始

早稻田大學

理工學部紀要

第九號

昭和八年十二月

MEMOIRS OF THE FACULTY OF SCIENCE & ENGINEERING, WASEDA UNIVERSITY.

TŌKYŌ, JAPAN.

No. 9.

1933.

Department of Architecture,
Waseda University.

PUBLISHED IN WASEDA UNIVERSITY.

露光量違いの為重複撮影

EDITING COMMITTEE.

TADAOKI YAMAMOTO, *Kōgakuhakushi*, . . . PROF. *of Electrical Engineering.*
DEAN OF THE FACULTY

IWAO OKI, *Kōgakuhakushi*, ... PROF. *of Mechanical Engineering.*

IWAKI KIKKAWA, *Kōgakuhakushi*, ... PROF. *of Mine Engineering.*

TACHŪ NAITŌ, *Kōgakuhakushi*, ... PROF. *of Architecture.*

KIŪHEI KOBAYASHI, *Kōgakuhakushi*, . . . PROF. *of Applied Chemistry.*

All communications relating to this collection should be addressed to the Dean of the Faculty.

Errata

P. 103
"Reprinted from the Proceedings ...."
should read "Proceedings ...."
P. 113 No. 8
"Abraded" should read "Abridged"
"Preferen" should read "Prepared"

早稻田大學

理工學部紀要

第九號

昭和八年十二月

MEMOIRS
OF THE
FACULTY OF SCIENCE & ENGINEERING,
WASEDA UNIVERSITY.

TOKYO, JAPAN.

No. 9.

1933.

Department of Architecture,
Waseda University.

PUBLISHED IN WASEDA UNIVERSITY.

All communications relating to this collection should be addressed to the Dean of the Faculty.

Errata

P. 103
"Reprinted from the Proceedings......" should read "Proceedings......"

P. 113 No. 8
"Abridjed" should read "Abridged"
"Prefered" should read "Prepared"

早稻田大學

理工學部紀要

第九號

昭和八年十二月

MEMOIRS
OF THE
FACULTY OF SCIENCE & ENGINEERING, WASEDA UNIVERSITY.

TŌKYŌ, JAPAN.

No. 9.

1933.

Department of Architecture,
Waseda University.

PUBLISHED IN WASEDA UNIVERSITY.

Introductory Note.

The present memoirs are a collection of abridged papers and abstracts dealing the latest studies by the Faculty of Architecture, and the original articles have previously been published in other scientific magazines.

Readers interested in any article of this volume are referred to the original for fuller information.

We acknowledge our indebtedness to the following journals for the original publications :—

(1) Journal of the Institute of Japanese Architects. (J. of I. J. A.)

(2) Journal of Architecture in Waseda University. (J. of A. W. U.)

Department of Architecture,
Faculty of Science and Engineering,
Waseda University.

Waseda, Tokyo,
November, 1933.

CONTENTS.

PAGE

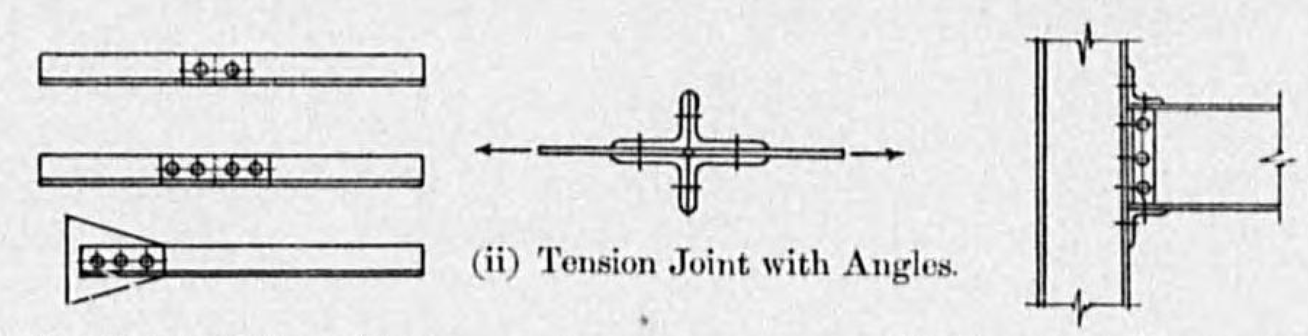
On the Strength of Structural Steel Connections.

BY

Prof. **Tachu Naito.** *Kōgakuhakushi.*

(1) Introduction.

Connections of structural steel can be classified as (a) those transmitting direct stress, tension or compression, and (b) those transmitting moment as in the case of beam and column joints. (Fig. 1)



(ii) Tension Joint with Angles.

(i) Joint for Direct Force. (iii) Beam to Column Joint.

Fig. 1.

The methods of obtaining such connections are generally as follows:

1. With rivets,
2. With electric welds,
3. With combination of rivets and welds,
4. With steel bar reinforcements in the case when the structure is embedded in concrete.

Considerable amount of experimental data concerning shearing connections are available but reliable data in regard to connections with angles and the strength and rigidity of beam to column joints are relatively few. Researches into the latter type of connections are of utmost value in the design of earthquake resisting construction. The author has made a series of experiments for the last ten years, and during the process he has accumulated data concerning the elastic behavior of several different connections. A summary of these experiments is presented in this paper.

The author acknowledges sincere gratitude for the untiring efforts and assistance afforded him in conducting these experiments to Mr. A. Tsuruta and to the students of the Architecture Department of Waseda University.

(2) Shearing Connections.

A. Riveted Joints.

The test performed on riveted joints of the type shown in Fig. 2 indicates that the rivets resist external force with friction during the first stage. When the external force becomes such magnitude as to produce stress near the allowable limit of shear, the frictional force is destroyed and slip begins. For example, when $f_s = 700\ \text{kg/cm}^2$, the slip is about 0.05 mm. (Fig. 2). When the magnitude of the stress is increased, however, the external force is chiefly resisted by the shearing strength and the ultimate resistance is generally, above the value expected. This is attributable, in the author's opinion, to the fact that the shaft of the rivet becomes enlarged in the driving, thereby filling up the rivet hole. Table 1 shows that a 3/4 in diameter cold rivet, when driven, gives strength which is equivalent to the value usually assigned to a 13/16 in. or 7/8 in. diameter rivet.

