Welland Railway in 1857, it had to be blown down, and in several cases the line of fracture passed straight through the stone and cement. The masonry could not be taken apart with wedges. This was in 1873.

The cement was considered by the late Mr. Page to be a proper article to use in hydraulic works under these peculiar circumstances; and in further proof of its fitness, when the Government decided to change the draft of the Welland Canal from 12 to 14 feet, this was partly done by raising the lock walls, etc., from No. 25 downward towards Port Dalhousie on Lake Ontario. To make proper bond between the new and old masonry, the copings were taken off, and an attempt was made to remove the frost batter of the walls by means of bars. This was found to be impracticable, as the stone and cement formed a compact mass, which had to be drilled and wedged off as if it had been a conglomerate rock.

Nevertheless, this same cement, which was also used in the raising of the locks and weirs, would have been washed out of them had not the joints been well pointed with Portland cement; because there was not time for it to set in the new work where it was used in changing an important waste weir at Allanburgh, and it was washed out of the masonry to such an extent, that when the weir was taken down and rebuilt, it had all the appearance of being laid dry. From this it will be seen that in situations where the water has to be turned on soon after the completion of the structures, natural cements such as that of Thorold are not to be relied upon, whereas a Portland cement of sound quality cannot fail to give entire satisfaction.

As to the question of cement testing, he submits a few remarks prepared at his request by Mr. J. L. Allison, Mem. Can. Soc. C. E., which clearly describes the method followed at the office of the Soulanges Canal, where over 25,000 briquettes have been made and tested by the same man. He may also state that Mr. Leedham White and other experienced persons have examined into the mode of conducting this work, and have expressed their entire satisfaction with the same. He agrees with the general conclusions arrived at by the author, and will at some future time go into further details.

Mr. Irwin said, that unfortunately he had not been able to read Mr. H. Irwin. Mr. Smith's paper at all as carefully as he would have wished, but that there were a few points he would like to discuss.

As to fine grinding, he thought that there could be no doubt as to its value. On this point, and also on the question of temperature of the water used in mixing the mortar, some light might be thrown by the action of a salt such as glauber salt (sulphate of soda). This salt, if in small crystals, say from the size of a pin's head to that of a small pea, will dissolve readily in warm water, while if powdered finely and put *en masse* into cold water, it will set suddenly into a solid lump. This would seem to shew that possibly fine grinding helps the cement to take up its water of crystallisation more easily and more rapidly, as well as to give it more capacity to cover particles of sand, and that there is probably some temperature for each kind of cement at which it will set best, and that if the water in which it is mixed is much colder or hotter, the mortar will be weaker.

He thought that the difference in amount of evaporisation for different cements mixed neat was not altogether unaccountable. The strong Portland cements probably were able to take up more water of crystallisation, being composed of a greater proportion of active ingredients, though the No. 3 Portland was not in line with the others. However, a very extended set of experiments would be needed, in conjunction with chemical tests, before any law could be established from the evaporation of specimens.

He was glad to see that Mr. Smith agreed with his previous statements as to the usefulness of natural cements for structures above water except for winter work.

He thought also, that for rapid examination, a powerful microscope would be useful, and had already alluded to its use.

He had made a few experiments on some very different samples of cement, with a view of trying to get a rapid method of testing by treating about one-twentieth of a cubic inch of cement, with $1\frac{1}{2}$ drachms of hydrochloric acid (B.P. standard), the cement being first moistened with water. All the 6 samples tested filled the acid completely with gelatinous silica, the effervescence from carbonate of lime was very marked in some cases, and a strong English Portland bubbled up as much as a poorer Canadian. Two samples smelt strongly of sulphuretted hydrogen; both of these were poor.

The poorer cements all had a large insoluble residue, and the only sample which gave a perfectly clear jelly was a very finely ground, strong, Danish cement.

An extended series of experiments in this line would probably lead to some useful results, especially if the proper proportions of cement and acid were first determined, as tests of this nature can be made in a few minutes.

Mr. C.B. Smith. Mr. Smith, in reply, wished to express his gratification on reading the many interesting and instructive discussions which his paper had brought out. This should be one of the chief aims of any paper, to draw out the opinions of practical men by which more may often be learned than from the paper itself. These discussions had brought up some points that might bear further mention.

Mr. Perley had referred to an insufficiency of time at the disposal of men in practice, who wished to judge quickly of the relative merits of a cement. This certainly would be a serious objection if it were deemed absolutely necessary to know the tensile strength of a cement at various periods ranging from 3 days to 1 month or longer; but if the writer were to range the tests in order of merit as he regarded them, he would place the blowing test first; this, as far as evidence can be adduced, is a severe test of the soundness of a cement to be used under water, and this test can be made in 1 day. The next tests should be those of fineness and specific gravity combined, which can be both made in 1 hour at most; also the times of set can be obtained in a few hours, therefore we can find out practically all that we need to know of a sample in a day. The strength is after all of relatively little importance when these three are satisfactory, although the knowledge is in itself valuable; because, if these three are up to the mark, many experiments show that the strength will be also. Surely twenty-four hours with apparatus costing from $5 to $15 cannot be considered very exacting.

Mr. Perley's remarks regarding the slaughtering of inferior brands of foreign cement on our markets should incite engineers to be more particular in their specifications, and in actually having tests made. The day is past when the brand is a sufficient guarantee of quality.

The idea of shipping in bags is not new. The American natural cements are largely shipped in 75 lb. paper bags, and the Owen Sound Portland Cement Co., if so desired, will ship in sacks; the suggestion is, however, doubtless a wise one, and would, besides, effect an actual paving of the world's store of energy.

As an authority on cement testing, Mr. Spaulding's remarks are worthy of attention, and his criticisms seem, in the main, just ones. It is probable, however, that he over-estimates the variations in results attributed to using different samples of standard sand. There is one thing on which all countries seem practically agreed, *i.e.*, that this angular quartz sand, caught between 20 and 30 mesh sieves, has very little variation, and gives uniform results. Experiments made by the author on sands of varying fineness all being, however, between 20 and 30 mesh sieves corroborate this belief.

The question of light or heavy pressure per sq. inch is not one of expense or difficulty, it is an endeavour on the part of the author, at least, to determine the *least* load which will make good 3 to 1 briquettes of uniform density with soft mortar such as the masons use. The per cent. of variation obtained in groups of 5 has been very satisfactory at 20 lbs. per sq. inch and 20 per cent. water, and more pressure would merely give higher results and lead us away from actualities where mortar sets under dead loads of only 3 or 4 lbs. per sq. inch.

The question of hot water is a very serious one, for its use is somewhat common amongst builders in cold weather. Since presenting this paper to the Society, the author has tested briquettes made of 2 naturals and 3 Portlands, which were mixed with hot water, cold water, and salt water. Both in the laboratory and in frost tests he has found that the hot water weakened the Portlands and strengthened the naturals, the reverse being the case with salt water. Mr. J. G. Kerry has made a plea for chemical analysis, and doubtless this is a very necessary thing for some one to make, but it seems probable that, as a test, it will always be confined, in practice, to the manufacturer. Apropos of this is Mr. Perley's quotation from a letter of the late Henry Faija, which will make the point clear. Mr. Kerry objects to placing any positive value on specific gravity tests, and later on he would seem to place little reliance on strength tests; but we must really cling to something. It will not do to tear down without building up. In what way are we to satisfy Mr. Perley's demand for expeditious tests and Mr. Kerry's rejections of two of those in most common use? Fineness alone is no criterion. It is necessary to specify either specific gravity or strength. It is probable that either one of them, when coupled with fineness and soundness, is a sufficient guarantee of quality.

The value of 3.10 proposed is such as will insure strength if fineness and soundness are satisfactory, because we cannot get a highly burnt cement, so over-clayed as to be weak, which will not fuse in the kiln before getting burnt to a density of 3.10.

Mr. Kerry's ideas on hot water and salt water are not in accordance with many tests, which, as Mr. Spaulding states on the authority of W. W. Maclay, is injurious in the case of hot water which the author has verified. It would seem best to leave it severely alone, whereas salt water seems to be actually a *benefit.*

In answer to Mr. Kerry's question on the strength of single bricks similar to those used in the pier tests, the average of 6 separate tests on single bricks bedded in plaster of Paris tested on their flat was as follows:—1st sign of cracks, 1210 lbs. per sq. inch; final collapse, 1860 lbs. per sq. inch.

Mr. Allison's very full exposition of the methods of testing adopted on the Soulanges canal cannot but be useful to members of the Society as embodying good practice; but when Mr. Allison goes into $\frac{1}{1000}$ths in his determination of density, he is open to the accusation of hair-splitting, for two determinations of this on the same sample will vary as much as $\frac{1}{100}$ and often more. Speaking of the Faija mechanical mixer, the author has found it to possess one weak point, the revolving vanes will drive the mortar more or less into the corner. To remedy this, an advanced scraper, throwing the mortar toward the centre in front of the revolving vanes, has been found to remedy the defect. The shrinkage of cement in a tube in air is to be expected, the most delicate determinations by the American Cement Committee showed that the soundest and best cements shrink slightly in air and expand in water.

The question of natural and Portland cements, dealt with by the President, Mr. Monro, seems to be rapidly solving itself in Canada by the construction of Portland cement works. The reason seems to be not that the natural Canadian cements are always poor, but that they are sometimes good and sometimes bad. The United States natural cement product is, on the other hand, holding its own, the reason being, probably, that the immense quantities made at a given spot allow of such thorough mixing as to give a uniform product, whereas intermittent burning of rock on a small scale is liable to produce a different quality at each "burn," depending on the exact spot from which the cement rock is taken.

Thursday, 28th February.

THOMAS MONRO, President, in the Chair.

The following candidates having been balloted for were declared duly elected as :—

MEMBER.

JOHN PATRICK O'DONNELL.

ASSOCIATE MEMBERS.

PETER FERRARA, JAMES ISAAC HAYCROFT.

ASSOCIATE.

HAMBURY A. BUDDEN.

STUDENT.

BERNARD MCENTEE.

The following was transferred from the class of Associate Member to the class of Member :—

JOHN LOGIE ALLISON.

The following were transferred from the class of Student to the class of Associate Member :—

WM. CHARLES PERCIVAL HEATHCOTE, WILLIAM MURRAY REID, ERNEST ALBERT STONE.

The discussion on Mr. Smith's paper on "Cement Testing" and on the Report of the Cement Committee occupied the evening.

Thursday, 14th March.

THOS. MONRO, President, in the Chair.

Paper No. 103.

A MICROMETER ATTACHMENT FOR THE TRANSIT INSTRUMENT, WITH EXAMPLES OF ITS USE IN SURVEYING, LEVELLING, ETC.

By W. T. THOMPSON, A.M.CAN.SOC. C.E.

The accompanying photograph represents a 6 inch reiteration transit, with micrometer attachment. The latter was constructed to my order by Mr. James Foster of Toronto, and in connection with a powerful transit telescope affords the means of measuring with great accuracy small vertical angles between the limits of 0″.8 and 3 degrees.

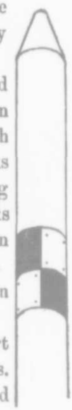
It consists of a metal box firmly attached to the vernier plate of transit in a plane at right angles to the horizontal axis of telescope, and containing a micrometer screw, with divided head and vernier, and two movable nuts N and I. The former has 40 threads to the inch, and bears against the vertical clamping bar B, being kept in close contact by the spring S.

The head of screw is divided into 100 parts, and is read by the vernier V to the $\frac{1}{1000}$th part of a revolution, and as each complete revolution moves the nut N through $\frac{1}{40}$th of an inch, the $\frac{1}{1000}$th part will move it through the $\frac{1}{40000}$ th of an inch, and as the length of the clamping bar B from centre of axis to point of contact with nut N is $6\frac{1}{2}$ inches, this will move the telescope through an angle of 0″. 8, which is the smallest that can be measured with this micrometer.

The index nut I is for recording the number of revolutions made by the screw; it has 20 threads to the inch, and the edge of box is divided into 20 parts to an inch, so that each turn of the screw carries the index nut through one division; therefore, in making any observation, the number of complete revolutions is read off from the scale, and any fractional part from the divided head and vernier.

The clamping bar B consists of two parts so arranged that the telescope may be moved in altitude either by the micrometer or by the ordinary tangent screw T, so that when desired the micrometer may be set at zero or any reading, and the telescope accurately set on any object by the tangent T.

In measuring distances with this micrometer, the writer has used for a base a light round rod 30 links in length, about 2 inches in diameter at the bottom, tapering to 1 inch at the top, and provided with a universal spirit level to ensure verticality, with 3 targets, one 5 links from the bottom, one 10 links above this, and one at top of rod, giving a clear distance of 25 links between the outside targets. The targets were formed of bright tin and black rubber tacked on the rod, as shewn in the margin.



The tin reflecting light and the black rubber absorbing it, the division between them was very distinct, specially in winter.

The lower targets 10 links apart were only used in measuring short distances, the outer targets 25 links apart being used in all other cases.

If a distance of say 40 chains be measured on a piece of level ground or upon the ice, and the number of turns of the micrometer screw required to move the horizontal wire of the telescope from one target to another be denoted by n, then as the base is very short as compared with distances to be measured, it may be considered to represent the arc which subtends the angle at the instrument, and this angle will vary inversely with the radius or distance. Therefore at one chain the number of turns of the screw would be represented by $40\,n = N$. If now the rod be held at any unknown distance denoted by X chains, and the number of turns of the screw is observed $= n'$ then $X = \frac{N}{n'}$ where the base subtending n' is very small as compared with its distance from the instrument, and the effect of differential refraction is assumed to be constant.

As, however, at different distances from the instrument the difference of refraction of the targets will vary slightly, it is necessary, in order to prepare an accurate table for reducing the observed readings to distances, to note the actual readings at each chain of distance from 5 chains up to 40 chains, and interpolate the readings for differences of 10 links. The distances corresponding to any observed readings can then be at once obtained by inspection. The condition of the atmosphere at the time should be noted, and on different days, if one or two distances are chained, and the observed readings compared with those given by the table, we shall be able to apply approximate corrections to the tabular distances due to different atmospheric conditions.

The horizontal wire of the telescope should be very fine, and the object glass and eye-piece must be very carefully focused. It is also important that the axis should be secured with moderate pressure in the Ys, and to obviate as far as possible the tendency to rise, the spring S must be slightly bent so as to grip the stud against which it bears.

The telescope used has an objective of 1.5 inches clear aperture and 10.5 inches focus, and the eye-piece a magnifying power of 32 diameters.

With this instrument and the 25 link target rod described, distances up to 30 chains may be measured, with an error seldom exceeding $\frac{1}{2}$ link per chain, and with a more powerful telescope it is probable even closer results could be obtained.

We shall now give some examples of the use of this attachment in surveying and engineering operations.

I.

A method of traversing with the transit and micrometer attachment.

In regard to traverse surveys, the Manual of Survey for Dominion Lands provides as follows:—

"The use of the micrometer for such work will be allowed, provided that the closing error does not exceed one chain in one hundred chains. The micrometer must be of an approved pattern, and must be submitted to the Surveyor General before being used on the survey."

The micrometer attachment described in connection with the transit affords the means of making traverse surveys with great facility.

The method used by the writer is as follows: the instrument being set up on the shore of a river or lake, and either on one of the survey lines or at a point fixed in position with reference to the same. It is carefully levelled, and the horizontal circle reading for the north point noted. Then the rod-man proceeding along the shore holds the rod at all points where marked deviations occur, the position of each point being fixed in direction and distance from the instrumental station, by readings of the horizontal circle and micrometer. At suitable points new stations are taken and the survey continued in the same manner. The notes are entered in the field book under the following headings, and written from the bottom upwards, the topography being shewn in margin. If a repetition instrument is used, the two columns headed H.C.R. and H.C.R. on N are not required.

Stn.	H.C.R.	H.C.R on N	Azimuth	Mic Readings S — L	Distance	Remarks.

It is convenient to have the rod-man travel uniformly from left to right, viz., in the direction given by the hands of a watch, and any topography will then be shewn in left hand margin.

If the initial station be called O, then the points fixed from it may be conveniently designated O_1, O_2, O_3, etc., O to 1: 1_1, 1_2, 1_3, etc. The reduced notes are placed in three columns, under the headings, Station, Azimuth, Distance, and from this data the points are plotted on a scale of 20 chains to an inch, and the shore line defined by joining these points.

No matter how irregular the shore line may be, a perfect representation of it can be obtained by this method, and in much less time than would be required by the system of chained survey lines and offsets.

Regarding the areas of the broken quarter sections, they may be readily calculated from the above data; but it may be stated that as a water boundary is a variable one, depending on variations of the water level, extreme accuracy in determining these areas is generally not necessary, and in many cases the planimeter or some graphical method will give sufficiently close results, especially when the shores are flat and the water line subject to wide fluctuations. The plot in all cases being carefully made.

II.

To determine differences of level and establish grades on preliminary railway and other surveys.

The telescope must be provided with a good spirit level, and the horizontal wire adjusted to define a horizontal line when the bubble is at zero.

Then (in the same manner as with the gradienter) if we note the point on a rod at the distance of say 500 feet where this line strikes, and turn the micrometer screw through one revolution, the distance between the two points on the rod being measured, one-fifth of it is the rise or fall in 100 feet for one turn of the screw, and we can now prepare a table giving the number of turns required for various grades, also of the rise or fall in feet at different distances. These tables should include the effect of curvature and refraction.

We also require a target rod consisting of two pieces sliding upon each other, as shewn in margin, in order that the piece carrying the targets may be pushed up or down, so that the lower target can be set at the height of the telescope above the ground, and clamped in position. The distance between the outside targets may be five or six feet, and a table for

reducing observed micrometer readings to distances can be prepared in the manner already described.

We are now prepared for surveying and obtaining the levels and distances along any preliminary line.

The mode of proceeding will be as follows: The instrument being set up at the starting point of the survey, and carefully levelled, the direction of the line is fixed by readings of the horizontal circle, the bubble of telescope level brought to zero and reading of micrometer noted; then the lower target being adjusted to the height of the telescope, the rod-man proceeds along the line and holds the rod at all points where any marked changes of inclination occur, the distance to each point being determined from readings on the targets, also the difference between the micrometer reading for level zero and the reading on the lower target gives the difference of level by consulting our table.

We may also in open country obtain the direction, distance, and difference of level of points on either side of the line referred to the Instrumental Stations, and without planting any stakes except at these stations, collect the necessary data for preparing a plan, profile and cross sections of the line, from which a location can be decided on, which would then be chained, staked and levelled in the usual way.

III.

A very important use to which this attachment can be applied is the determination of the latitude by measuring small differences of zenith distance of North and South stars by a method somewhat similar to that by the zenith telescope.

For this purpose a very sensitive spirit level must be attached to the vertical clamping bar B in a plane at right angles to the horizontal axis of telescope, and the bubble should be adjusted to read zero when the index nut I is at the centre of the scale; this level should read to say $3''$ for one *mm* space, so as to readily show a displacement of $\frac{1}{2}''$. The time, azimuth, and approximate latitude may be readily obtained from observations on Polaris and another star in the same vertical plane.

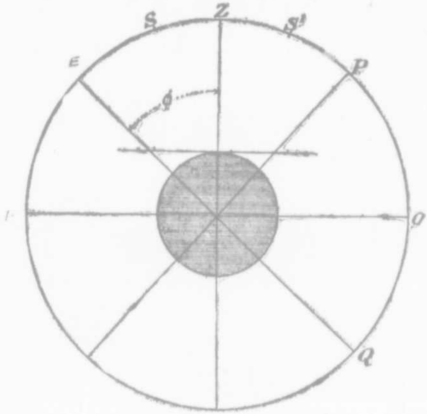
Then with the approximate latitude or declination of the zenith point, we select from a Star Catalogue, such as the Berliner Jahrbuch, a pair of stars between the 2nd and 5th magnitudes, which culminate as nearly as possible at equal distances to the north and south of the zenith, and within say 30 degrees of it, differing not more than two degrees in zenith distance, nor more than say 30 minutes in right ascension. The telescope, it may be stated, is provided with a diagonal eyepiece with powers of 30 and 60 for star work.

The observer should be supplied with a chronometer or watch adjusted to sidereal time.

Shortly before the time of transit of the first star the telescope will be brought into the meridian plane by readings of the horizontal circle, the vertical finding circle set for the mean zenith distance of the two stars and bubble brought to the centre of its run by inclining the telescope. The latter will now be securely clamped by the screw K, so that the relation between the telescope and clamping bar B with its attached latitude level will thereafter remain unchanged. The latitude level will then be brought exactly to zero by turning the micrometer screw, and reading of same noted; the screw will then be turned to the right or left, according as it is necessary to depress or elevate the telescope, to set it at the zenith distance of the star, and when it appears in the field, the horizontal wire will be set upon it, and a precise bisection made when it reaches the middle wire; the micrometer reading will then be noted, the screw reversed and level again brought to zero, the micrometer reading again noted and mean of the two readings taken as the true reading for level zero at the instant of the star's transit. The instrument is then turned 180°, in Azimuth, and similar observations taken on the other star.

With this micrometer, a right hand motion of the screw will increase the readings and zenith distances. If, therefore, we denote the reading on the star nearest the zenith by m and the reading for level zero for same star by m_o, then the arc measured by the micrometer is represented by $m_o - m$; and if we denote similar readings for the other star by m_1 and $m_{o\,1}$, then the arc measured will be represented by $m_1 - m_{o1}$; and the sum will represent the total change of inclination of the telescope, or difference of apparent zenith distances $= m_1 - m + m_o - m_{o\,1}$ which must be reduced to seconds of arc by multiplying by R the number of seconds in one revolution of the screw; this will be determined from observations on Polaris near its elongation, or by measuring the difference of declination of close stars at their transit over the Meridian; the value will vary slightly with the number of turns, and should be tabulated for different intervals. Then using the value corresponding to the observed interval, we shall have for the apparent difference of zenith distance $\pm (m_1 - m + m_o - m_{o\,1})R'' = (z - z')$, in seconds of arc, where z denotes the apparent zenith distance of southern and z' of northern star.

In the diagram let P denote the North Pole, Z the Zenith, EQ the Equator, S the Southern, and S′ the Northern Star; S E and S′ E = s and s' their declinations; Z S, and Z S′, their true zenith distances = Z and Z′, and r and r' their refractions.



Then denoting the latitude Z E by ϕ. We have $\phi = (s + Z) = (s' - Z')$. Therefore $2\,\phi = s + s' + Z - Z'$, and since $Z = z + r$, and $Z' = z' + r'$, inserting these values, our formula becomes $\phi = \left(\frac{s + s' + z - z' + r - r'}{2}\right)$ and inserting the value of $z - z'$ as measured by the micrometer, the final formula is $\phi =$

$$\left(\frac{s + s' + r - r'}{2}\right) \pm \left(\frac{m_1 - m + m - m_{01}}{2}\right) R''$$

in which the sign of the second term is the same as that of $(z - z')$, viz., if the southern star has the greater apparent zenith distance it will have the + sign, and *vice versa*.

By consulting a Star Catalogue it will be seen that in most latitudes several pairs of stars between the 2nd and 5th magnitudes, and differing not more than 30 minutes in R, A, nor more than 2 degrees in zenith distance, would be available for observation with a good transit telescope.

This method might be found useful in determining latitudes in exploratory surveys, in connection with micrometer work, and should give the latitude within 3″ or 4″ by combining the results of several observations.

DISCUSSION.

Mr. G. W. McCready.

Mr. G. W. McCready said, on receiving a few weeks ago an advance proof of "A Micrometer Attachment for the Transit Instrument, with Examples of its use in Surveying, Levelling, etc.," by W. T. Thompson, A.M. Can. Soc. C.E., he became interested in reading Mr. Thompson's description of the instrument, and of the uses to which it might be applied.

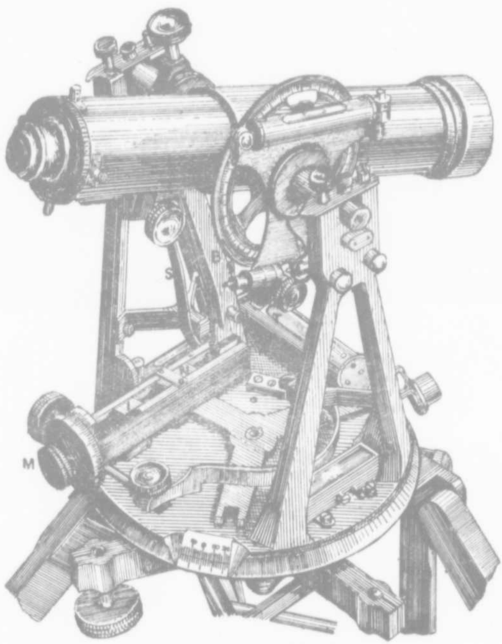
It suggested to the writer a very simple device upon which he experimented many years ago, for the purpose of measuring very minute angles, either horizontal or vertical. The instrument which the writer used was a good achromatic pocket-telescope, with compound eye-piece of about 5 or 6 inches in length. Having this firmly set up on a stand, for the purpose of making a close inspection of objects on a distant mountain, he removed the eye-piece, and again inserted it just so far as to have a hold within the tube; in which position he could move the outward end either horizontally or vertically, perhaps 4 or 5 degrees from its normal direction, and still have a good view through the glass. Having spider-lines in the telescope, the writer noticed how slowly and regularly they appeared to move over the object as the eye-piece was inclined either way,—a deviation of probably a degree or more being required to produce 1′ in the angle of sight. This led him to devise a graduated arc, with index and vernier, or micrometer arrangement,—not merely to read the deflection of the eye-piece, but the exceedingly small angle subtended by the distant object over which the sight appeared to move.

This being a very old contrivance, as above stated, Mr. Thompson's paper has suggested that further experiments might be made to determine whether there is anything in it which can be made practically useful.

Mr. H. Irwin.

Mr. H. Irwin said he thought that it would be better not to attempt any such fineness of measurement as an angle of 0.8″ with a 6 inch transit, this angle subtending only one-seventh of an inch at 3000 feet.

He thought that all who have used a 6 inch transit would agree with him that it was impossible to set the cross-hairs to anything much


B
S
M

closer than an inch at such a distance, which would correspond to about five seconds of arc. He had a very heavy English six inch transit, which worked more accurately than any instrument of its size he had ever seen ; it was graduated to read to 20 seconds of arc, and always gave the same angle between two pickets, and he thought it was good enough to read to 10 seconds of arc, but considered that that was the limit of accuracy for a 6 inch transit.

He said that no doubt the vernier of Mr. Thompson's micrometer read to 0″.8 of arc, but the errors in his micrometer screw probably amounted to 2 or 3 seconds of arc, so that it would have been better to use a coarser micrometer screw which could be more accurately made.

He noted that Mr. Thompson stated that distances up to 40 chains could be measured with a 25 link target rod to within half a link per chain or one-half of one per cent., which amounts to stating that the error in reading the 25 link rod would be about one-half of one per cent., or about one inch, which would correspond to an angle of about seven seconds at 40 chains, so that it would seem quite sufficient to have the micrometer arranged to read to 5 seconds of arc, and not to attempt anything so fine as 0.8 seconds. He also thought that it was useless to attempt to read a displacement of $\frac{1}{2}$ second with a level set on a 6 inch transit, as he had an eighteen inch level which had a bubble ground to about five seconds of arc to each division of about one-eighth of an inch, and he found it so sensitive that it was almost impossible to keep the bubble from moving constantly.

He thought that a bubble reading to 3″ for one millimeter space or about 9″ to $\frac{1}{8}$th of an inch was a very fair arrangement, but did not see how it could be depended on to show displacements of half a second when it is remembered that that minute angle is subtended by only one-eighth of an inch at 4,000 feet.

He thought that Mr. Thompson's attachment was somewhat similar to the gradienter, but was better in so far as it had a longer arm and was more firmly attached, and being much interested in instruments was much obliged to Mr. Thompson for bringing his arrangement before the notice of the Society.

He thought that the weak point of all such arrangements lay in having to shift the instrument in reading the two ends of the rod. With stadia hairs this movement is avoided, and he would be glad if Mr. Thompson could give any comparison between the work done by the two methods, as he understood that Mr. Thompson had many years' experience in accurate instrumental work.

Mr. W. T. Thompson.

Mr. Thompson in reply said that the form of attachment described by Mr. McCready was quite new to him.

In reply to Mr. Irwin he said that he was obliged to him for his investigation of the instrument, and that the limit of accuracy of the usual form of 6 inch transit, as stated by Mr. Irwin, was in accordance with his own experience.

He would, however, remark that the accuracy of micrometer measurements by the method described depends upon the power of the telescope and the steadiness of the stand upon which it is mounted, and both these elements are to a great extent independent of the diameter of the horizontal circle of transit. In the case of the reiteration transit the tripod is of a special construction, being of the trussed form, with a broad head upon which the three foot screws rest in grooves, the distance between the bearing points being 5.6 inches, also the instrument having only a single centre has much greater steadiness than one with a compound centre, and the whole forms a very firm stand for the telescope; the latter also is much more powerful than those usually employed, having an eye-piece magnifying 60 diameters for star work, and in observing the transit of a star under very favourable conditions a change of one division of the vernier or 0″.8 is perceptible, and two divisions or 1″.6 in sighting on a fine terrestrial mark about 20 chains distant. A star being a fine bright point of light without appreciable dimensions, the wire can be set upon it with much greater accuracy than on a terrestrial mark. In order to obtain close results under general conditions, however, the power of the telescope should be increased so as to make its pointing power equal to the lowest vernier reading of micrometer, and this would be effected by using an objective of about 2 inches aperture and 12 to 13 inches focus, so that magnifying powers of from 60 to 80 diameters could be employed to advantage. 0″.8 is certainly a very minute angle, but in the determination of latitude it represents a distance of about 80 feet, and is therefore not too small to be considered. If the micrometer was to be used only for the measurement of distances, however, it would no doubt be an advantage to use a coarser screw as suggested by Mr. Irwin; but in regard to the accuracy with which such screws can be made, he would say that screws with 100 threads to the inch are made with almost perfect accuracy for use with astronomical instruments.

Regarding a displacement of half a second of arc the author's meaning is that a change in the position of the bubble of that amount could be seen on the scale; so that in determining the readings for level zero the bubble could be brought to the same position each time with an error not exceeding $\pm 0''.5$.

Regarding the steadiness of the bubble this depends upon the form of transit and construction of tripod upon which it stands, as well as in setting up the instrument so as to be as free as possible from surface vibrations. For example, in setting up the transit for close latitude observations, small pits would be dug so that the feet of tripod would stand upon the firm subsoil, and any movement of the observer would then not be communicated to the instrument. When set up in this way the author has found the bubble of latitude level having a value of $5''$ to the sixteenth of an inch to remain quite steady for a considerable time on his instrument.

Regarding the principle of its construction, the author would point out that this form of micrometer is quite different from the gradienter, the divided head having a motion of rotation only; the use of a vernier admits of very close readings being taken. It also differs by the use of an index nut for recording the number of revolutions and of a sliding nut for moving the telescope.

Regarding the use of stadia wires, the author has not had a very extensive experience; he has found them useful for short distances as a check on the calculation of triangulations where only a single distance is to be determined; but where a number of points are required to be fixed from the same station, as in the method of traversing described, they would, in his opinion, be unsuitable, as the rod man would have to be depended on to record the length of base; he therefore thinks it preferable to use a constant base, especially when the angle subtended by it can be accurately measured, which can be done with this form of micrometer, provided the pivots of the telescope are secured so that they cannot shift when the micrometer screw is rotated.

Thursday, 29th March.

WM. KENNEDY, JR., Member, in the Chair.

Paper No. 104.

AN APPLICATION OF THE STONEY PATENT SLUICE TO RIVER IMPROVEMENTS.*

BY G. E. ROBERTSON, B.A.SC., M. CAN. SOC. C.E.

Among the more important contributions of late years toward the improvement of canal works is the Stoney Patent Sluice.

This invention renders it possible to raise, by a small expenditure of power, a counter-balanced vertical bulkhead of steel, of unusual dimensions and with a heavy head of water against its face. The bulkhead bears against rollers set in a moveable frame, and the friction which would otherwise result from the immense pressure is thereby reduced to a minimum.

Under certain conditions of river improvement it has occurred to the writer that these sluices might be employed in such a way that the usual form of lift-lock could be dispensed with. The conditions chosen as an example are as follows: A rapid, in an otherwise navigable river with a fall of ten feet in about a mile in length.

Referring to the sketch, it will be seen a bank is formed on one side of the river for the entire length of the rapid, to form a canal.