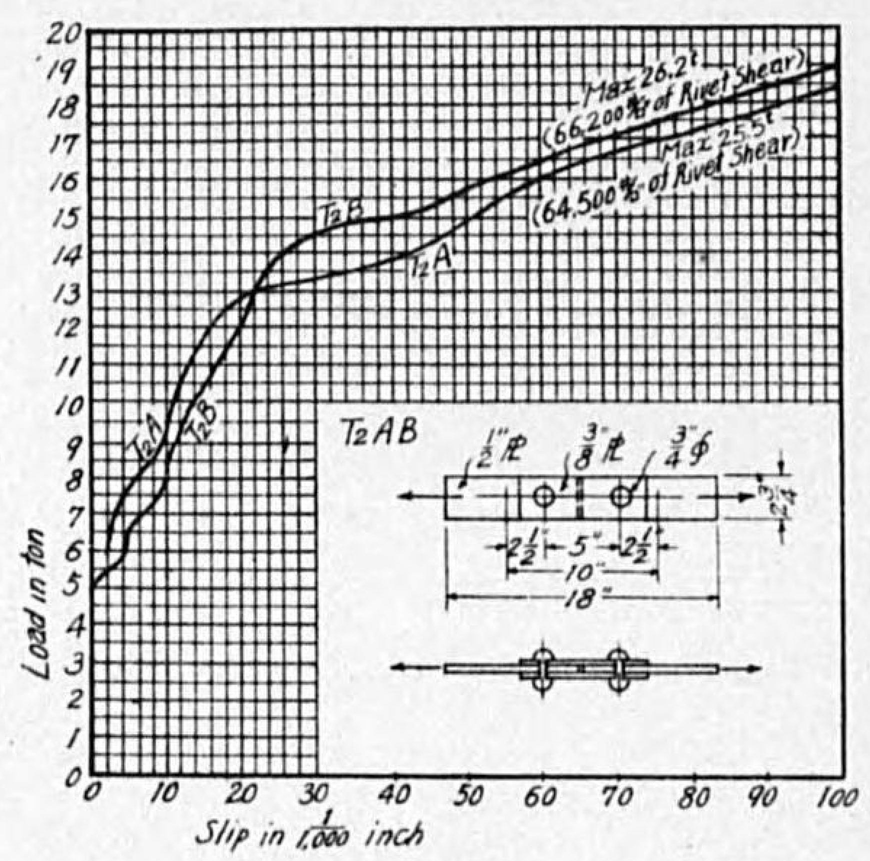


Fig. 2.

Table 1. Shearing Strength of Rivet. (Test Result)
Unit of the figures in ( ) is kg/mm².

Dia. of cold rivet 3/4 in.	Breaking load in tons	Strength lbs/sq.in. as 3/4 in. dia.	Strength lbs/sq.in. as 13/16 in. dia.	Strength lbs/sq.in. as 7/8 in. dia.
T 2 *A*	25.5	64,500 (46) kg/mm²	56,000 (40) kg/mm²	47,000 (33) kg/mm²
T 2 *B*	26.2	66,200 (47)	57,000 (40)	48,000 (34)
3-5/8 in. rivet	45	as 5/8 in. dia. 56,500 (40)	as 11/16 in. dia. 45,000 (32)	

B. Welded Joints.

Welds produced by a skilled welder show sufficient strength. Fig. 3 shows the details of the test specimens, and Fig. 4 the test results. The joints (whether riveted, welded, or the combination of the two) were designed on the basis of six 5/8 in. diameter rivets in single shear. The results of the experiments have shown a much higher strength than was expected.

It is to be noticed that the welded specimens *B* and *D* deformed

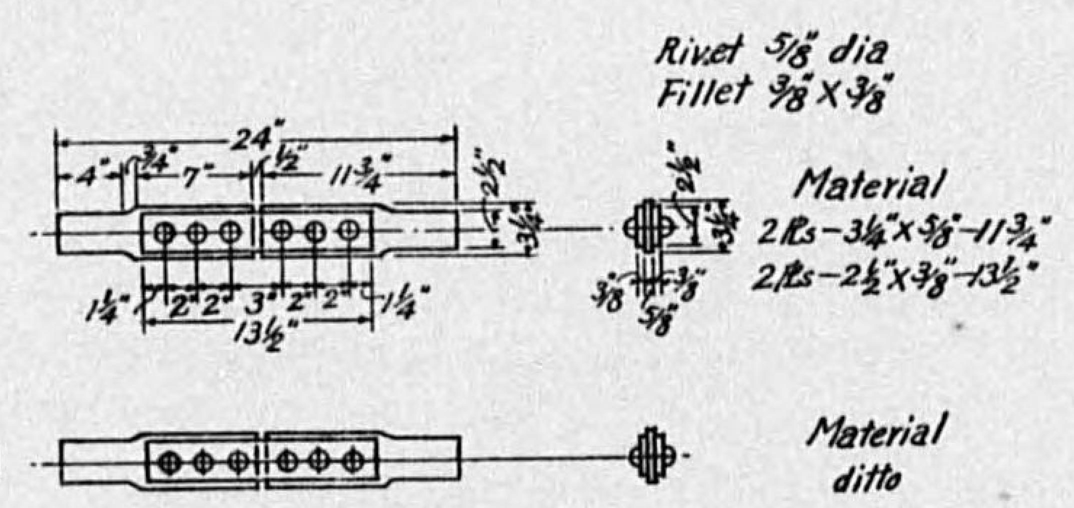


Fig. 3. (A) Details of Test piece

(B)
Plug weld

(C)
Rivet and Weld combined

(D)
Weld only

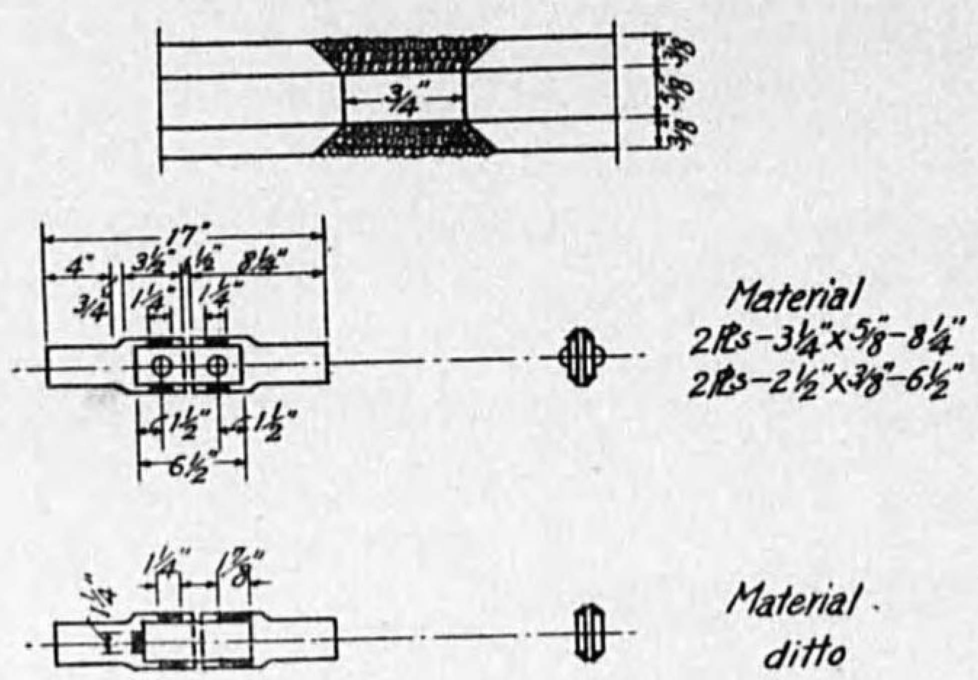


Fig. 3. (B) Details of Test piece

very little, indicating that a welded connection offers greater rigidity than a riveted connection such as specimen A illustrated in Fig. 4.