At intervals of about half a mile three pairs of gates are placed dividing the canal into two reaches. At each end of each reach are Stoney sluices connecting directly with the rapid. A vessel ascending passes through the first pair of gates, which are then closed, and as she proceeds through the first reach, the sluices at the upper end of it are opened and the water enters from the rapid, raising the reach to the level of the water half way up the rapid.

The second pair of gates can then be opened, and the vessel passes into the second reach, which is raised in a similar manner by opening the sluices at the upper end, connecting with the river at the head of the rapid.

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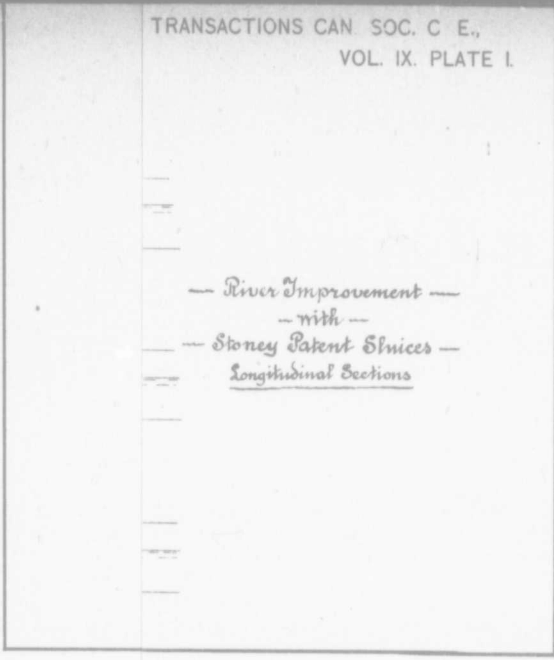
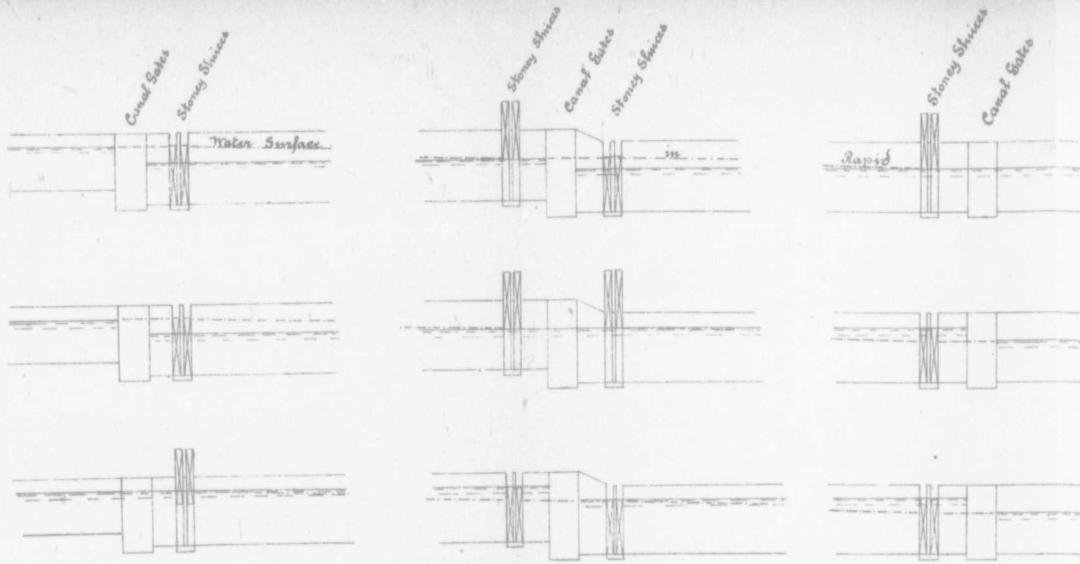
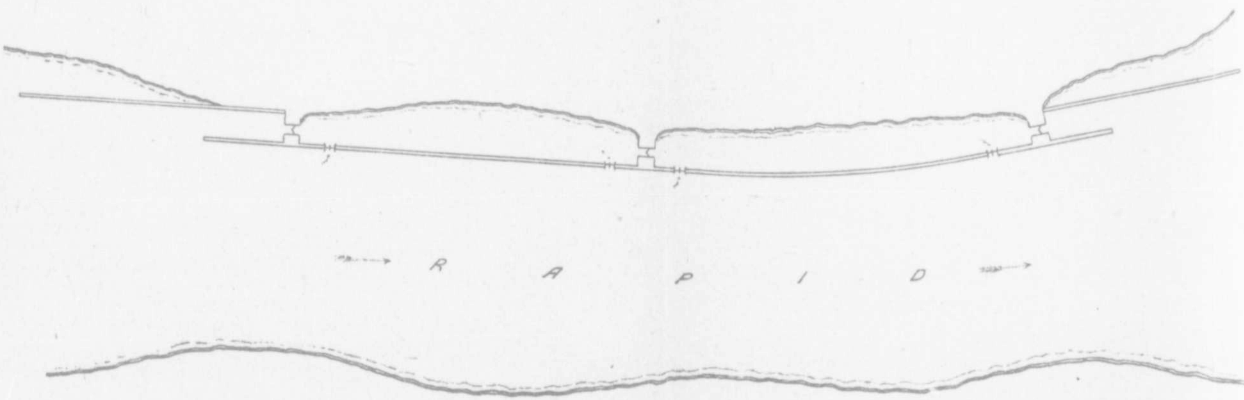
* See plate I.


— River Improvement —
— with —
— Stoney Patent Sluices —
Longitudinal Sections


— Plan of River Improvement —
— with —
— Stoney Patent Sluices —



— River Improvement —
— with —
— Stoney Patent Sluices —
Longitudinal Sections



— Plan of River Improvement —
— with —
— Stoney Patent Sluices —

The third pair of gates can then be opened, and the vessel continues on her way.

The advantages attending this scheme of river improvements may be set forth as follows:

Vessels may pass through the canal without stopping, at the usual rate fixed for canals; this, particularly in case of long tows of barges, would shorten the time of passage very much.

Vessels would never be near a canal gate when there was a head of water against it; the dangers attending the usual kind of lockage, the bringing of large vessels to rest within a few feet of closed gates, as well as the damage done to shipping, would be done away with.

The head of water against banks and structures would never be more than a few feet, thereby lessening the cost of construction.

The length of vessels is not limited.

Under favourable conditions there would be a saving in cost of construction, principally in masonry.

The reason for placing the sluices between the canal and the river is that each reach may thus have an independent supply of water; but when this system is applied in its simplest form, that is, with only one reach and a pair of gates at each end, it is then possible the sluices would be placed on the landward side of the gates, which would have the advantage of more accessible foundations, but the disadvantage of causing currents in the entrances.

In cases where the whole discharge of a river is controlled, the adjoining reaches, providing they are not of too great an area, might be brought to the same level at suitable intervals of time by means of these sluices, permitting the passage of vessels up or down without the intervention of lift-locks.

Thursday, 11th April.

THOMAS MONRO, President, in the Chair.

The discussion on Prof. Bovey's paper on "The Strength of Canadian Douglas Fir, Red Pine, White Pine and Spruce," and on Mr. Smith's paper on "Cement Testing," occupied the evening.

Thursday, 25th April.

John Kennedy, Past President, in the Chair.

Paper No. 105.

THE BARRIE FLOOD OF 1890.

By Willis Chipman, M.Can.Soc.C.E.

The town of Barrie is situated on the northwest corner of Kempenfeldt Bay, an arm of Lake Simcoe, having a width of one mile opposite the main part of the town, the depth at the centre varying from 50 to 100 feet, but increasing to the eastward. The land to the north and to the south rises to a height of from 200 to 170 feet, extending to the west, forming a valley about $1\frac{1}{2}$ miles wide, which continues to the Nottawasaga River; the highest point of the valley being only 65 feet above the Bay. This valley may have been at some period in geologic time the outlet of Lake Simcoe. Around the head of the Bay are several small spring brooks, the one with the largest drainage area discharging into it near its northwest corner. During the summer months this stream is apparently smaller than some of the others to the south, but, having a larger drainage area, the flow during rains and when the snow is melting is much greater.

The total drainage area of this stream is about 1200 acres, or less than two square miles, of which about 1,000 acres or one and one-half square miles is north of and outside of the built up portion of the town, or say northeast of Peel st., this 1,000 acres being entirely cleared farm land. The external limit of the water-shed is approximately a circle one and one-fourth miles in diameter, the rim of which has an elevation of 170 feet, that of the centre of the depression being 110 feet, and the outlet at Peel st., 61 feet above the Bay. From the point where the water course crosses Peel st., the first built up street of the town crossed by it, along the stream to its outlet, is about 5,000 feet. For more than half of this distance the bed of the stream is dry during the greater part of the year.

When Barrie was first laid out for a town, this stream flowed westerly from Peel st. to Ross st. in a tortuous channel through a swampy tract of land lying between the high terrace, a little to the north, and a remarkably narrow and high gravel ridge extending westerly from Mulcaster st. to Bayfield st. After being cleared, this low tract was a skating pond in winter and a wet marshy place in summer. When Sophia st. was laid out, the water course was straightened in such a way as to confine it north of the street, all the southerly bends being cut off and filled up. The distance from Peel st. to Ross st. is approximately 2,600 feet. For about half of this distance the bed of the stream was north of Sophia st. on private lots, and for the other half of the distance, being from Owen st. to Bayfield st., it was along the north side of the roadway. The fall in the bed of the stream from Peel st. to Ross st. was found to be 25 ft., but for half of this distance the fall was only one in two hundred.

From Ross st. the stream flowed southerly 700 feet through the town park, with a fall of 24 feet, then southerly and south-easterly, crossing Park st., Toronto st., Elizabeth st., and Mary st. to the Bay, a distance of about 1,600 feet, with a fall of 11 feet.

In grading Clapperton st. the high gravel ridge was cut through and removed to the full width of the street. Immediately west of Owen st., the ridge was also removed on several town lots. In 1846 the stream overflowed its banks, and ran down Clapperton st., washing out a channel which was not filled in for several years.

In 1860 another overflow took place, after which the channel was straightened along Sophia st., and the roadway raised to form an embankment about three feet high above the bed of the stream. About 1870 a timber drain 3 feet wide inside and 4 feet high was built from the bay to Sophia st. along Clapperton, the idea being to relieve the stream in time of freshets. The northerly portion of this drain collapsed in 1886, and was replaced by an 18 inch tile pipe for 500 feet.

The top and sides of this drain were of three inch planking laid longitudinally and spiked to bents placed $3\frac{1}{2}$ ft. centres. The bottom of the drain was of two inch planking. Each bent was built of four pieces of timber top and bottom $5'' \times 8''$, verticals $5'' \times 8''$, joints halved and spiked. The planking was outside of the bents. There was no way of inspecting this old drain except by walking up it from its outlet. When the stream at its head raised to about half the height of the Sophia st. roadway, a portion of the water discharged through the 18 inch pipe into the old drain.

A number of culverts were constructed in the town along the course of the stream, the cross sections varying from 10 to 40 square feet, the average being 20 square feet. Several of the longest culverts were on private property, the street culverts as a rule being of superior construction to the others.

All those street culverts and culverts on private property were of wood, generally with sides of square timbers laid horizontally, rag-bolted together or anchored back at intervals in height to resist side pressure, or occasionally braced across inside. All were covered with timber or planking. Some few of them were built of round timber in whole or in part.

With no town engineer to advise the town authorities, and with immunity from damage from flood for 30 years, the water course had been neglected, the street culverts had not been cleared out thoroughly, the open channel or ditch along the north side of Sophia st. had become a dumping ground for old tinware, old boots, etc., while the Council had permitted parties to cover the stream on private property.

These private culverts were not inspected when built or afterwards; they were irregular in size, crooked in alignment and in shape, and were not repaired or cleaned out except as each owner or tenant might please. Floating boards, timbers, blocks, brushwood, grass, etc., became lodged in the bends and irregularities in the private culverts, at points where they could not be seen.

During the first week in June, 1890, the corporation labourers were repairing some break in the old timber drain on Clapperton st., and had an opening made in the street near Worsley st. for this purpose.

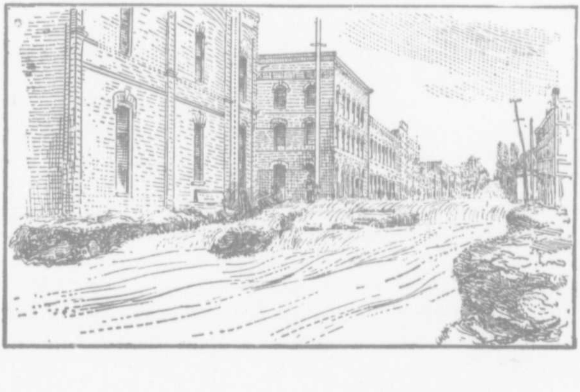
On Thursday, June 5th, 1890, the day of the Provincial Elections, an unprecedented fall of rain occurred in the vicinity of Barrie as well as in different portions of the central parts of the Province of Ontario. The heavy rain was not uniformly distributed, as in some places, not five miles from Barrie, there was no rain to speak of. The downpour commenced about 2 o'clock in the afternoon of the 5th, increasing in intensity until 3 o'clock. The rain was accompanied by heavy thunder and lightning, and the town was in almost total darkness during the heaviest showers.

At 2.15 a pond of water had formed northeast of Peel st., reported as covering about 10 acres.

This pond could not have been more than 500 feet in length, about 150 feet in width at Peel st., and about 6 feet deep at its deepest point. From these dimensions it is evident that the pond could not have covered 10 acres, but it may have covered three, and the average depth could not have been more than two feet.

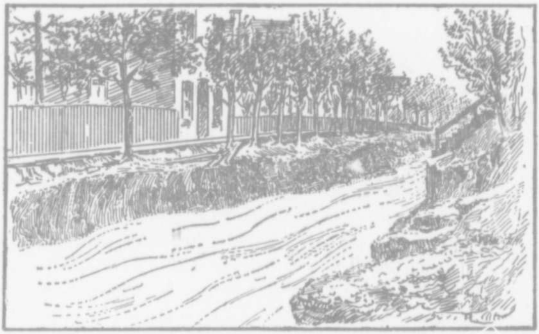
It is more than probable that the Peel st. culvert became blocked, as the water rose nearly or quite to the surface of the roadway. The roadway broke away about 2.20 p.m., and it is probable that the channel of the stream below this was about filled by this time by the drainage from Sophia st. and the lands to the north of it.

About 2.30 the stream overflowed Sophia st., for nearly the total distance between Owen st. and Bayfield st. some 1,000 feet, filling cellars and basements and invading the floors of dwellings, the Central Public School, and the business portion of the town on both sides of Dunlop st. from west of Bayfield st. to east of Owen st., a total distance of about 1,300 feet. The post office and railway station were surrounded by the flood, the water rising above the ground floor of the latter.



The flood down Clapperton st. entered the old box drain at the point where the workmen were repairing it, and in a few minutes a torrent was rushing through it. The street was washed out in places from curb to curb from Worsley st. to the bay, the depth at Worsley st. being 16 feet.

The detached residences and the fences along the south side of Sophia st. so obstructed the current that the greatest flow was along the streets.



Upon reference to the plan, it will be seen that Clapperton st. is 2 or 3 feet lower than Owen to the east or Bayfield to the west, also that the fall from north to south is much greater than at right angles. The gravel ridge before mentioned formed wing walls to concentrate the flow down Clapperton below Worsley. The streams down Bayfield and Owen also did some damage by filling basements and cellars, and by entering on the ground floor of the Public School. The ground floor of this building at the rear is but little above the ground level, but at the front there are several steps from the ground to the floor. The street in front is 40 feet above the Bay and only 800 feet distant, the inclination, therefore, being 1 in 20, and uniform.

It is not surprising that a panic ensued among the children and teachers. The darkness, the lightning and the sudden inrush of a torrent of water at that height above the bay would be sufficient to make any heart quail.

The children were rescued by the fire department with some difficulty, as the velocity of the current was such that the smaller children could not in their frightened condition make headway against it.

On Clapperton st. below Worsley the greatest damage was done.

The old box drain collapsed, was washed out or filled up. For several hundred feet it was not only destroyed, but nearly every trace of it was removed, and the debris washed into the bay. The depth excavated by the torrent at Worsley st. was about 16 feet, decreasing to 7 feet at Collier and 4 feet at Dunlop.

From Worsley to Collier the height of the water did not exceed one foot above the surface of the ground, but at Dunlop st. the flood mark was fully 3 feet above the sidewalk.

From Sophia st. to Worsley st. the surface inclination of Clapperton is 1 in 100; Worsley to Collier 1 in 25; Collier to Dunlop 1 in 50 and below; Dunlop about 1 in 50.

A stream of water a foot deep, 66 feet wide, and flowing down an incline of 1 in 50, would be considered a large stream. Clapperton st. proper ends at Dunlop, meeting Bayfield at an acute angle. The force of the stream struck the west side of Bayfield below Dunlop, completely demolishing one rough cast house, from which the occupants barely escaped with their lives. The total quantity of earth removed from Clapperton st. by the flood was approximately 7,000 cubic yards. The earth, timbers and debris were deposited along the railway tracks and carried into the bay. Cars were shifted on the tracks by the flood, the tracks undermined, and traffic suspended for some time.

Great difficulty was met with by the writer in securing satisfactory detailed descriptions of the flood on Clapperton st., as all who witnessed it were too interested in their personal safety, or in the saving of life and property, to make many observations of scientific value. The reports of the time at which the flood first rushed down the street and the time it ceased, as given by eye-witnesses, varied most unaccountably, and the reports in the newspapers were of little value except in itemizing the damages done.

The following facts are, however, to be relied upon:

1. The rain began early in the afternoon, the heavy rain beginning at 2 o'clock.

2. The pond above Peel st. broke away between 2.15 p.m. and 2.30 p.m.

3. The culverts between Bayfield and John became blocked, causing the stream to overflow Sophia st. about 2.30 p.m.

4. Very heavy rain continued until 3 p.m., the heaviest downpour occurring at this time.

5. The greatest flow down the street was about 4 o'clock.

6. The creek continued to overflow its banks on Sophia st. until 5 p.m., if not longer.

7. Dunlop st. and Collier st. were flooded from Clapperton to Owen for two hours.

8. The pond formed above Peel st. had an area of about 3 acres, with a maximum depth of 6 feet.

9. The water course was practically dry at noon on the day of the flood.

As previously stated, the drainage area of the stream does not exceed 1,000 acres above Peel. The average slope of the surface of the ground in this area towards the lowest point is 1 in 50.

The following is the reported rainfall for the 3rd, 4th, 5th and 6th June, as given by the Head Master of the Collegiate Institute, who had for many years acted as observer for the Meteorological Department:—

	Tue. 3rd.	Wed. 4th.	Thur. 5th.	Fri. 6th.
Temperature, Max.		98		
" Min.			62	
Rainfall, inches	0.54	1.24	2.90	0.05
" duration	2 hrs	6 hrs	5 hrs	$\frac{1}{2}$ hr

In 4 days, rain fell to the depth of 4.73 inches.

The observations were taken at a point about three-fourths of a mile easterly from the town hall, the observer being furnished with an ordinary surface rain gauge.

The rainfall, as reported by a corporation employee, was much greater than this, and his statements were corroborated by other witnesses.

The following is his report :—

" Wednesday afternoon and night $2\frac{1}{2}$ inches of rain fell.

"Thursday, 5th June.	1.45 p.m. to 2.15 p.m.	$4\frac{1}{8}''$
	2.15 " " 3.45 "	$0''$
	2.45 " " 3.15 "	$4\frac{3}{4}''$

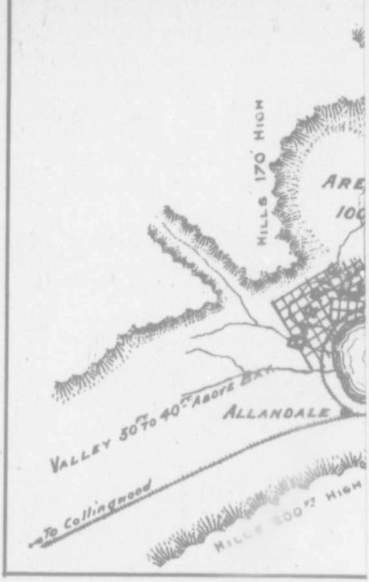
The rainfall as above given was determined by measuring the depth collected in open vessels, barrels, pans, etc., within or near the flooded district.

It is probable that between $1\frac{1}{2}$ and 2 inches of rain fell during Wednesday and Wednesday night, and that during the afternoon of Thursday the rainfall was between 3 inches and 6 inches over the drainage area of the stream that caused the flood.

Three miles south of Barrie, ten miles north of Barrie, and 15 miles east of Barrie there was but little rain on Thursday.

The distance from the centre of the drainage area of the stream to

BARRIE



HILLS 170' HIGH
VALLEY 50FT TO 40FT ABOVE BAY
ALLANDALE
To Collingwood
HILLS 200FT HIGH

THE BARRIE FLOOD OF 1890
Old Culverts
Flooded Streets
SOPHIA
ROSS ST
PARK

7. Dunlop st. and Collier st. were flooded from Clapperton to Owen for two hours.

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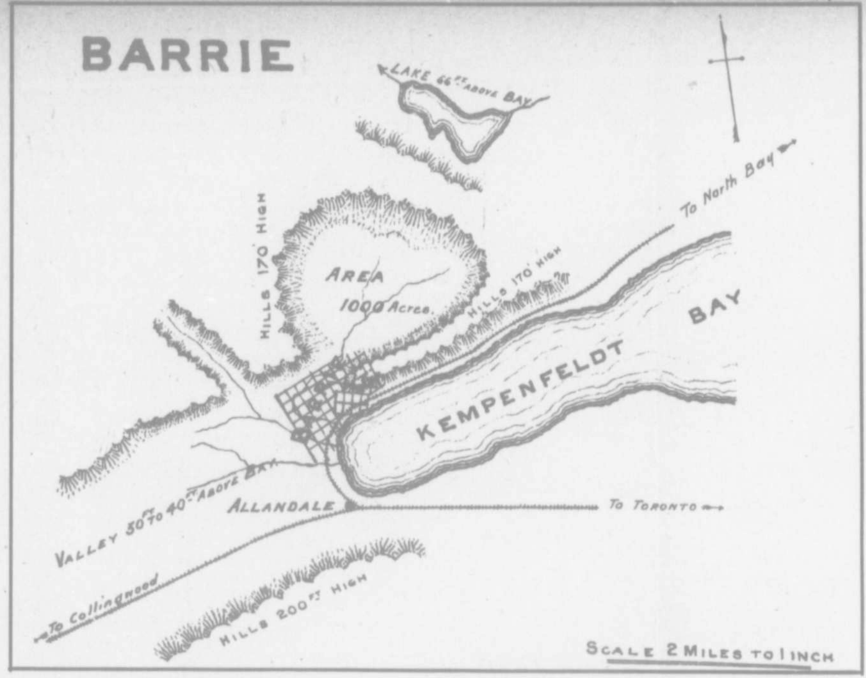

BARRIE
LAKE 66 FT ABOVE BAY
To North Bay
HILLS 170' HIGH
AREA
1000 Acres.
HILLS 170' HIGH
BAY
KEMPENFELDT
VALLEY 50 FT TO 40 FT ABOVE BAY
ALLANDALE
To TORONTO
To Collingwood
HILLS 200 FT HIGH
SCALE 2 MILES TO 1 INCH


THE
BARRIE FLOOD
OF
1890
Old Culverts
Flooded Streets
JAMES ST.
McDONALD ST.
WORSLEY ST.
ROSS ST.
COLLIER ST.
DUNLOP ST.
Town Hall
PARK
G.T.R.
KEMPEFELDT BAY

Clapperton st. is approximately 4,000 feet, the fall being over 50 feet or an average inclination of one in eighty, the water course being very crooked above Peel st. and obstructed by vegetation. The velocity of the stream was not observed, but it was probably not less than 2 feet or more than 5 feet per second.

The writer was engaged to report upon the best means of repairing the damage done to the streets, and the works necessary to prevent a re-occurrence of the disaster.

The works consisted in :

1. Laying a sewer to the full length of Clapperton st., and then re-filling the street.

2. Straightening and enlarging the channel of the stream below Peel and diverting it from private property where possible.

3. Raising the roadway on Sophia st.

4. Constructing culverts of uniform cross-section straight from end to end, and all built to grade.

DISCUSSION.

Mr. H. Holgate. Mr. Henry Holgate said that for some time previous to the above date he had kept a record of rainfall at Allandale which is adjacent to Barrie, and upon the day above referred to he found that the rainfall was $6\frac{1}{4}$ inches in three hours. The speaker, however, is unable to say what the fall was during any portion of this time, not having been where the gauge was during the rainfall; but as the rain was not steady, being a succession of heavy downpours, with intervals of about half an hour, he is sure it will be admitted that the maximum rainfall must be greater than would be given by dividing the total rainfall in inches by the length of time given above.

The speaker regrets that he cannot give the real maximum rainfall. His rain gauge was of such construction as to give accurate measurements, and was located in an open place, in no way interfered with by buildings or trees.

Should further proof be sought of this extraordinary rainfall from the Meteorological Office at Toronto, he would suggest that at the same time the records of rainfall for Sept. 17th, 1879, within the district from Barrie to Toronto be procured.

See clipping from *Northern Advance* of May 9th, 1895, as to repetition of rainfall, though not so severe as in 1890 :—

"NEARLY A DELUGE.

"History repeats itself, and so do floods. Barrie came near having a "repetition of the flood of June 5th, 1890. Between 5 and 6 o'clock in "the afternoon of Tuesday, this locality was visited by a thunderstorm "and an unusual fall of rain, and Sophia street creek was filled beyond "the capacity of the culverts to carry the water away, and so flooded "the low lands in its course. The old railway bridge at the entrance "to the old agricultural park and the roadway were washed away, "leaving a wide gap about 12 or 16 feet deep. The water covered "the flats from Toronto to Mary street, flooding the basement of the "Elizabeth street Methodist church and completely filling the cellar of "Mrs. Hind's store, covering Mr. Scroggie's property and lower part "of the Water Works grounds. The culverts near the railway track "became choked with driftwood, endangering the railway track. One "half hour's more rain and we should have had quite as disastrous a "flood as the one in 1890. Sophia street creek overflowed the bank "near Bayfield street, the water running through the block to the south. "It is quite evident that the culverts are not large enough for an "emergency such as these storms. The culvert at the corner of Sophia "and Bayfield streets is manifestly defective. The entrance should flare "so that the drain may do all that is required of it, but it is much "narrower at the entrance than the creek channel, and backs the water "instead of carrying it away. The whole bed of the creek should be "widened. The Board of Works has quite a chore on hand to make "things right."—May 7th, 1895.

Mr. Chipman in reply to a communication from the Secretary said : Mr. W. Chipman.

In reference to the rainfall, there is nothing inconsistent in the fact that the quantity observed in the flooded area in 1890 was greatly in excess of that recorded ¾ of a mile away. The writer does not say, however, that the fall reported by the Corporation employee is correct.

In a Paper by E. Kuichling, M.Am. Soc. C.E., on Rainfalls and Discharge of Sewers, Trans. Am. Soc. C.E., Jan., 1889, the following recorded rainfalls are given :—

Place.	Date.	Amount in inches.	Time hrs. min.
Washington..............	1872	1.50	1 00
Boston......	1888	1.17	30
St. Louis.............	1884	5.05	1¼
Providence......	1878	4.49	1 00

Rudolph Hering, M.Am. Soc. C.E., in discussing the Paper gives the following :—

Place.	Date.	Amount in inches.	Time hrs. min.
New Lake, Mass............	1878	6.50	2 00
New Brunswick, N.J.......	1887	4.50	1 00
Auburn, N.H............	1877	3.00	35
Grace, Ohio......	1888	7.00	2 00
Cresco, Iowa................	1883	4.30	1 00
Des Moines, Iowa.........	1879	3.00	1 00
Clear Creek, Neb............	1880	4.80	1 27
Dodge City, Kan..............	1888	3.24	45
Galveston, Texas...........	1871	3.95	14
New Market, Alabama......	1888	4.80	2 00
Greenville, Tenn............	1885	2.00	15

Place.	Date.	Amount in inches.	Time hrs. min.
Embarras, Texas	1881	2.30	15
Galveston, Texas	1873	3.50	30
Keswick, Va	1881	2.00	30
Norfolk, Va	1888	2.48	10
Elsworth, N.C	1880	9.00	3 30
Aikens, S.C	1878	4.00	1 00
Jacksonville, Fa.	1873	3.72	41
Biscayne, Fa	1874	4.10	30

In regard to the flood on Tuesday, May 7th, 1895, the present Town Engineer, Mr. Ardagh, writes that the registered rainfall was 1.44 inches, all of which fell in 45 minutes, or at the rate of nearly 2 inches per hour.

Below the Park the flooding was caused by the collapse of an abandoned railway culvert in the Park, the debris from which obstructed the culverts below it. One stump removed was 6 feet in length with roots spreading to 7 feet in diameter. It is stated that the flooding of Sophia Street at Bayfield was not caused by any accumulation of debris, but the evidence is not conclusive. The new culvert at this point constructed in 1890 has more than double the capacity of the old culvert, and more than three times the cross-sectional area of the old culvert on John Street.

In Barrie the matter of first cost determined to a certain extent the size of the new culverts. The professional literature on the proper sizes of culverts is scanty.

Given a watershed as described in the paper, is a culvert with a uniform cross-section of 35 square feet with a grade of 0.66 per 100 considered of sufficient size by the Engineering profession?

Thursday, 9th May.

THOMAS MONRO, President, in the Chair.

The discussion on Mr. Chipman's paper on "The Barrie Flood of 1890," and on Mr. Thompson's paper on "A Micrometer Attachment" occupied the evening.

Thursday, 23rd May.

P. ALEX. PETERSON, Past President, in the Chair.

The following candidates, having been balloted for, were declared duly elected as :—

MEMBERS.

EDWARD Z. DUCHESNAY, EDWARD HENRY KEATING.

ASSOCIATE MEMBERS.

JOSEPH P. B. CASGRAIN, ARTHUR CRUMPTON.

STUDENTS.

WILLIAM F. ANGUS, HUGH C. BAKER,
HARRIE MILES DIBBLEE, ALEX. R. GREIG,
ARCHIBALD MCGILLIVRAY, KENNETH MOODIE,
SAMPSON P. ROBINS, ROBERT P. ROGERS,
JOHN KIMBALL SCAMMELL.

The following were transferred from the class of Associate Member to the class of Member :—

JOHN SEABURY O'DWYER.

The following were transferred from the class of Student to the class of Associate Member :—

ROBERT BICKERDIKE, JR., GEORGE HENRY RICHARDSON.

Thursday, 23rd May.

P. ALEX. PETERSON, Past President, in the Chair.

Paper No. 106.

SPECIAL TRACK WORK FOR ELECTRIC STREET RAILWAYS, ESPECIALLY REFERRING TO THE MONTREAL AND TORONTO SYSTEMS.

BY E. A. STONE, MA.E., A.M. CAN. SOC. C.E.

Special work is the general term applied to all track work not included in the ordinary straight track; its construction for electric railways has undergone great improvements during the last few years, and is still improving. The introduction of electric power for the purpose of city passenger traffic gave rise to the present substantially constructed cars, which, with their additional weight of motors, brought about radical changes in the construction of the track.

Besides electricity as used in the trolley system, other motive powers have been tried to take the place of the horse, such as gas and compressed air motors, cables, electric conduits and storage batteries; but up to the present time, the trolley system has demonstrated its practical superiority over all others.

The track which had answered all purposes for the old comparatively lightly constructed horse cars became utterly useless for the motor cars. As the special work is subjected to the greatest wear, and consequently requires the most frequent renewal, it changed form completely. The old cast-iron curves, with their short, lightly constructed switches and poor joints, had to give way to the heavier steel construction, bearing a greater resemblance to that of a steam railroad.

Special track work should be of good substantial construction, with the greatest care paid to the designing of the parts which wear most rapidly. It is most important that track, especially in the central parts of a city, should require renewal as seldom as possible, for such renewals are very expensive, apart from the actual cost of the new track work, as traffic is interrupted, causing great inconvenience and sometimes loss of business to the public, and generally demoralising a whole route of cars, and sometimes the greater part of the entire system. Special work should be made in such a manner as to cause the least possible obstruction to vehicles, no part rising above the level of the paving more than is unavoidable; the necessary recesses, grooves, etc., should be as narrow and shallow as possible, to prevent wheels of vehicles from catching Flat surfaces should have a rough top to prevent horses from slipping upon them. All pieces should be finished so as to facilitate the paving, no long, unnecessary projections being left on bolts, etc. The curves should be of as great a radius as the width of the streets will allow. The sharper the curve, the greater is the wear on the track and wheels of cars; the slower the rate of motion, the more power required to drive the cars, the more uneven the motion and the greater liability to derailment.