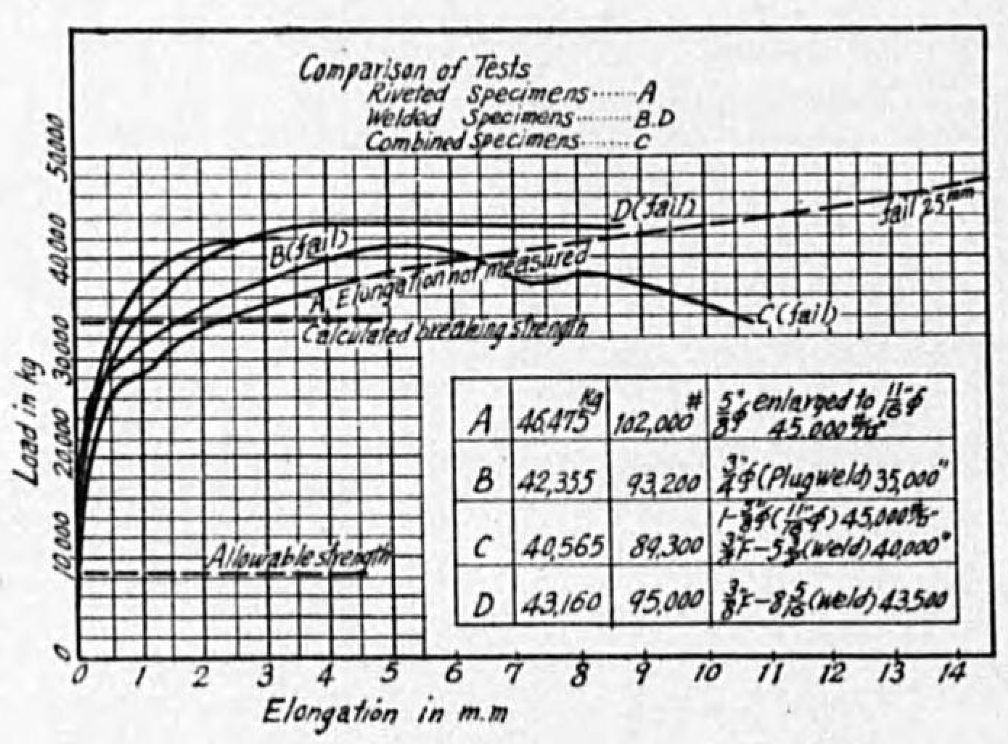


Fig. 4.

C. Rivets and Welds Used Together.

In the light of the author's experiments, the strength of the joints obtained by the use of a combination of rivets and welds may be regarded as about 90% of the sum total of their individual strength (refer. Fig. 3-c.) However, a further study of the combination joints is now under progress from which some additional information will be available in the near future.

(3) *L* Connections in Tension.

(1931 tests, laboratory of Waseda University.)

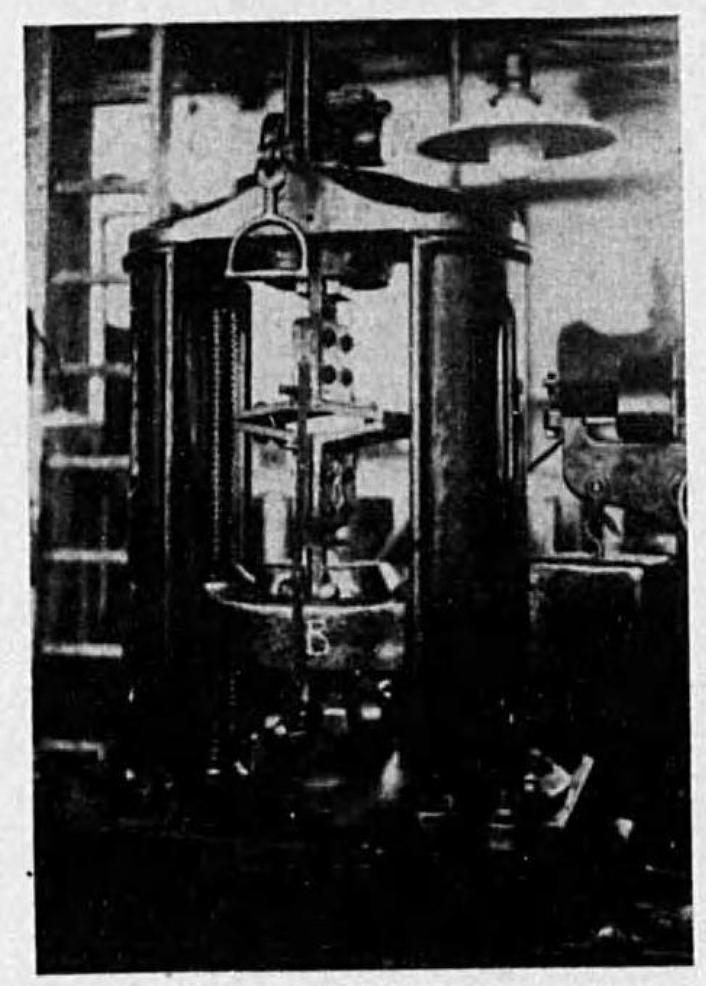


Fig. 5.

When tension members are joined by connection angles in the manner shown in Figs. 5 and 6, the angles will assume such shapes as shown in the above figures. The rivets A will be highly stressed in tension and the maximum bending moment generally occurs at point B.

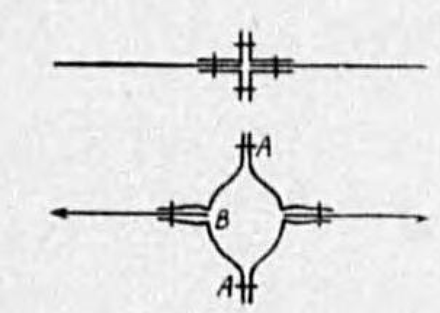
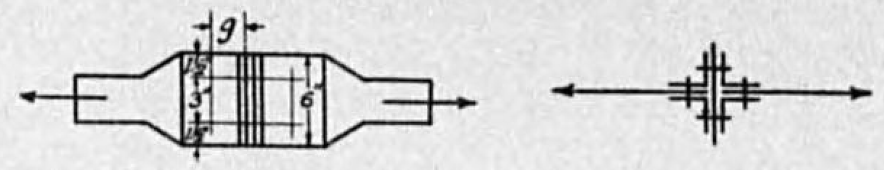


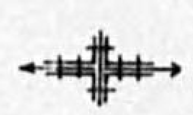
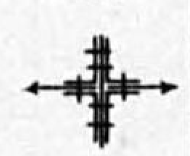
Fig. 6.

When the angle is thinner than 9 mm., it will easily bend and the rivets A will be merely pulled, and the strength of the joint will be governed by the tensile strength of the rivets. However, when the angle is thicker than the above, it will fail at point B. (The above is from the experiments made in 1920). Table 2 shows the results of the tests.

Table 2. *L* Connections in Tension.

(Tests of 1920.) Rivet dia. 3/4 in.



Size of *L*	*t* (in)	Gauge (in)	Rivet pitch (in)	Breaking load (tons)	Fracture	Safe tension load for one rivet (lbs)
3″×3″	3/8	1¾	3	31	plate	
”	”	”	”	34	failed	5,000
”	1/2	”	”	39		
4″×4″	3/8	2¼	3	24.9	*L* broken	
”	”	”	”	27	”	4,000
”	”	”	”	24.4	”	
”	”	”	”	21.4	”	
”	7/16	”	”	35.6	”	
”	1/2	”	”	39.2	rivet sheared	5,000
6″×4″	7/16	”	”	28.2	*L* broken	
	”	”	”	36.6	”	5,000
	1/2	”	”	31.6	”	
	”	”	”	32.2	”	
4″×6″	7/16	”	”	38.2	*L* and plate failed	
	”	”	”	38.6	rivet sheared	
	1/2	”	”	38.2	”	5,000
	”	”	”	29.2	*L* broken	

Note: Breaking load is comparatively high for thin *L* though it suffers large strain.
Thick *L* is apt to be easily broken at the root under a little strain.

If the angle is extremely thick or if the rivet diameter is very small, the strain will be comparatively small and the rivets will fail in tension. (Experiments of 1931)

In this type of connection, the strain is considerable which makes it difficult to decide upon the safe strength of the rivets. If a strain of 0.010″–0.030″ is allowed, the tensional strength of a 3/4″ diameter rivet may be regarded as from 4,000 to 5,000 pounds. This type of connection is not recommended, owing to the fact that the strain is large as has been mentioned before. (Exp. 1920)

(4) Ribbed Angle Connections in Tension.

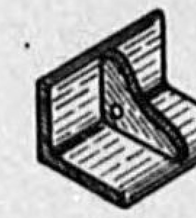


Fig. 7.

Top angles in beam and column connections are liable to bend with resultant loss of strength in tension as has been pointed out previously. The author has, therefore, designed a ribbed angle to give additional strength and an angle of this type is shown in Fig. 7.

In order to compare the strength of these ribbed angles with ordinary angles, four test pieces were prepared as shown in Fig. 8. Specimens *A* and *B* are ordinary angles while *C* and *D* are ribbed. (Some of the ribbed *Ls* are of cast steel). 16 mm. rivets and 17 mm. holes were used.

Specimens *A* and *B* produced large strain while *C* and *D* showed small strain.