The track may be laid on longitudinal stringers, on cross ties, or directly on concrete with tie bars connecting the rails. The old tracks of strap rail were laid on stringers, and the rail generally called stringer rail. (Figs. 1 and 2.) The greater part of the new construction is laid on ties, and in many respects is similar to steam track work. A combination of these two methods, consisting of planks laid longitudinally on cross ties, in order to give a more even surface, has been tried, but the results do not seem to have been so satisfactory as were expected. In several streets in Montreal, where permanent paving has been laid, the rails have been laid directly on concrete, and bound together by flat tie bars with threaded ends and double nuts. This, with the concrete between the ties, and paving, makes a very solid bed; however, it does not seem to have so much elasticity as track laid on ties in macadam.

The rails used in Toronto and Montreal are "Girder" rails. Those first laid have a height of $6\frac{1}{2}$ in. with a flange of $4\frac{1}{2}$ in., while those laid later are $6\frac{5}{8}$ in. high with a flange of 5 in. The web of the rail is not directly below the centre of the head as in the "tee" rail, but nearer the gauge line, while a flangway $1\frac{1}{4}$ in. wide at the top is provided for by a projecting lip. These rails average 75 lbs. per yard. This type of rail (Fig. 3) is used on all straight pieces and outside rails on curves in the special work. The inside rails are made of a section very similar to this, the principal difference being that the lip is much heavier, being one inch in width at the top and rising 5-16ths in. above the level of the head of the rail; this provides an efficient guard for the cars in running round a curve, the groove is $\frac{1}{4}$ in. wider than in the ordinary girder rail. This rail weighs 84 lbs. per yard. (Fig. 4.) Another section (Fig. 5) is, however, coming into use, and will no doubt largely replace these sections for special work; it is the same as the guard rail section, except that the groove is filled up with solid metal to within 9-16ths in. of the top of the head, thus providing a double bearing for the wheels, as both flanges and treads of wheels rest on the metal, so that the cars pass over all points without jolting, and the wear on the least durable parts of special work, viz., points, is greatly diminished. This section gives a rail of 89 lbs. to the yard. The peculiar sections of these rails, with their thin flanges and webs, and much thicker heads, cause a variable amount of toughness in the section; the head having received the least amount of rolling proportionally and taking the

FIG. 1.
STRINGER RAIL.

FIG. 2.
STRINGER GUARD RAIL.

FIG. 3.
GIRDER RAIL
ORDINARY SECTION.

FIG. 4.
GIRDER RAIL
GUARD SECTION.

FIG. 5.
GIRDER RAIL
SOLID FLOOR SECTION.

FIG. 6.
TEE RAIL
56 LBS.

longest time to cool is not so tough as the web and flange. Tests on pieces taken from the guard rail (Fig. 4) have given the following results:—

Head:—Tensile strength—64,300 lbs. per sq. in.

Elastic limit—75 per cent. of tensile strength.

Elongation on 4 in.—3½ per cent.; reduction in area—2 per cent., with an even and uniform whitish gray fracture, moderately fine grained.

Web:—Tensile strength—91,250 lbs. per sq. in.

Elastic limit—75 per cent. of tensile strength.

Elongation on 4 in.—27 per cent. ; reduction in area—20 per cent., with a fine grained light gray fracture.

The necessity for the increase in the weight of the new rails over the old is made apparent when it is considered that the weight of a motor car averages about 6 tons, while the weight of the old horse cars averaged only about 2 tons ; and whereas horse cars run at the rate of about 6 miles per hour, electric cars frequently have a speed of 15 miles per hour. Tee rail (56 lbs.) is also used largely for this work, but its use is generally confined to macadamised roads in the suburbs, as its height is not suitable for paving purposes (unless raised on chairs), although otherwise quite as efficient. (Fig. 6.) The girder rail being so high admits of block paving, and by the lip on the inside provides a good edge for the pavers to work to, whilst the narrow groove offers a very slight hindrance to vehicles.

In tee rail special work, the inside rail on curves is generally guarded by a second rail being bolted to it, the two rails being held apart by cast iron filling pieces ; the space between these rails is afterwards filled with cement to within an inch from the top, so as to cause as little obstruction to traffic as possible. The guard rail is slightly elevated above the running rail. Frequently rails are used in paved streets of insufficient height to admit of a paving block between the ties and the head of the rail ; when this is the case, the difference in height has to be made up by the use of chairs. This leads to rather complicated joints, and requires a longer time to lay than the method of direct spiking to the ties.

MAIN DIVISIONS OF SPECIAL WORK.

Special work may be divided into four classes considered with respect to its use and its position when in place, viz. :—intersections, passing sidings, crossovers and turnouts, and miscellaneous combinations.

1. *Intersections.*—By the term intersection is meant the special work placed at the intersection of two or more streets, and may assume an almost endless variety of forms as regards number and direction of curves and the alignment of the main tracks. The work must be so constructed as to guide the cars in whatever direction required, without any other external assistance than the moving of the tongues in the switches by the motor men. The cars must ride as smoothly as possible, *i.e.*, there should be no jolting ; in places where a groove is to be crossed that would cause the car to run unevenly, the floor should be raised so as to give a bearing on which the flanges may run. On double

track lines the distance between tracks is usually from four to five feet, but in order that cars may pass one another on the curves, and not be obliged to wait at the ends, this distance is increased to about seven or eight feet to provide ample clearance. This extra width is obtained by striking the curves from different centres, *i.e.*, the curves are not concentric. The practice in Montreal and Toronto has generally been to make the inner and outer curves of the same radius when the apex angle has been nearly 90° ; but when the angle varies greatly from a right angle, the outer curve has generally been made sharper than the inner when running round the obtuse angle. When the centre line of a street

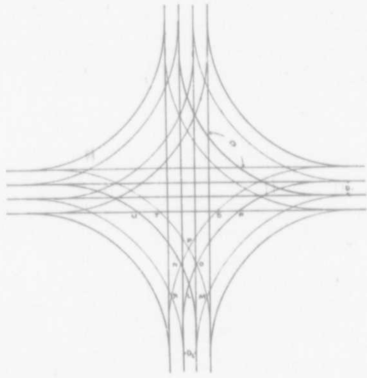


Fig. 7
FOUR CONNECTING CURVES

changes direction, or has a "jog" at the intersection, necessitating a plain or reverse curve on the through tracks, the complications increase very rapidly.

2. *Passing Sidings.*—These are used on single track lines where cars run in both directions; they may be divided into two classes, viz.: diamond and thrown-over sidings.

In the diamond siding (Fig. 8) the track diverges like a Y at

FIG. 8.
DIAMOND SIDING.

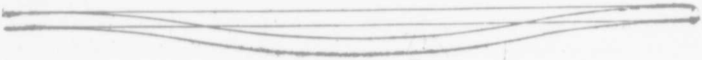


FIG. 9.
THROWN OVER SIDING.

either end, so that the centre line between the tracks in the siding is on line with the centre line of the single track; this is the form usually adopted on single tracks running through narrow streets. If it is desired that cars shall run either to the right or left at these points, the switches of the sidings must be provided with movable tongues; but if the cars always run in the same direction, they may be guided in the direction required by a movable tongue held to the proper side by a spring, so that a car facing a switch is always guided to the same side, and a car trailing it compresses the spring, and passes on, the tongue of the switch falling back to its proper position. (See Fig. 25.) This guiding of the car in one direction, however, may be provided for much more simply by means of a switch without any movable part, commonly called a blind switch. One side of the switch is straight and the other curved. The front of the switch coincides approximately with the end of the curve of the switch, whilst the curve to the opposite side begins near the back of the switch, as shown in Fig. 10. If the cars always

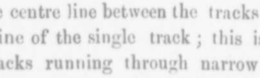


Fig. 10

run to the right (as in Montreal and Toronto) the switch is made left hand, *i.e.*, the P.C. of the curve turning to the left is in front of the P.C. of the curve turning to the right by the length of the switch (approximately). Thus, a car approaching the siding travels straight along on the tangent past the point of the switch, and is then curved out of its path to the side by the curve in the rail behind, and when leaving the siding runs over the curve of the switch; this is the best arrangement for such sidings, as it is the simplest, most durable, and causes least delay to the cars.

In the thrown-over siding (Fig. 9) one track is continued straight through, whilst the other is thrown over to one side of it; this is suitable for single track lines on a wide street, or in places where the track is on one side of the street. If cars are to be run to either side, switches with movable tongues are necessary; but if the cars always keep to the same side, the tongues must be provided with springs, or blind switches used. With the latter the problem is not so simple as in the diamond siding, and in order to solve it the main track has a slight reverse curve placed in it extending from the front of the switch to a short distance inside the curve cross; by introducing this, the general arrangement for the diamond siding holds good. (See Fig. 11.) The radius for the curves of passing sidings in Montreal and Toronto is 300 feet to inside gauge line.

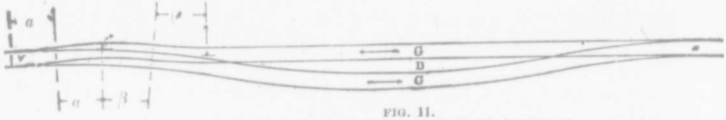


FIG. 11.
THROWN-OVER SIDING WITH BLIND SWITCHES.

3. *Crossovers and Turnouts.*—Crossovers (Fig. 12), sometimes called connecting tracks, are used on double track lines for the purpose of transferring cars from one track to the other, and consequently are placed at the terminations of regular routes and at points which are made temporary terminii to accommodate special traffic.

Turnouts (Fig. 13) are used where a double track runs into a single track, the centre line of the single track being on line with the centre line of one of the tracks of the double track line.

These crossovers and turnouts, as well as all special work, should change the direction of the car's motion from one line into another with the least amount of resistance possible consistent with the data given. Those in Montreal and Toronto have 75 feet radius curves and about 25 feet of tangent, the latter varying with the distance between tracks. This gives a crossover of about 60 feet between extreme ends of switches.

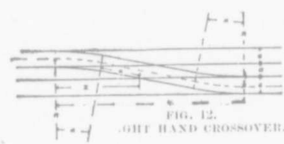


FIG. 12.
RIGHT HAND CROSSOVER.

FIG. 13.
RIGHT HAND TURNOUT.

Crossovers and turnouts are said to be either left or right hand, according to the direction in which they curve from the track, as seen from the switch when looking towards the cross. Fig. 12 shows a right hand crossover. If a crossover of either hand is suitable at a certain point of the line, one of the same hand as the side to which the cars run should be chosen, *i.e.*, right hand crossovers are preferable for systems on which the cars run to the right and left hand, on those in which the cars keep to the left; this is on account of the fact that cars running always to the right will trail all switches of right hand crossovers and face those of left, so that they cannot possibly take the wrong track in the first case, while they may be suddenly thrown out of their course in the second, and accidents result.

In addition to permanent crossovers it is always necessary to have temporary ones during construction, which are laid directly on top of the paving wherever required. These are so constructed as to be easily and quickly laid in place and readily moved from one part of the line to another by a small gang of men.

4. *Miscellaneous Combinations.*—Besides the work already mentioned, there are several kinds of diamonds made to fill various requirements; there are also special combinations for car houses, etc. The simplest kinds of diamonds are those used where electric lines cross electric lines, and only require the running rails. When an electric road crosses a steam road, the steam road track requires guard rails for greater safety, and the electric line should also be guarded either by an additional rail or plate.

SUB-DIVISIONS.

Intersections, cross-overs, etc., are composed of several pieces, which may be divided into the following sub divisions, viz. :—Tongue switches (single and double curve), blind switches, mates (single curve, double curve and combination), curve crosses (single curve, double curve and combination), diamonds (for electric and steam crossings), split switches, stub switches and lengths of rail (curved and straight). (See Figs. 24 to 32.)

1. *Tongue-Switches.*—The tongue switch is perhaps the most important piece in any combination of special work, as it is subjected to greater and more frequent shocks than any other piece, its duty being to change the direction of the car's motion from one line to another. When made of girder rail, it is constructed of the guard rail section to ensure the perfect guidance of the wheels. When made of

tee rail, a guard is formed either by bolting on another piece of rail, or by carrying up the casting on the side to form the required guard. The switch generally consists of four main parts, viz.:—the tongue, a casting and two pieces of rail. The tongue is made of steel, and should be of a substantial size, having a cross section near the point, proportioned to resist violent shocks; at the same time the point must be rather sharp to ensure the car "taking" it exactly; if blunt, the car may mount the tongue, and drop again, causing a severe jolt. If the top of the tongue rises above the level of the head of the rail, it is sloped at both ends so as to allow the rise and fall of the car to be imperceptible. The pin must be so placed as to make it impossible for a wheel to touch the tongue behind the pin, and so throw the switch before the back wheels have reached the point. If the tongue were made so long that the distance from the centre of the pin to the tongue point were greater than the wheel base of the cars (about 7 feet) this would be impossible; this method, however, would necessitate a too expensive switch, and the difficulty is easily overcome by rounding the back of the tongue and placing the pin sufficiently far back. The pin should also be placed so that the wheels do not run over it, and so cause it to become loose, and should be so fastened to the casting that the tongue may easily be removed at any time. The top of the casting on which the tongue slides and the bottom of the tongue should be truly even, as, if not, dirt will collect between the two, and after a short time the tongue will tilt when a car runs over it, and may cause the tongue to throw to the opposite side, or the back wheel may strike the point, either of which may be sufficient to throw the car off the track. Single curve switches are those curved only on one side; double curve switches are curved on both sides. (Figs. 24, 25 and 29.)

2. *Blind Switches.*—The blind switch is used in place of the tongue switch where cars always run off the curve at that point and never enter it. It closely resembles the mate in general construction. In order that the guidance of the car facing the switch may not altogether depend on the fact that the car will naturally take the straight track in the direction in which it is moving, rather than turn into the curve, a ridge is left along the floor on the straight track which acts as a gauge line, to make it practically impossible for the car to enter the curve. (Fig. 30.)

3. *Mates.*—The mate is the piece opposite the switch, on which the wheels of one side of the car run while the wheels on the other side are being pulled around by the switch; its sole use is to provide a

surface for the wheels to run upon, and has nothing to do with the change in direction of the car's motion. It is made of two pieces of rail, and sometimes there is a castin g. One piece of rail extends over the whole length, and is straight if for a single curve mate, and curved if for a double curved mate ; the other piece is shorter and always curved, the head terminating in a point. This point should be so designed that the gauge at the point is quite slack, so that a wheel facing the mate may not strike upon it. The width of the point should not be less than $\frac{1}{2}$-inch, as if made sharper it will wear to this. In girder rail the solid floor section makes the best mate, as it provides a wide floor for the wheels to roll upon, and the depth of the floor below the head of the rail being less than the depth of the flange of the wheel, it quickly wears so as to provide a double bearing for the wheels, so that the point is passed without the wheels dropping heavily upon it. If the mate is not made of the floor section, but of the ordinary girder rail as used on the straight track, or if of tee rail construction, a steel casting is necessary to carry the wheels over the point from the long rail on to the short one. This casting is more efficient if carried up on the inside to provide a guard; for in case of the gauge being too slack, the tongue may have a tendency to jerk the car off the track. This casting must project considerably inside the gauge line of the short rail, the path of the rear wheels on a truck not coinciding with that of the front ones but lying about $\frac{1}{2}$-inch inside, as may be clearly seen on any worn mate. (Figs. 26 and 31).

4. *Curve Crosses.*—Curve cross is the name given in this work to the piece corresponding to the frog in steam railroad work; it differs considerably from the frog, however: one, at least, of the rails in a curve cross is generally curved to a very sharp curve, whilst the frog is straight on either track. The frog has wing rails, and a wheel crossing a frog runs from one piece of rail across the channel on to another rail, whilst in the curve cross a wheel generally runs the entire length of the cross on one piece of rail, the channel for the flanges being shaped out of the head of the rail. According as one or both rails are curved, the cross is said to be a single or double curve cross. (Figs. 27 and 32.)