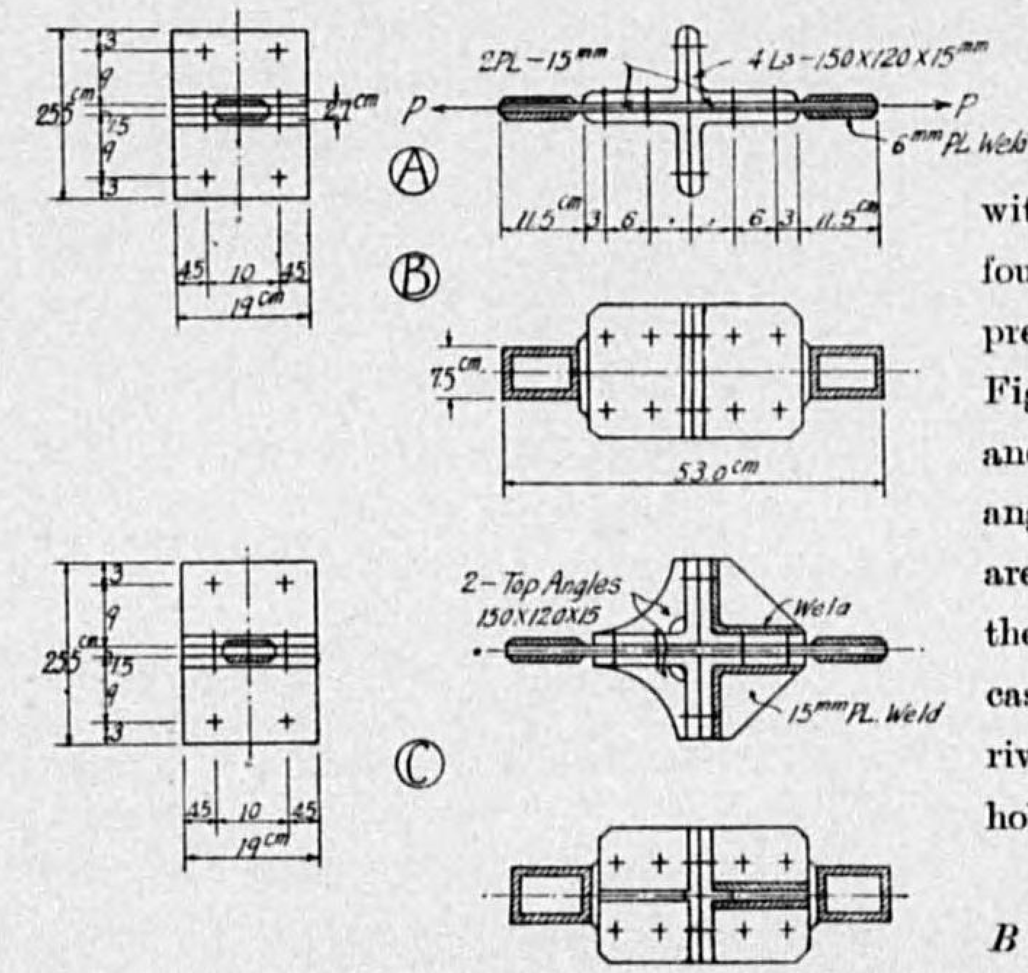


Fig. 8. (A) Test pieces for Top Ls in Tension

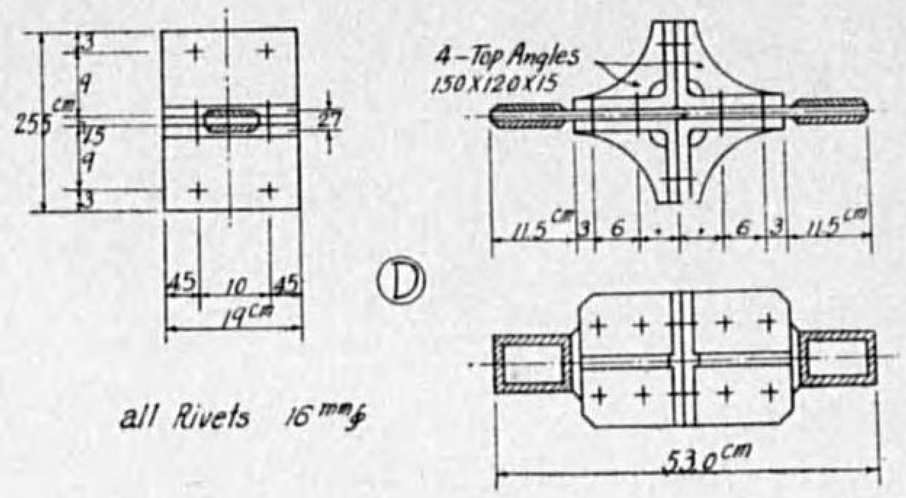


Fig. 8. (B) Test pieces for Top Ls in Tension

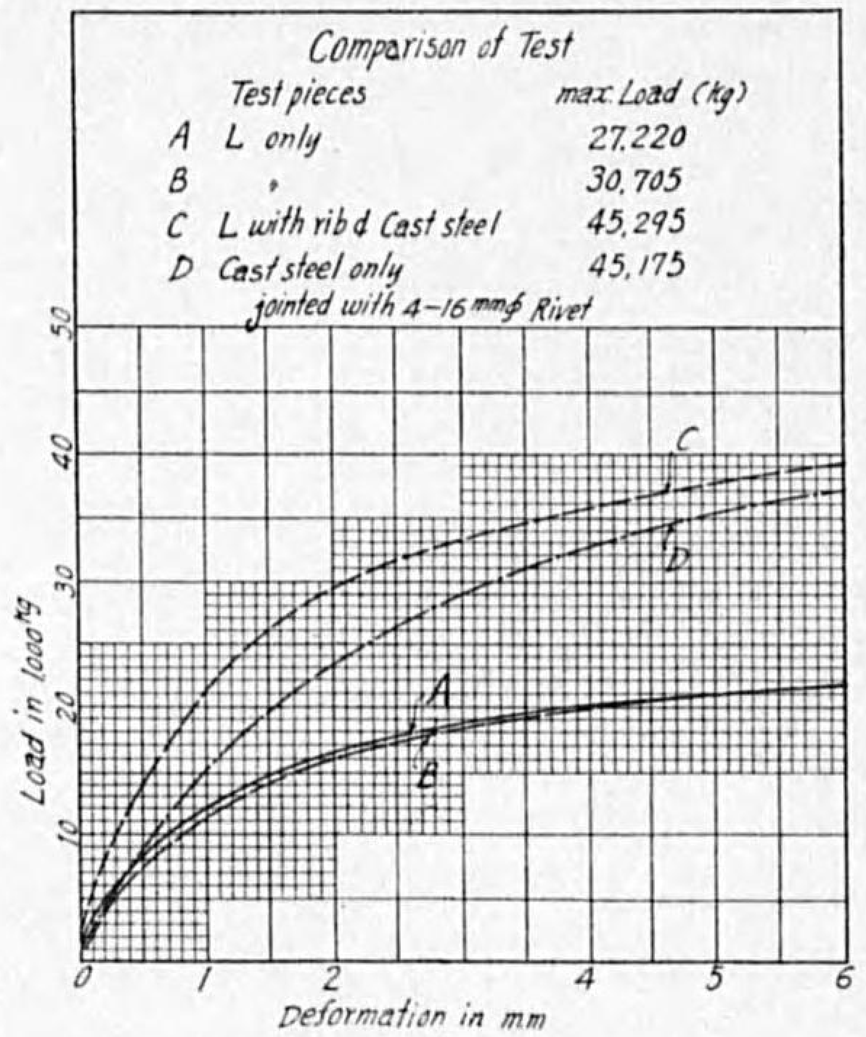


Fig. 9.

(Fig. 9) The rivets in specimens C and D seemed to have resisted full tension. Table 3 shows the stress in the rivets at failure. In A and B, the rivets were subjected to an eccentric tension, owing to the deformation of the angles and the average stress of rivets at failure was 3,000–3,400 kg/cm². In C and D, the stress at failure was about 5,000 kg/cm² which is more than the full tensile strength of the steel. This may be due to the enlargement of the rivet shaft in filling up the 17 mm. rivet hole.

Concerning the Strain of Angle Connections, A and B.

When the angle is under load, as shown in Fig. 10, it bends and undergoes deformation. The mode of deformation may be expressed mathematically as follows :—

Table 3. L Connections against Tension (Test Result).

Test piece	Max. load (kg)	For one rivet (kg)	Strength of rivet in tension as 17 mm. dia.
A	27,220	6,800	30 kg/mm²
B	30,705	7,676	34
C	45,295	11,324	50
D	45,175	11,294	49.8

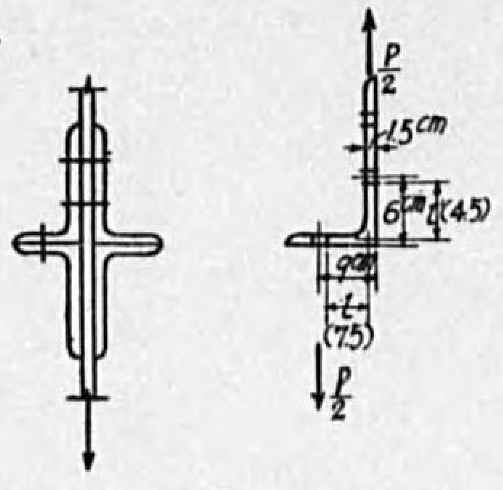


Fig. 10.

$$\delta \doteqdot \frac{\frac{P}{2} l^3}{3EI} + \frac{\frac{P}{2} l^2 l'}{4EI}$$

$l = 7.5$ cm $\quad l' = 4.5$ cm

where $t = 1.5$ cm $\quad b = 19$ cm

then $\delta\,(\text{mm}) = 0.09P\,(\text{ton})$,

when $P = 10$ t $\quad \delta = 0.9$ mm......(agreed with experiment)

$P = 15$ t $\quad \delta = 1.35$ mm......(1.5 mm in experiment)

(5) Beam to Column Connections (I).

(1930 Experiments, Laboratory of Waseda University)

To find the strength of beam to column connection, specimens A and D in Figs. 18 and 19 were tested as a double cantilever, as shown in Fig. 11. Specimen B (Fig. 18) of reinforced concrete was also made and tested in the same manner. Specimen C is the combination of specimens A and B, and specimen E was similarly made by the combination of D and B. Specimen C and specimen E were tested in order to determine how much strength the encasement in reinforced concrete would add to the rivet jointed beams A and D.

Calculation of Strength of Test Pieces.

Test piece (A) Fig. 18.

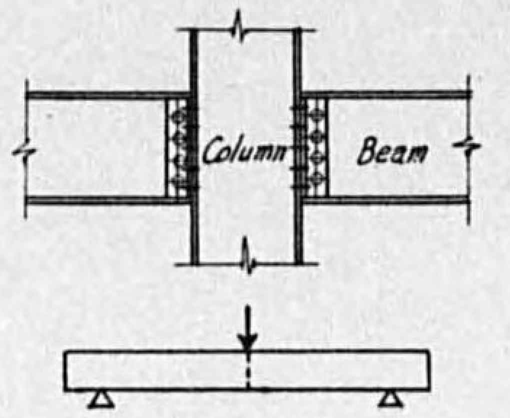


Fig. 11.

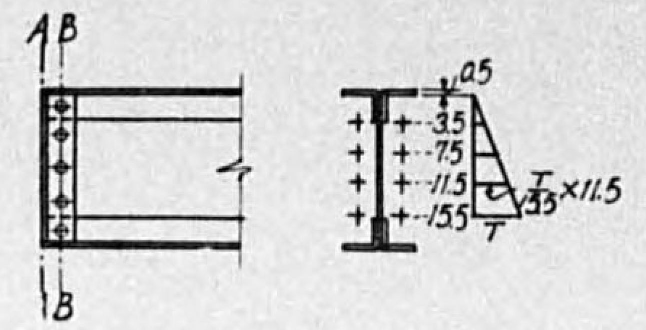


Fig. 12.

Strength at the A-A rivet line in Fig. 12.

The rivets on the lower side being under tension and the angle deforming, the compressive force will concentrate on the top. Assuming that the center of compression is 0.5 cm. from the top, the rivets will be stressed as shown in Fig. 12. Then,

$$R.M.=\frac{T}{15.5}(15.5^2+11.5^2+7.5^2+3.5^2)\times 2$$

$$=57\,T\,\text{cm.t}=0.57\,T\,\text{t.m.}\ldots\ldots T \text{ is rivet tension.}$$

Let a 9 mm rivet be enlarged to a 11 mm one, then,

$$a=\frac{\pi\times 1.1^2}{4}=0.96\ \text{cm}^2$$

As the tension rivet suffers eccentric stress, we assume

$$f_t=3{,}500\ \text{kg/cm}^2\ ;$$

$$T=.96\times 3{,}500=3.4\ \text{ton.}$$

$$\therefore\quad R.M.=.57\times 3.4=1.93\ \text{t.m.}$$

$$B.M.=\frac{Pl}{4}=\frac{P}{4}\times 1\ \text{t.m.}\qquad \therefore\quad \frac{P}{4}=1.93\ \text{t}$$

$$\therefore\quad P=7.72\ \text{t}$$

The rivet failed in tension when the test load was 7.95 t. (Table 4.)

Compression side

$$\Sigma C=\Sigma T$$

Here $\quad \Sigma T=2\times T\times\left(1+\frac{11.5}{15.5}+\ldots\ldots\right)=\frac{2\times T}{15.5}(15.5+11.5+7.5+3.5)$

$$=\frac{T}{15.5}\times 38\times 2=4.9\,T$$

$$\therefore\quad \Sigma T=4.9\times 3.4=16.5\ \text{t}=\Sigma C$$

flange area $\qquad .5\times 8=4\ \text{cm}^2$

$$\frac{16.5}{4}=4{,}125\ \text{kg/cm}^2\qquad \text{O.K.}$$

Neutral Axis of the Joint.

In the L of fig. 13 assume that the rivet does not elongate during the first stage of the L deformation, and also assume that the inflection point of the angle leg is located at $0.6l$,

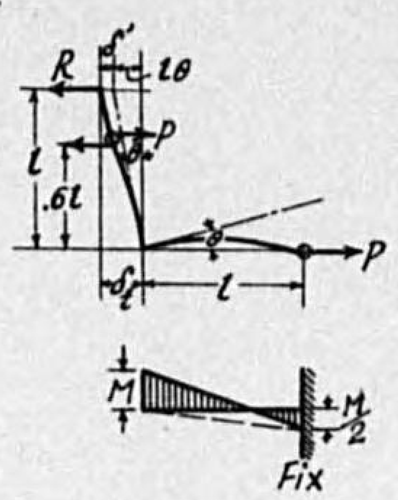


Fig. 13.

$$\delta_t=l\theta+\delta'\qquad M=0.6Pl$$

$$\theta=\frac{1}{EI}\left(\frac{Ml}{2}-\frac{Ml}{2\times 2}\right)=\frac{Ml}{4EI}=\frac{0.6Pl^2}{4EI}$$

$$\therefore\quad l\theta=.15\frac{Pl^3}{EI}$$

$$\delta'=\frac{0.13Pl^3}{EI}$$

$$\therefore\quad \delta_t=\frac{0.13Pl^3}{EI}+\frac{0.15Pl^3}{EI}=\frac{0.28Pl^3}{EI}$$

$$=0.28\frac{Pl^3}{E\frac{At^2}{12}}=3.36\frac{Pl^3}{EAt^2}$$

The deformation caused by compression (here neglecting the slip and deformation of the rivet) is :

$\delta_c=\dfrac{Pl}{AE}$ let the ratio of deformation of each be e,

$$e=\frac{\delta_t}{\delta_c}=\frac{3.36\frac{Pl^3}{EAt^2}}{\frac{Pl}{AE}}=3.36\left(\frac{l}{t}\right)^2$$ (denominate it " modular ratio of L joint ")

here assume $\qquad l=\text{gage}-\dfrac{d}{2}\qquad (d=\text{dia. of rivet})$

Now assuming that the gauge is 22.5 for the 40 mm L,

$$l=22.5-9.5=17$$

$$\therefore\quad e=3.36\times\left(\frac{17}{6}\right)^2=27$$

Then, the neutral axis can be found as long as the top parts of the beam do not abut themselves :—

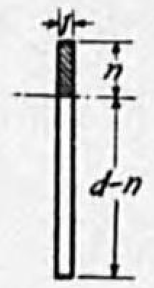


Fig. 14.

$$\frac{n^2}{2}=\frac{(d-n)^2}{2e} \quad \therefore \quad n=\frac{d-n}{\sqrt{e}} \quad \therefore \quad n=\frac{d}{1+\sqrt{e}}$$

when $e=27, \quad n=0.162d$

If the top parts of the joint, however, abut each other, these parts will resist compression at the part of the top-flange of the L, so that the neutral axis will rise up.

Regarding e as 27 for the part below the neutral axis, we see that the N.A. is at a point 1 cm down from the top.

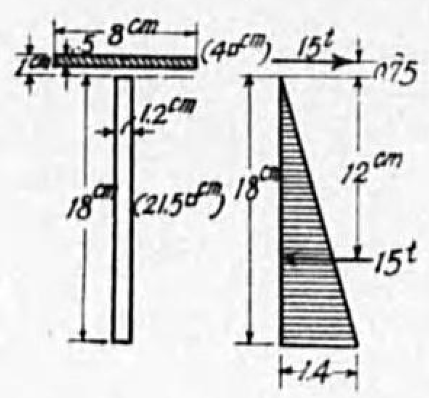


Fig. 15.

$$I=\frac{21.5\times 18^2}{3\times 27}+4\times(.75)^2+\frac{4\times .5^2}{12}$$

$$=87+2.3+0.083=90$$

(bottom) $S_t=\frac{90}{18}\times 27=135$

(top) $S_c=\frac{90}{1}=90$

The strength of the tension side rivets will be $0.96\times 2\times 3{,}500=6.8$ t

The pitch of the rivets is 4 cm, and the sum of the thicknesses is 1.2 cm. Therefore, the tension of the L becomes :

$$\frac{6.8}{4\times 1.2}=1.4 \text{ t/sq. cm.}$$

Then $R.M.=1.35\times 1.4=1.9$ t.m.

or $R.M.=.9\times 4{,}000=3.6$ t.m.

$$M=\frac{Pl}{4} \quad (l=1 \text{ m}) \quad \therefore \quad P=4\times 1.9=7.6 \text{ t}$$

Thus, almost the same results are obtained.