5. *Diamonds.*—Diamonds are made in various ways, according to the requirements they are to serve. A simple single track diamond for the crossing of two electric lines consists of two main parts, each part being made of five pieces of rail, one long piece with four short pieces butting up against it, two on each side; the long rail is usually made to form part of the track on the street having the greater amount

of traffic. When an electric road crosses a steam road, the diamond is usually all made of tee rail, of the same section as the rail of the steam road. If the rails of the steam road are not to be cut, the diamond is made in three parts (Fig. 28), two outside and one inside the steam track, the whole being so constructed as to lift the street car before reaching the rails of the steam track on to the flanges of the wheels, and running across on them to the other side, and then dropping gradually to the ordinary level again, so that the only place where any jolt can occur to a car while crossing such a diamond is when it crosses the channel of the steam track rails, notwithstanding the fact that the rails of the steam track are not cut to the smallest extent to provide a passage for the flanges of the street-car wheels.

6. *Split Switches.*—Split switches are used to a comparatively small extent on this class of work. They are more especially adapted to suburban traffic where tee rail is used, rather than crowded thoroughfares of cities. They are especially suitable where cars always run to the same side, when the switch may be made to work automatically by means of a spring, and in this way they have been found very satisfactory.

7. *Stub Switches.*—Stub switches are suitable for yard purposes and sidings only occasionally used; they are cheap, which is always a point in their favour. The use of a stand prohibits their use in city thoroughfares.

8. *Lengths of Rail.*—Rails for all special work should be accurately cut to the required lengths, and carefully bent to the proper template if for use on a curve, or accurately straightened if required for straight track. If part of a rail is to be straight and the remainder curved, the rail must not only agree with straight edge and template for the required lengths, but it must be tested, to determine whether the straight part is tangent to the curve, for if not, the piece will not fit correctly when placed in the work of which it forms part.

THE DETERMINATION OF NECESSARY SPECIAL WORK.

Having laid down the routes of any street railway system necessary for the accommodation of the present traffic and that of the near future, the special work required becomes apparent. It is most important that curves likely to be required in a few years, but not necessary at the present, should be laid, if at all possible, during construction, as the addition of a single curve to an intersection in some cases necessitates the reconstruction of the greater part of the whole intersection.

SURVEYS.

A careful survey must be made of the intersection of streets requiring special work, and all measurements of lines and angles taken which are necessary to plot with the greatest accuracy the centre lines of the proposed tracks together with the street and curb lines.

PLOTTING.

These measurements are plotted to a suitable scale (say 10 feet to 1 inch), and the most suitable radii for the required curves determined, which are usually from 40 to 75 feet radius (45 and 50 ft. are most common in Montreal and Toronto).

The attempt is sometimes made to ease these curves as on steam railroad work; but when it is remembered that the length of most of the curves is about 80 ft., it will be seen how limited the space is in which to attempt anything of the kind; however, an improvement may be introduced by making the switches at the ends of curves of a longer radius than the main part of the curves, such as using 75 ft. radius switches on 45 ft. radius curves. This eases the curves for 10 ft. at each end and meets all practical requirements. Any further steps in this direction would seem to lean towards "hair splitting."

It might here be mentioned that although these curves would appear very sharp to engineers accustomed to steam railroad work, yet there is a case on record of a 50 ft. radius curve on a trestle being used on a steam railway, and operated successfully, the speed on it being from 8 to 10 miles per hour. (U.S. Military Railway, Petersburg, Va.; see Trans. Am. Soc. C.E. 1878.) The Manhattan Elevated Railway in New York city has curves of 90 feet radius.

There should be, if possible, sufficient space between the inside rail of the curve and the curb stone for a vehicle to pass a car easily; this, however, requires very wide streets. If this cannot be done, the rail should be at about two feet from the curb stone at the corner, for if at say four feet, there would not be sufficient room for a car and vehicle to pass, but the attempt might be made and an accident ensue. The radii of the curves should also be determined with a view to sufficient room for the switches; if this is not looked to, special short switches may be required, which is not desirable. The intersecting points of the gauge lines should also be carefully observed, as, by the slight alteration of a radius, combination pieces of complicated construction and of an unendurable character may often be avoided. The radii having been fixed, the gauge lines alone may be laid down to a large scale (say 4 feet to 1 inch), and the calculations proceeded with.

CALCULATIONS.

The data on which the calculations are based are :—the gauge, distance between tracks, angle of intersection, radii of curves, and sometimes distances between apexes and deflection angles.

First, the tangents and lengths of all curves are found ; next, the distances between the ends of the curves are determined.

In the case of a double track branch-off, with inner and outer curves of the same radius and equal central distances, this distance, a (Fig. 15), is given by——distance between *P.C.'s*, $a =$ (gauge + central distance) $\tan\frac{\text{centre angle.}}{2}$

If the radii are equal, but the central distances on the two streets are unequal, the distances required may be found as follows :—

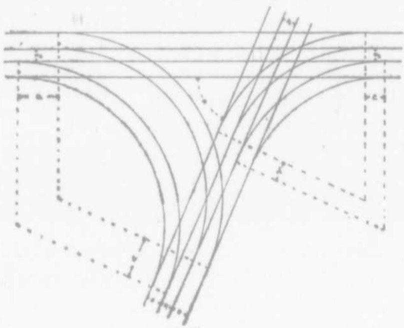


FIG. 14.

Let $G =$ gauge. (See Fig. 14.)
" D_1 and D_2 = central distances.
" $\alpha =$ angle of intersection.

Since the radii of the inside and outside curves are equal, the tangents (for the same angle) are equal.

$\therefore$ distance between *P.C.'s* = distance between apexes.
(both measured parallel to gauge lines)

$$\therefore\ a = (G + D_1)\operatorname{cosec}\alpha + (G + D_2)\cot\alpha$$
$$b = (G + D_2)\operatorname{cosec}\alpha + (G + D_1)\cot\alpha$$
$$c = (G + D_1)\operatorname{cosec}\alpha - (G + D_2)\cot\alpha$$
$$d = (G + D_2)\operatorname{cosec}\alpha - (G + D_1)\cot\alpha$$

When both the central distances and radii vary, the distances between *P.C.'s* are found by adding and subtracting the lengths of the tangents, making allowance for the apex angle if differing very much from a right angle.

Next, the number of pieces into which to divide the intersection is determined, and the proper lengths for switches and mates fixed.

The points where the curves intersect the straight gauge lines are next found; this may be done by either of the two following methods:

Taking Fig. 15 with distances as marked.

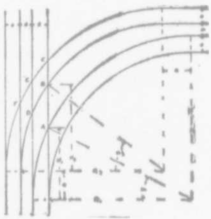


FIG. 15.

1st Method. Consider the point A,

$$H_1 = \sqrt{(R_1 + G)^2 - R_1^2}$$

$$= \sqrt{2\,G\,R_1 + G^2}$$

$$sin\ a_1 = \frac{H_1}{R_1}$$

$$\therefore\ a_1 = sin^{-1}\left(\frac{\sqrt{2\,G\,R_1 + G^2}}{R_1}\right)$$

Similarly for B, $H_2 = \sqrt{R_2^2 - (R_2 - D - G)^2}$

$$sin\ a_2 = \frac{H_2}{R_2}$$

$$\therefore\ a_2 = sin^{-1}\left(\frac{\sqrt{R_2^2 - (R_2 - D - G)^2}}{R_2}\right)$$

and so on for other points.

2nd Method.—For A, $vers\ a_1 = \frac{G}{R_1 + G}$ $\therefore\ a_1 = vers^{-1}\left(\frac{G}{R_1 + G}\right)$

$$H_1 = R_1 \sin a_1$$

For B, $a_2 = vers^{-1}\left(\frac{D + G}{R_2}\right)$

$$H_2 = R_2 \sin a$$

Similarly for other points.

At a distance s, the spread $w = 2\,s \sin \frac{a}{2}$ (see Fig. 16), which is

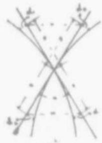


FIG. 16.

the distance between two points at a distance s from the intersection point, one on the straight gauge line and the other on the tangent to the curve at the intersection point.

The straight lengths of the figure (Fig. 15), *i.e.*, the distances along the straight track between the points A, B, etc., are found by means of the lengths H_1, H_2, etc., and the distance between the $P.C.$'s. The arc to any point from the $P.C.$ is given by :—

$$\text{arc} = \text{radius} \times {}^{*}c.m.\ a.$$

So that the curved lengths, *i.e.*, the distances between the points D, B, – F, E, etc., are found by taking the differences between the arcs to these points; while the distances beyond A, B, etc., to the other end of the curve are found by taking the differences between the total lengths of the curves and the arcs to these points.

The following tables have been calculated by means of the preceding formulæ :—

	Radius of inside gauge = 45′ 0″ Gauge = 4′ 8½″. Central distance = 4′ 0″				Radius of inside gauge = 50′ 0″. Gauge = 4′ 8½. Central dist. = 4′ 0″			
Points as in Fig. 15.	Perpendicular from P. C. in feet.	Angle at centre subtended by arc to point.	Arc from P. C. to point in feet.	Spread at two feet.	Perpendicular from P. C. in feet.	Angle at centre subtended by arc to point.	Arc from P. C. to point in feet.	Spread at two feet.
A & *F*	21.117	25° 08′	21.812	$10\frac{7}{16}$″	22.204	23° 57′	22.863	$9\frac{15}{16}$″
B	26.607	36° 15′	28.467	$14\frac{15}{16}$″	28.196	34° 20′	29.995	$14\frac{3}{16}$″
C	33.968	43° 06′	37.396	$17\frac{5}{8}$″	35.889	41° 00′	39.144	$16\frac{13}{16}$″
D	18.547	24° 21′	19.116	$10\frac{1}{8}$″	19.596	23° 05′	20.137	$9\frac{5}{8}$″
E	28.105	34° 26′	29.870	$14\frac{3}{16}$″	29.614	32° 46′	31.293	$13\frac{9}{16}$″

When the intersection has curves branching in both directions, as shown by Fig. 7, the points where the curves intersect as K, L, etc.,

* c.m. = circular measure.

have to be found, in order to determine the different lengths; the problem thus becomes "to determine the intersection point of two curves branching in opposite directions from parallel lines." This may be solved by either of the two following methods, the second of which is much the more readily applied. (See Fig 17.)

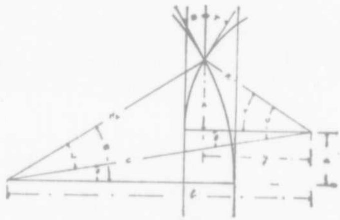


Fig. 17

Let R_1 = radius of curve with upper *P. C.*

" R_2 = " " " lower *P. C.*

" a = distance between *P. C.*'s measured parallel to gauge lines.

" b = " " centres " perpendicular " "

" c = " " " " in a straight line.

" x = " of intersection point from upper *P. C.* measured parallel to gauge lines.

" θ = angle between a line perpendicular to gauge lines and line joining centres.

" U = angle at upper centre between radius to intersection point and line joining centres.

" L = angle at lower centre between radius to intersection point and line joining centres.

" B = angle at centre subtended by arc between lower *P. C.* and intersection point.

" T = angle at centre subtended by arc between upper *P. C.* and intersection point.

1st Method:—$x^2 + y^2 = R_1^2$.

$$\therefore y = \sqrt{R_1^2 - x^2}$$

$$(x+a)^2 + (b-y)^2 = R_2^2$$

$$\therefore x^2 + 2ax + a^2 + b^2 - 2b\sqrt{R_1^2 - x^2} + R_1^2 - x^2 = R_2^2$$

which becomes

$$4x^2(a^2+b^2) + 4ax(a^2+b^2+R_1^2-R_2^2) = R_1^2(2b^2 - R_1^2 - 2a^2 + 2R_2^2)$$
$$+ R_2^2(2a^2 + 2b^2 - R_2^2) - b^2(b^2 + 2a^2) - a^4$$

Corollary. When $R_1 = R_2 = R$

$$\text{then } x^2 + ax = \frac{1}{4(a^2+b^2)}\left\{ b^2 \left(4 R_2 - b^2 - 2 a^2\right) - a^4 \right\}$$

These formulæ are very laborious to use in practice; however, as in the majority of cases $R_1 = R_2$, the corollary is the more frequently required.

Having found x, the angles B and T are given by

$$\sin B = \frac{x+a}{R_2}$$

$$\sin T = \frac{x}{R_1}$$

and the spread at a distance $s = 2s \sin\left(\frac{B+T}{2}\right)$

These formulæ apply also when the two curves branch off in the same direction, with the exception that the spread is given by

$$\text{spread} = 2s \sin\left(\frac{T-B}{2}\right) \text{ (see Fig. 18.)}$$

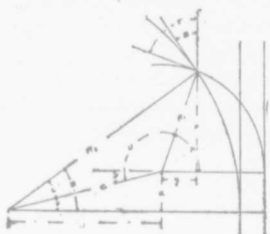


Fig 18

2nd Method:

$$\tan \theta = \frac{a}{b}$$

$$c = b \sec \theta$$

$$\cos U = \frac{c^2 + R_1{}^2 - R_2{}^2}{2\, c\, R_1}$$

$$\cos L = \frac{c^2 + R_2{}^2 - R_1{}^2}{2\, c\, R^2}$$

$$T = U - \theta$$

$$B = L + \theta$$

$$\text{spread} = 2s.\ \sin\left(\frac{B+T}{2}\right)$$

Corollary. When $R_1 = R_2 = R$

then $U = L$

$$\sec U = \sec L = \frac{2R}{c}$$

$$\text{spread} = 2s \sin U$$

When two curves branch in the same direction (Fig. 18) the above applies with the following exceptions:—

$$T = 180^\circ - (U - \theta)$$

$$\text{and spread} = 2s \sin\left(\frac{T - B}{2}\right)$$

Having fixed these points, the straight lengths are found as before by means of the perpendicular heights to the intersection points of the single curve crosses, and the distances to the diamond by means of the tangents. The arcs to the intersection points of the double curve crosses are given by :—

For arc to intersection point on curve with upper *P.C.*,

$$\text{arc} = R^1 \text{ c.m. } T.$$

For arc to intersection point on curve with lower *P.C.*,

$$\text{arc} = R_2 \text{ c.m. } B.$$

so that the distances along the arcs between the points are given by taking the differences between the arcs.

In Fig. 7 it may be noted that when the radii of all the curves are equal, the angle θ for the points L, N, O and P = intersection angle $\sim 90^\circ$.

that for the points K, L, M and P; — $R_1 = R_2$

" " " L, N, O and P; — a, b and consequently θ and c are the same.

that the angle U for the point N = the angle L for the point O, and *vice versa*.

that $L\,N = L\,O$, $N\,U = O\,R$, $O\,P = N\,P$, and $P\,T = P\,S$.

The following table has been calculated by the preceding formulæ from the following data :—(refer to Fig. 7) $D_1 = 4'\,9''$, $D_2 = 4'\,0''$, $a = 86^\circ\,33'$, gauge $= 4'\,8\frac{1}{2}''$, radius of inside gauge line of all curves $= 45'\,0''$.

Points (Fig. 7).	Perpendicular from upper $P.C.$ (x).	Angle at centre subtended by arc branching to left.	Angle at centre subtended by arc branching to right.	Spread at 2 feet.
K	5.313	24° 21′	6° 08′	$12\frac{5}{8}''$
L	10.394	20° 15′	13° 21′	$13\frac{7}{8}''$
M	13.104	15° 17′	19° 50′	$14\frac{1}{2}''$
N	16.851	29° 19′	19° 49′	$19\frac{13}{16}''$
O	17.162	26° 43′	22° 25′	$19\frac{15}{16}''$
P	22.165	33° 23′	26° 29′	$23\frac{15}{16}''$

Note :—$2(90°-86° 33') = 6° 54'$
= difference between left and right angles of L and P
= " " " of N and right of O
= " " right of N and left of O

To determine the $P.C.$ of a branch-off curve from a curved main track:

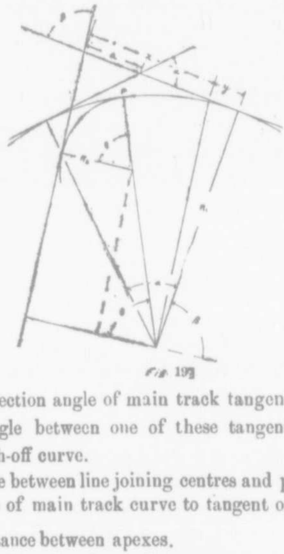


Fig. 19

Let α = deflection angle of main track tangents

Let β = angle between one of these tangents and tangent to branch-off curve.

Let θ = angle between line joining centres and perpendicular from centre of main track curve to tangent of branch off curve.

Let a = distance between apexes.

Let R_1 = radius of main track curve.

Let R_2 = " " branch-off "

It is required to determine the point P.