Test piece (D) (Fig. 19)

Same as (A)

$$R.M.=.35\,T$$

$$T \text{ (double shear)}=.96\times 2\times 3{,}500=6.8 \text{ t}$$

$$R.M.=.35\times 6.8=2.4 \text{ t.m.}$$

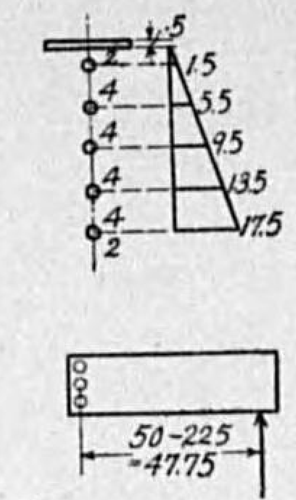


Fig. 16.
Test piece A.

$$R.M.=.4775^{m}\times\frac{P^t}{2}=2.4 \text{ t.m.}$$

$$\therefore \quad P=10 \text{ t}$$

Test load is 12.3 t

$\therefore$ Shear on the rivet $=3{,}500\times\frac{12.3}{10}=4{,}300$ kg/cm²

Test piece (B) Fig. 17.

Reinforced concrete beam.

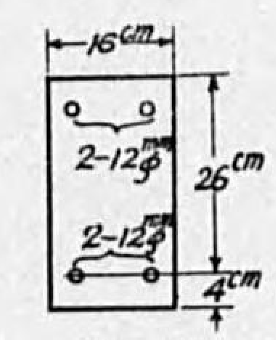


Fig. 17.
Test piece B.

$f_t=4{,}300$ kg/cm²

$f_c=150$ kg/cm²

$b=16$ cm

$d=26$ cm

$a_c=a_t=2.26$ cm²

$n_1=0.236$

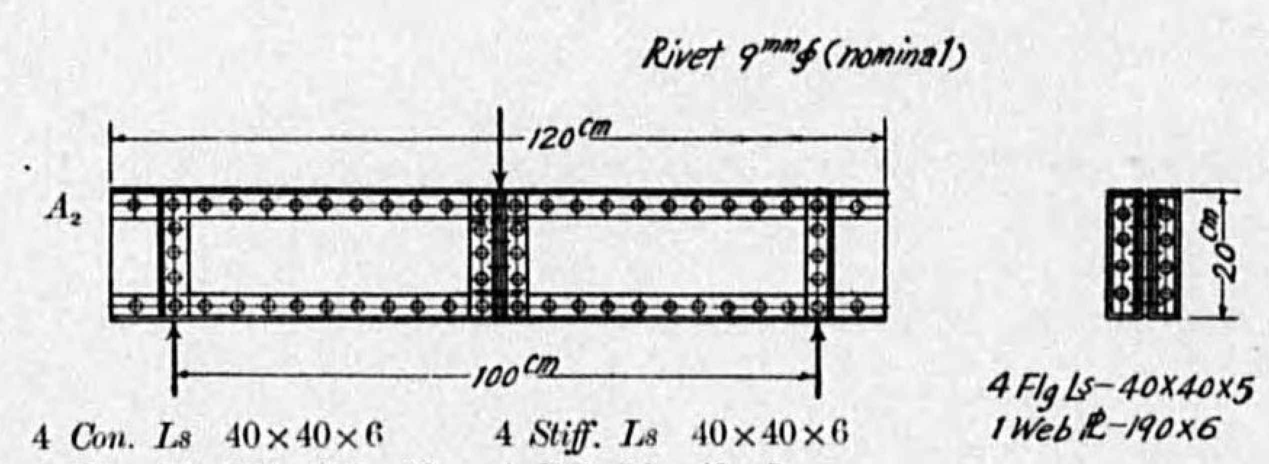
$$M_s=\frac{3-n_1}{3m_t}f_t bd^2=2.4 \text{ t.m.}$$