Taking x and y as shown by Fig. 19 :

$$x = a + R_1 \tan \frac{\alpha}{2} - y$$

$$= a + R_1 \tan \frac{\alpha}{2} - R_1 \cot \beta$$

$$= a + R_1 \left(\tan \frac{\alpha}{2} - \cot \beta \right)$$

$$\text{and } \cos \theta = \frac{x \sin \beta - R_2}{R_1 \mp R_2}$$

$$= \frac{\left(a + R_1 \tan \frac{\alpha}{2} \right) \sin \beta - R_1 \cos \beta - R_2}{R_1 \mp R_2}$$

$R_1 - R_2$, when curves branch in the same direction as in Fig. 19.
$R_1 + R_2$, " " " opposite directions.

This determines the point P with respect to either $P.C.$

To determine the intersecting points of the gauge lines when the main track curve lies wholly between the $P.C.$ of the branch-off curve and the nearest intersecting points.

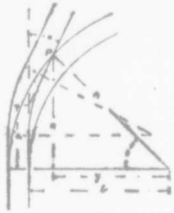


Fig. 20

P is the point to be determined (Fig. 20), taking lengths as marked.

$$x^2 + y^2 = R^2$$

$$y = b - (x - a) \tan \alpha$$

$$\therefore x^2 + \{b - (x - a) \tan \alpha\}^2 = R^2$$

which becomes

$x^2 \sec^2 \alpha - 2x \tan \alpha \, (b + a \tan \alpha) = R^2 - b^2 - a \tan \alpha \, (2b + a \tan \alpha)$

When the main track curves in the opposite direction to that of the branch-off, this equation becomes

$x^2 \sec^2 a + 2\,x \tan a\,(b - a \tan a) = R^2 - b^2 + a \tan a\,(2\,b - a \tan a)$

$$\theta = \sin^{-1} \frac{x}{R} \text{ for both cases,}$$

and spread $= 2\,s\ \sin\left(\frac{\theta - a}{2}\right)$ when main track and branch-off curve in same direction,

or, spread $= 2\,s \sin\left(\frac{\theta + a}{2}\right)$ when main track and branch off curve in opposite directions.

If the distance (h) from the *P.C.* of a curve is known, the deflection (d) to the curve at that point is given by

$$d = r - \sqrt{r^2 - h^2}$$

$$\text{or } d = r \text{ vers}\left(\sin^{-1} \frac{h}{r}\right) \text{ (See Fig. 21)}$$

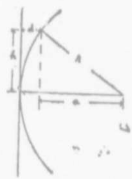


Fig. 21

In order to make templates to which the rails are bent, calculations are necessary for flat curves (over 60 ft.); but those of a shorter radius may be trammelled out. To calculate these templates, the deflections at every 3 inches from zero up to half the length of the required template are calculated by one of the above formulæ. These deflections are laid off on a board, a curve is drawn through the points so found, and the board is then cut to the curve. Of course the trammelling process is preferable whenever practicable.

Calculations for Crossovers.—Taking lengths as shown by Fig. 12.

$$2\,R \text{ vers } a + \text{tangent} \sin a = D + G$$

First, a length may be fixed upon approximately as desirable for a tangent; with this length, solve for a (most easily done by trial), having found a approximately, assume an even value for it (say to nearest 10 minutes) for simplicity, and with this value solve the equation again for the length of tangent, determining it exactly, which will be very close to the desired length (practically the same).

The distance from centre *P.C.* to intersecting point of inside gauge is given by

$$x = D \, cosec \, a - \left(R - \frac{G}{2}\right) tan \, \frac{a}{2}$$

The total length between extreme end *P.C.*'s is given by

$$y = 2 \, R \, sin \, a + tangent \, cos \, a$$

The distance from end *P.C.* to nearest intersecting point measured along main track is given by

$$Z = \left(R - \frac{G}{2}\right) sin \; a + x \, cos \, a$$

$$= D \, cot \, a + \left(R - \frac{G}{2}\right) \left(sin \, a - 2 \, sin^2\frac{a}{2}\right)$$

By making tangent $= 0$, the conditions for a reverse curve are given $2 \, R \, vers \, a = D + G$

and $y = 2 \, R \, sin \, a$

When a crossover is required for a width between tracks, D_1, the only change necessary in a crossover designed for a width D is in the length of the tangent which is changed by a length $= (D_1 \sim D)$ $cosec \; a$.

To determine a reverse curve (short tangent between curves) between two tangents not parallel, at an intersection.

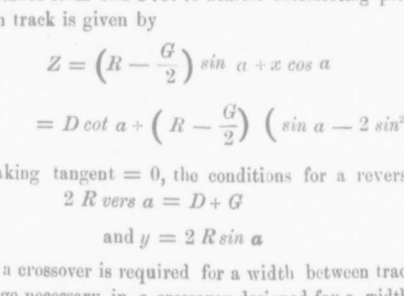


Fig. 22

A A. and *B.B.* are the two tangents not parallel, representing the centre lines of a street with a deflection at the intersection of another street, the centre line of which is represented by *C.C.*

Take distances as shown in Fig. 22.

Fix upon a point which will be convenient to form one end of the curve, and let its distance from an apex be b.

Then, R_1 *vers* θ + *tangent sin* $\theta + R_2$ *vers* $\theta = a \sin a - b \sin (a - \beta) + R_2$ *vers* $(a - \beta)$.

As in the ordinary crossover calculations, fix θ by trial and then solve for the tangent,

$$\text{tangent} = \frac{1}{\sin\theta}\left\{ a \sin a - b \sin (a - \beta) + R_2 \operatorname{vers} (a - \beta) - \operatorname{vers}\theta\,(R_1 + R_2)\right\}$$

Having determined upon the angle θ, and found the tangent, the other lengths are easily found.

Calculations for Diamond Siding.—Consider end A, Fig. 8.

$$\operatorname{vers} a = \frac{D + G}{4R}$$

total length between extreme $P.C._s = 2 R \sin a$

cos (angle at centre subtended by arc from right hand $P. C.$ to intersection point) $= \dfrac{R - \frac{1}{2}(G + D)}{R - \frac{1}{2} G} = \cos \beta$

angle of curve cross $= 2\beta$

distance from right hand $P. C.$ to intersection point $= (R - \frac{1}{2} G) \sin \beta$.

These calculations apply when the curves begin at the same point to branch to either side as in Fig. 8 ; but when the curves begin at different points (for blind switches) as in Fig. 10, the intersecting point does not lie on the centre line, and may be found as follows :—(Fig. 23.)

$$\tan\theta = \frac{R_1 - R_2 - a}{b}$$

$$\cos\pi = \frac{R_2{}^2 + (b \sec\theta)^2 - R_1{}^2}{2\ R_2\ b \sec\theta}$$

$$\cos\phi = \frac{R_1{}^2 + (b \sec\theta)^2 - R_2{}^2}{2\ R_1\ b \sec\theta}$$

$$a = 90^\circ + \theta - \pi \text{ and } \beta = 90^\circ - \theta - \phi$$

$$x = R_1 \sin\beta \qquad \text{and spread} = 2\,s.\ \sin\left(\frac{a + \beta}{2}\right)$$

Calculation for thrown-over siding with blind switches.—The calcuations are generally similar to those already described for crossovers and diamond sidings, except for the curves in the main track; these are solved as follows:—(See Fig. 11, end A)

$$a = (R + \tfrac{1}{2}\ G) \text{ vers } a + \text{width of switch at back}$$

$$\text{vers } \beta = \frac{a}{2\ R}$$

Total centre angle for curve adjoining switch $= a + \beta$.

WORKING DRAWINGS.

Having completed the calculations for an intersection, the detail drawings for each piece are made, and sent to the shop, together with a print showing the whole intersection with the distinguishing marks of all pieces and lengths of the connecting rails. A drawing is also made for assembling the work in the street, showing all necessary measurements for laying out the work together with the position and marks of the various pieces.

SHOP WORK.

A bill of the rails required and the necessary new prints and references to old ones having been obtained from the Drawing Office, the manufacture of the work may be proceeded with. The bill of rails required (made out so as to give a minimum amount of scrap) is given into the hands of the man in charge of the rail saw, who proceeds to cut up the rails into the required lengths, marking the length of each and whether required straight or curved upon the web. The rails next, with few exceptions, go to the rail bender, to be either curved to the required radius, or straightened; they next proceed to the "marker off," who carefully marks the necessary lines for all machine work required to be done upon them, he also stamps the rails on the end with their distinguishing marks. The rails afterwards pass on to the machines (milling machines, slotters, shapers, planers, etc.) suited to the work required;

they then go to the fitting shop to be assembled according to the drawings.

In a tongue switch the long rail has to be properly curved, and slotted or bent for the tongue to fall into place. The tongue is made of hammered steel, and the turned pin is shrunk in; this is dropped into place, and all measurements checked before being considered ready for the track.

In the blind switch and mate, one rail is planed so as to leave a long notch on one side, while the other rail is planed to a point which fits into the notch; the two are strongly bolted or rivetted together and sometimes finished on a planer.

The curve crosses have usually two pieces of rail, one of which has the upper part so shaped at the crossing point as to allow the second one to drop down on the first, and fit accurately into the place allowed for it; while the second has the lower part shaped so as to allow the first rail to pass through, the two rails jointing neatly into one another. Great care is necessary in the fitting to have the angles of intersection exactly as required. In order to obtain the correct angle, the drawing shows the spread, w, at a fixed distance, together with the deflections, d_1 and d_2 of the curves at that point; so that this distance is measured along the rails from the intersection point and the deflections marked from the gauge line, the spread is then measured between the points so marked. (See Fig. 16.)

CHECKING.

When an intersection has been made, it is sometimes advisable to have it assembled as a final check before shipping. For this purpose a large piece of ground, as level as possible, is required, and much more than is actually occupied by the work when in place should be available; the tangents of the intersection should be laid out, and a sufficient number of points fixed to accurately check the end of each curve. Having laid out the ground, the pieces are assembled, and any errors observed may be corrected; this last step ensures the work being absolutely correct, and is the best check on the work that can be adopted.

ASSEMBLING IN THE TRACK.

In laying an intersection, it makes a great deal of difference whether the whole space required is graded at once and all traffic stopped, or if only part of the intersection is graded, leaving part undisturbed so as not to interrupt traffic. When the work has to be performed in the latter way, great care is necessary in placing the work, so that the remaining part when laid may fit up to and line in accurately with the first part. If it is necessary to lay out a curve, it is generally most easily performed by tangent and chord deflections or by ordinates from a chord. In grading a corner where an important intersection is to be laid, care should be exercised in excavating to the correct depth and having the grading done evenly, for if the track has to be lifted say six inches after being laid, it means very much more than the same lift on ordinary track, as the weight of rail is sometimes enormous as compared with the extent of ground it covers ; also, if the work has been carelessly done, and presents a very uneven bed, much more time is necessary to couple up the joints than would have been required had the grading been properly performed. The spacing of the ties for this work should receive more attention than is sometimes given to it, as it is a very important matter. The ties should be the very best available, and spaced more closely than those on the straight track.

The centre lines of tracks for both streets are accurately fixed, and if there is no diamond, the ends of the curves must be found ; otherwise, this is not essential. If there is a diamond in the intersection, this is laid first, bolted up and lined accurately. The other pieces having been scattered about in their approximate positions are next drawn to place and bolted together. The rails are then securely spiked to gauge, and lifted (if necessary) to grade, when the intersection may be paved and so completed. If there is no diamond to lay, an end of a curve may be taken as the starting point. To lay the intersection so as to have the through straight tracks in perfect alignment requires great care, as the joints are usually very close together.

An idea of the amount of rail that may be used in a single intersection, and the consequent amount of labour required to make one, may be formed from the following figures, for one laid at the intersection of St. Lawrence Main and St. Catherine streets, Montreal (same as Fig. 7). It is built of 75 lbs. and 84 lbs. girder rail (Figs. 3 and 4). It contains 2,150 feet of rail, and has a total weight of about 26 tons. There are 86 built up pieces (switches, mates and curve crosses), and 78 lengths of connecting rails, making a total of 164 pieces in the complete intersection. The extreme length between ends of opposite switches is about 110 feet. The radius of the inside gauge lines of all the curves is 45 feet, and the distance between tracks varies from 4 ft. to 8 ft. 6 in. This intersection, as well as all others in Montreal and Toronto, was made by the Canada Switch Manufacturing Co., Lim., of Montreal.

Such work, when properly constructed and laid, represents a large amount of capital, and deserves much more attention and care than the old cast iron work; but, unfortunately, it seems sometimes to be treated no better. The curves at intersections are necessarily very sharp, and in order to diminish the amount of power required and the wear on the rails (as well as on tires), they require oiling at least once a day for heavy traffic, while the rate at which cars run over special work should be strictly regulated to a low speed. The groove of the rail and the tongue switches require to be constantly cleared of the dirt which inevitably collects, and if not removed causes great inconvenience. The life of such work may be appreciably prolonged by such attention, and when one considers the cost of renewal and the consequent interference to traffic while doing so, it will be readily seen that it pays in the end.

TEE RAIL SPECIAL WORK.

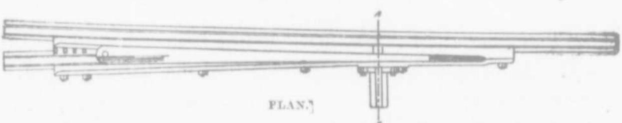
FIG. 24.
TONGUE SWITCH.

LONGITUDINAL SECTION.

SECTION A-A. SECTION B-B. SECTION C-C. SECTION D-D.

FIG. 25.
TONGUE SWITCH WITH SPRING.

ELEVATION.



PLAN.

LONGITUDINAL SECTION OF TONGUE.

SECTION A-A.

FIG. 26.
MATE.

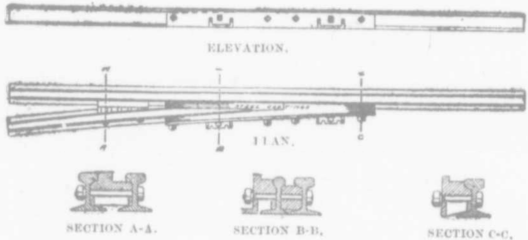
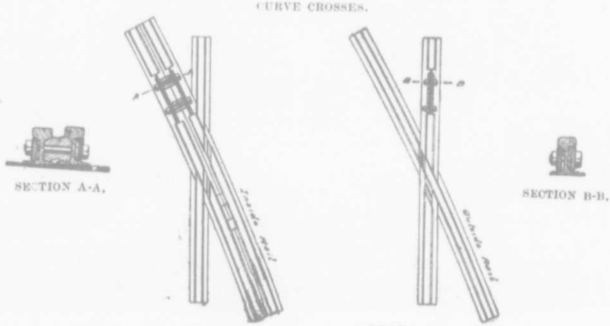


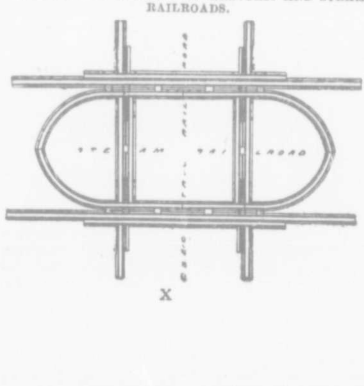
FIG. 27.
CURVE CROSSES.



PLAN.

FIG. 28.

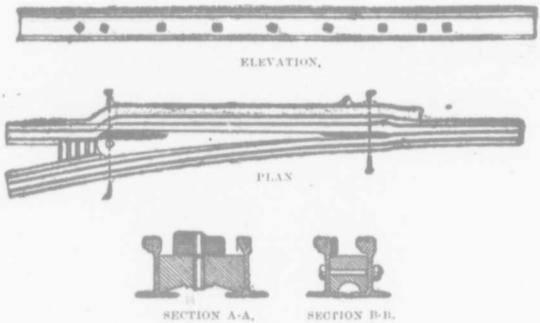
DIAMOND FOR CROSSING OF ELECTRIC AND STEAM RAILROADS.



X

GIRDER RAIL SPECIAL WORK.

FIG. 29.
TONGUE SWITCH.



PLATE

FIG. 30.

BLIND SWITCH.

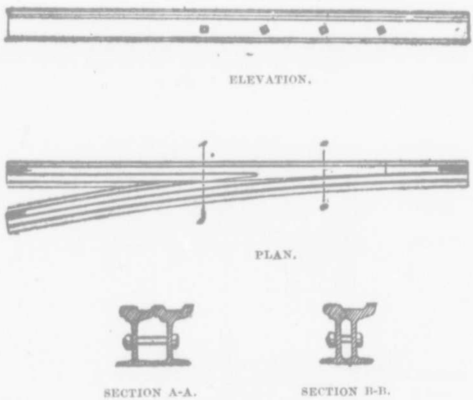


FIG. 31.

MATE.

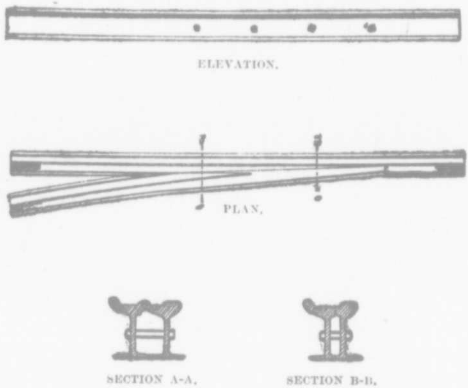
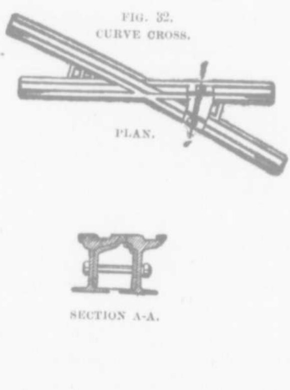


FIG. 32.

CURVE CROSS.



DISCUSSION.

Mr. A. E. Childs.

Mr. A. E. Childs said :—

This paper is one which the Society, I am sure, is very glad to have, as it is well written, and contains practically all the latest ideas on special track work for electric railways. It is written logically and clearly, and although there are a number of small points with which we may not all be able to agree, still the main matter of the paper is thoroughly in line with the latest ideas and the latest practice.

The question of track construction has been studied by steam railway engineers for nearly half a century, and although great advances have been made in the practice, still there are many changes going on, which indicate, that although the present system of building railways is a thoroughly good one, it is not yet all that railway men desire.

It is of course well known that the rolling stock of electric railroads, owing to the addition of motors, is much heavier than that of the old horse car lines, and that the speeds are also much greater. These two facts have caused heavier rails to be used and a higher class of steel to be put into them in order to insure long life and freedom from breakage. Although a few years ago 65 and 75 pound rails were considered to be very heavy, it is now a fact that the elevated railroads of New York are using a section weighing 90 pounds per yard, in an effort to secure the best possible construction. A few weeks ago, the Pennsylvania R.R. decided, at a meeting of its directors, to use in the future nothing but 60 foot rails, each weighing one ton, on the division between Jersey City and Philadelphia. This is the heaviest rail yet manufactured; but although we now consider them to be excessively heavy, there is nothing to assure us that in a few years more heavier rails may not be used.

The American Society of Civil Engineers has considered fully the question of standard sections of rails for steam roads, as well as a uniformity of method of testing such rails; and from the fact that steam engineers are giving this question so much attention, it is advisable for electric railroad men to adopt their practice as much as possible, especially on surburban and interurban lines.

As to the wearing qualities of rails, it is a well-known fact that the higher the percentage of carbon the longer will be the life of the rail, and at the same time it is also well known that too much carbon renders the rail brittle and liable to breakage. This latter fact has usually influenced engineers in specifying a rail to have the carbon low in percentage, as the rolling mills are very liable to exceed the amount specified and thus get too near the limit. The amount of carbon should in each case be proportioned to the weight of the rail to get the best results.

Owing to the increase in size of the street cars using electric motive power, it has been necessary to make the track as rigid as possible, as the lurching motion of long cars carried on four wheels with a rigid frame is very severe on the track when the cars attain a high speed, and this lurching motion is not only unpleasant to the passengers, but is very injurious to the track; in fact, in this city (Philadelphia), the result of the pounding motion produced by the cars is already seen on several lines at the rail joints. The fruitful source of bad rail joints is the fact that the steel mills have been furnishing a very soft, low carbon steel for angle and fish plates, and as this steel has a low elastic limit and tensile strength, it takes a set under a blow from a wheel moving at a high speed. To illustrate this, the N. Y. C. & H. R. R.R. tested some 80lb. rail and angle plate steel furnished from the same steel mill. The tensile strength of the rail steel was 120,000 lbs. compared with 57,000 lbs. for the angle plate steel. The elastic limits were 60,000 lbs. and 30,000 lbs. respectively. A test was made as to the breaking strength under a blow delivered by a falling weight, and the rail steel stood 2,000 lbs., falling 20 ft., while the angle plate steel only stood 2,000 lbs., falling 6 ft., and thus it would be seen that a track, although having good steel rails, may be weak at the joints owing to the inferior metal used at these points. The remedy to this is a higher carbon steel. The present practice is to leave the matter of composition of the rails entirely to the mills, and not to provide an inspector to make tests on the material delivered, and it would no doubt prove a very valuable aid to the railroads for them to appoint inspectors to make tests on the rails delivered to their companies, and thus preventing the mills from delivering bad material, which they frequently do at present, as it means a loss of thousands of dollars to them to reject their own bad material.

Mr. E. A. Stone.

Mr. Stone in reply said :—

Mr. Childs' remarks on rails are very interesting, but when referring to their wearing qualities, he would seem to lay rather too great an amount of stress on their chemical composition. While this is no doubt very important, yet the mechanical treatment which they undergo during manufacture is most probably of still greater importance. The tendency at the present time being to lessen the cost of production by quicker rolling at higher temperatures, the attempt is made to bring these rails nearer to the standard of the first steel rails produced by modifying the chemical composition. That the attempt has not been altogether successful is apparent in places where 56 lb. rails, after 10 to 12 years wear, may be seen with as good, if not better, joints than rails, 30 % heavier, which have been in the track only 2 or 3 years. To increase the percentage of carbon above a certain point becomes dangerous, as brittle rails in a cold climate are certainly not very desirable.

The long rails referred to have certainly the advantage of requiring fewer joints, and so cost less for fastenings ; but against this there is the greater difficulty in handling, higher cost of transport, wider joints for expansion, and greater liability to get crooked during transport.

Statement showing Tests of Paving Brick
made for
The Common Council, City of Dunkirk, N.Y.
by J. K. MacDonald, C.E.
City Engineer.

September 1895.

NAME	TUMBLING OR ABRASION			MOISTURE OR ABSORPTION			COMPRESSION
	Weight of Brick before Tumbling lbs.	Weight of Brick after Tumbling 8 hours. lbs.	Percentage of Loss.	Weight of Brick before Wetting lbs.	Weight of Brick after Wetting. 24 hours. lbs.	Percentage of Gain.	lbs. per sq. inch.
Hallaway Block	7·969	6·500	18·5 8	7·922	8·125	2·5 6	5467 8
Metropolitan Block	10·156	9·531	6·2 2	10·219	10·265	0·4 2	7526 5
Preston	6·468	5·875	9·2 6	6·422	6·562	2·1 4	10356 1
Darlington	7·000	6·359	9·2 6	7·047	7·312	2·7 7	6776 6
Canton Standard	7·109	6·468	9·1 5	7·062	7·094	0·4 2	8755 2
Park Standard	6·453	5·969	7·6 4	6·609	6·758	2·2 5	7786 4
Brady Run	7·171	6·687	6·8 3	7·254	7·281	0·6 3	7859 3
Grant	7·046	6·687	5·1 1	6·922	6·937	0·2 1	5642 7
McMahon-Porter	7·156	6·468	9·7 7	7·239	7·492	3·5 8	4941 9

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CHANGES AND CORRECTIONS.

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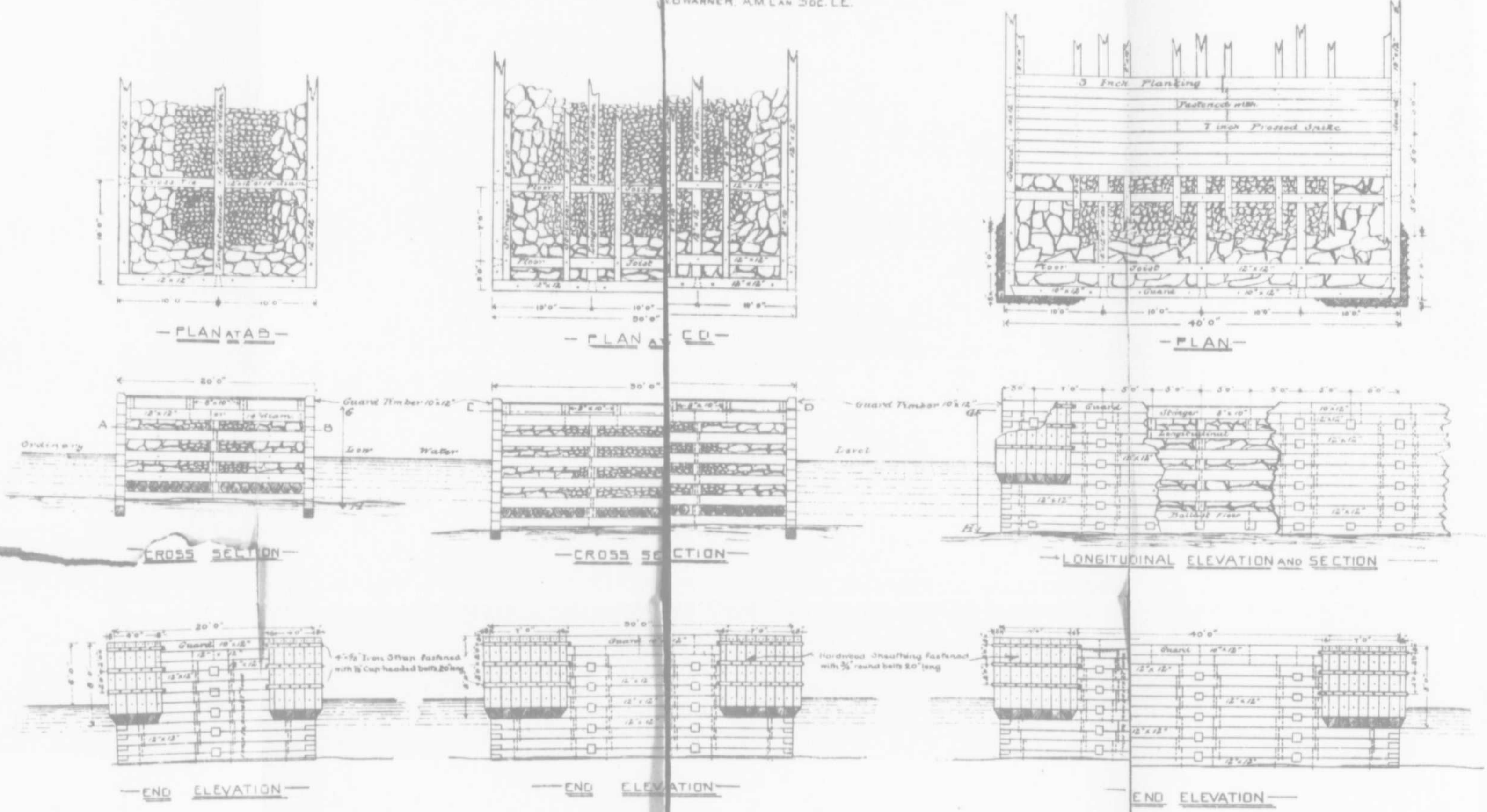
—TABLE OF QUANTITIES— —AND— —PLAN— OF —CRIBWORK PIERS—

A. G. WARNER. A.M. CAN. SOC. C.E.



SIZE OF CRIB	Crib: Face Timbers F.B.M.	Crib: Bolts lbs	Crib: Cross Ties F.B.M.	Crib: Cross Ties L.F.	Crib: Bolts lbs	Crib: Longitudinals F.B.M.	Crib: Longitudinals L.F.	Crib: Bolts lbs	Crib: Stone cubic yards	Ends: F.B.M.	Ends: Bolts lbs	Ends: Guard F.B.M.	Ends: Bolts lbs
10'0" × 20'0"	480	60.62	216.	18	4.76	120.	10.	4.76	11.92	960.	121.28	400	39.2
10'0" × 25'0"	-	-	270.	22.5	4.76	120	10.	4.76	15.335	1200.	121.28	500.	49
10'0" × 30'0"	-	-	324.	27.	9.52	240.	20.	9.52	18.386	1440.	181.92	600.	58.8
10'0" × 35'0"	-	-	378.	31.5	9.52	240.	20.	9.52	21.80	1680.	181.92	700.	68.6
10'0" × 40'0"	-	-	432.	36.	14.28	360.	30.	14.28	24.85	1920.	242.5	800.	78.4
10'0" × 50'0"	-	-	540.	45.	19.04	480.	40	19.04	31.31	2400.	303.2	1000.	98.
10'0" × 60'0"	-	-	648	54	23.81	600	30	23.81	37.77	2880.	363.84	1200	117.6
For Depth	Multiply by $\frac{d}{2}$ = D		Multiply by $\frac{d-4.0}{2}$ = D							Multiply by $\frac{d}{2}$ = Q		Add to Total	
For Total	D × L = Q		D × L = Q										

SIZE OF CRIB	Ballast Floor: Cross Ties F.B.M.	Ballast Floor: Cross Ties L.F.	Ballast Floor: Longitudinals F.B.M.	Ballast Floor: Longitudinals L.F.	Ballast Floor: Bolts lbs	Ballast Floor: Poles L.F.	Flooring: Joists F.B.M.	Flooring: Bolts lbs	Flooring: Stringers F.B.M.	Flooring: Bolts lbs	Flooring: Guard F.B.M.	Flooring: Bolts lbs	Flooring: 3" Plank F.B.M.	Flooring: Spike 7" Pressed lbs	Sheathing: Hardwood F.B.M.	Sheathing: Bolts lbs	Sheathing: Strap lbs	Sheathing: Bolts lbs
10'0" × 20'0"	456	38	120.	10.	10.05	196.	456.	10.05	258.72	37.24	200	19.6	514.8	17.55	3"×8"×80'-2 1152	238.08	383.3	210.3
10'0" × 25'0"	570.	47.5	120.	10	10.05	260.	570.	10.05	323.4	46.55	200	19.6	657.8	23.40	3"×8"×80'-2 1408	287.68	383.3	252.
10'0" × 30'0"	684.	57.	240	20	20.10	310.	684.	20.10	388.08	55.86	200	19.6	800.8	26.33	3"×8"×80'-2 1920.	380.88	383.3	336.
10'0" × 35'0"	798.	66.5	240.	20.	20.10	370.	798.	20.10	452.76	65.17	200.	19.6	943.8	29.25	1920.	380.88	383.3	336.
10'0" × 40'0"	912.	76.	360.	30.	30.15	420.	912.	30.15	517.44	74.48	200.	19.6	1086.8	32.18	1920.	380.88	383.3	336.
10'0" × 50'0"	1140.	95.	480.	40.	40.20	520.	1140.	40.20	646.8	93.10	200	19.6	1372.8	40.35	1920	380.88	383.3	336
10'0" × 60'0"	1368	114.	600	50	50.25	630.	1368.	50.25	776.16	111.72	200	19.6	1658.8	46.80	1920	380.88	383.3	336.
For Depth / For Total	Multiply by L = Q														Add to Total			

— WEIGHT OF BOLTS —

Bolts for Face Timbers	1"sq. 27" long weigh	7.58 lbs
" " Cross Ties	1" round 24"	5.28 "
" " Longitudinals	-	-
" " Guard	1" round 22"	4.9 "
" " Stringers		
" " Sheathing	⅝ round 20"	2.48 "
Iron Strap	per ft. run	3.833 "
Spike for Planking	7" pressed	4.5 ozs.

— NOTE —

Depth of Crib is taken as being from bottom of Guard to bottom of lowest Face Timber, see G. H in Cross Section and Longitudinal Elevation. In cribs of uneven depth; in construction, either add Face timber shown hatched in Cross Sections, or omit Joists, laying stringers across longitudinals. In taking out quantities for Cribs of uneven depths in the Formula $\frac{d-4.0}{2}$, the decimal is omitted. i.e. Crib, 17'0" depth. $\frac{17.0-4.0}{2}$ = 6.5 Multiply by 6 only. Quantities for Cross Ties and Longitudinals are given for both square and round timber.

— MEANING OF SYMBOLS —

d = given depth of Crib in feet
D = quantities req'd of given depth 10'0" long
L = given length of Crib in feet
Q = quantities required