$$M_c=\frac{15(3-n_1)(1-n_1)}{3n_1m_t}f_c bd^2=4 \text{ t.m.}$$

$$B.M.=\frac{Pl}{4}$$

$$\therefore \quad P=\frac{2.4\times 4}{1}=9.6 \text{ t}$$

Actual load 10.7 t



4 Con. Ls 40×40×6 4 Stiff. Ls 40×40×6
1 Con. Pl 200×100×10 4 Fill. Pl 40×5

Fig. 18. (A) Detail of Test specimens

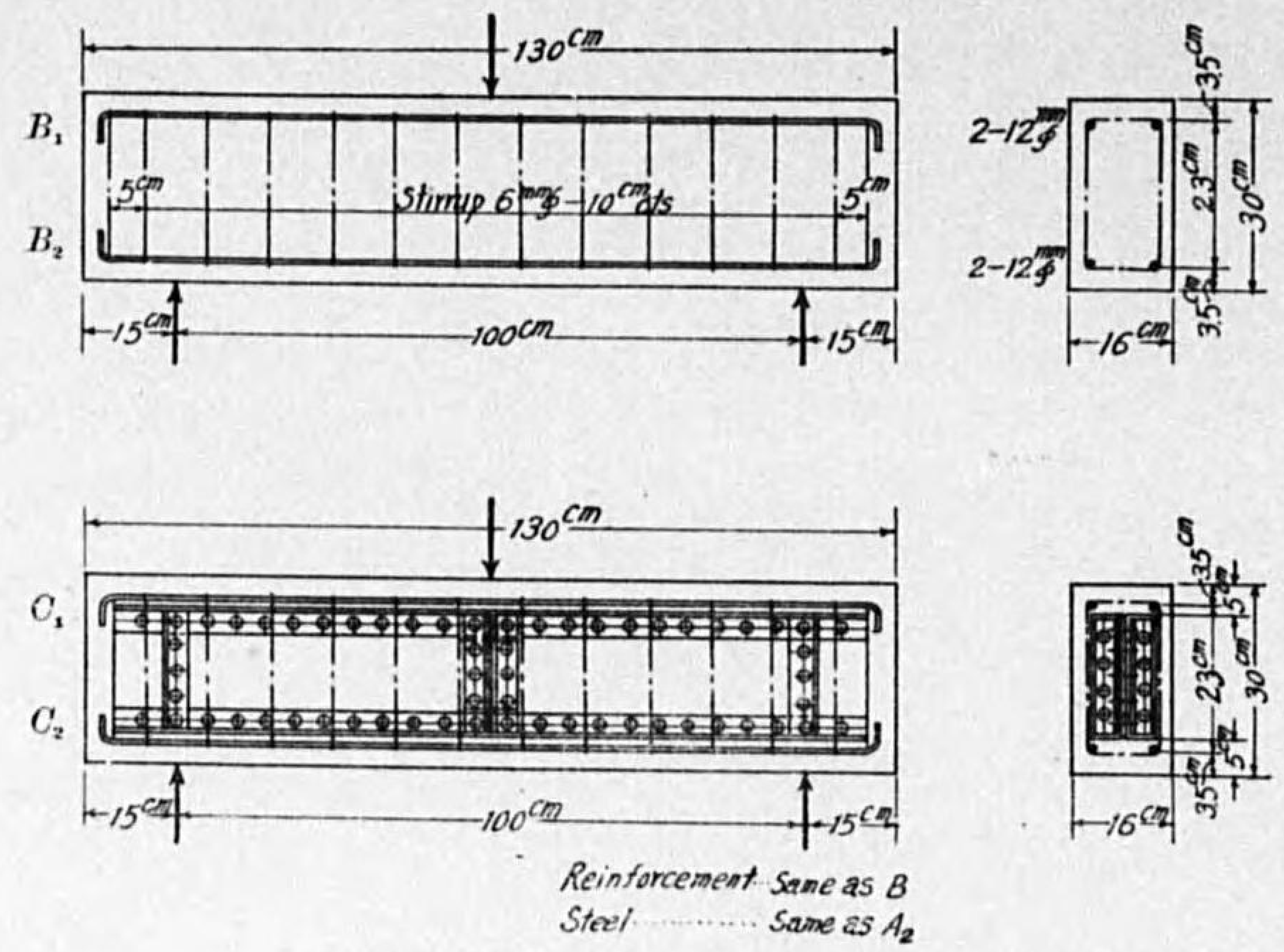


Fig. 18. (B) Detail of Test specimens

Test piece (C) and (E) were the same as $A+B$ and $D+B$ respectively.

The following tabulated comparisons show that the test and the calculated loads are nearly equal. (Table 4)

Table 4. Bending Resistance of Beam to Column Connections (test of 1930).

Specimen		Expected breaking load	First crack noticed at	Max. test load	$\frac{\text{Max. load}}{\text{expected load}}$
A	Steel only	7.72 t	——	7.95	1.02
B	Reinforced concrete only	9.6	7	10.7	1.12
C	$A+B$	7.72+9.6=17.32	9, 10	16.8 19.07	1.03
D	Steel only	10	——	12.3	1.23
E	$D+B$	10+9.6=19.6	9	21	1.07

Fig. 20 shows the stress strain diagram obtained from the test. The following are the results of the test summarized :—

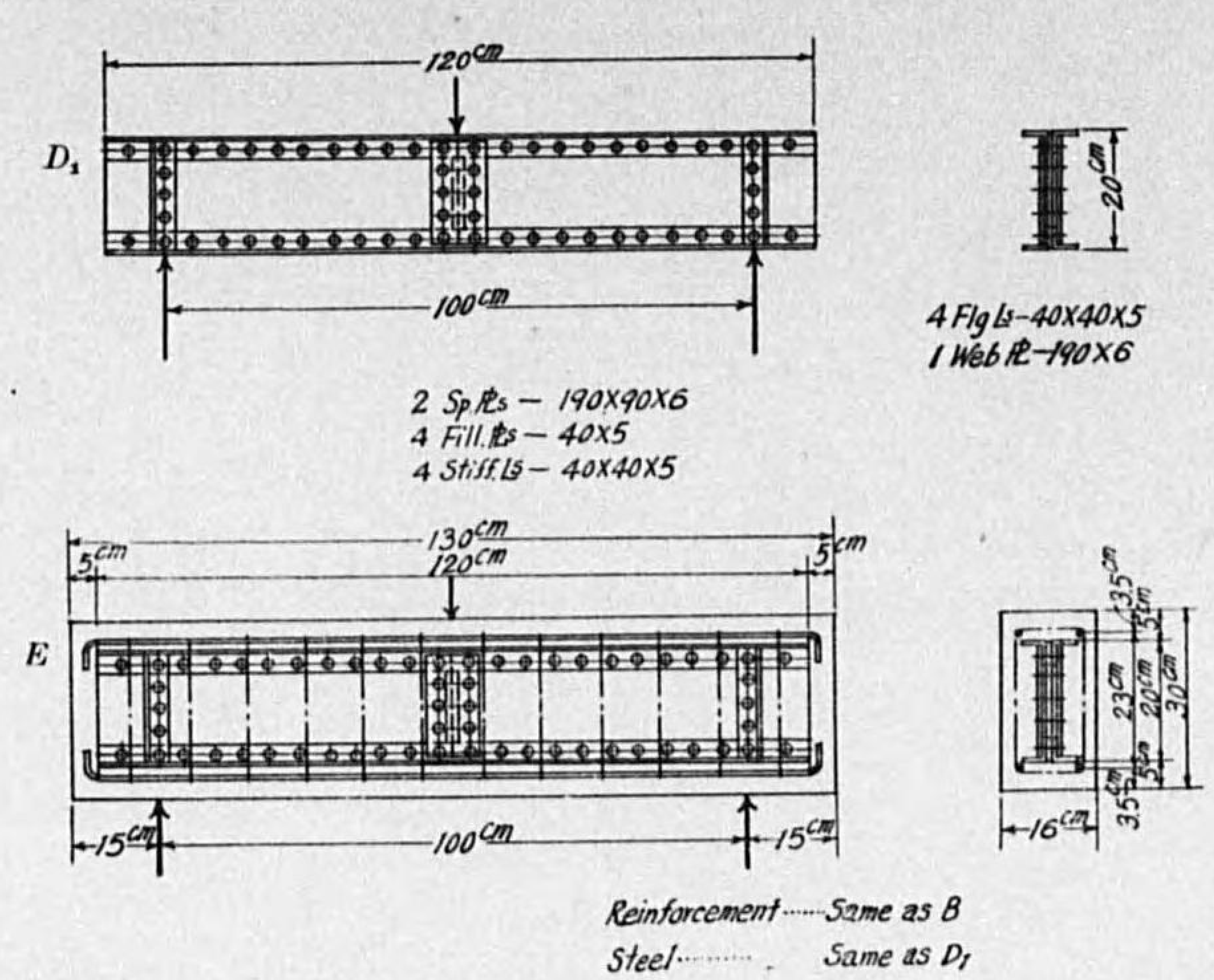


Fig. 19. Detail of Test specimens

(a) Steel connections with steel reinforcements (C.E.) give the strength approximately equal to the sum of the strength of the steel connections and the reinforced concrete.

(b) The calculated value divided by 4 (factor of safety) will give the safe working load. This can be verified from the deformation diagram (Fig. 20), showing that the strain at $\frac{P}{4}$ is very small.

(c) The stiffness of the connections : Fig. 20 shows the deformation of the L connection. (A) has a very large deformation while that of (D) connected by the plate, is rather small. This shows (D) is more rigid than (A).

Specimen (B) made simply of reinforced concrete is indeed stiff,

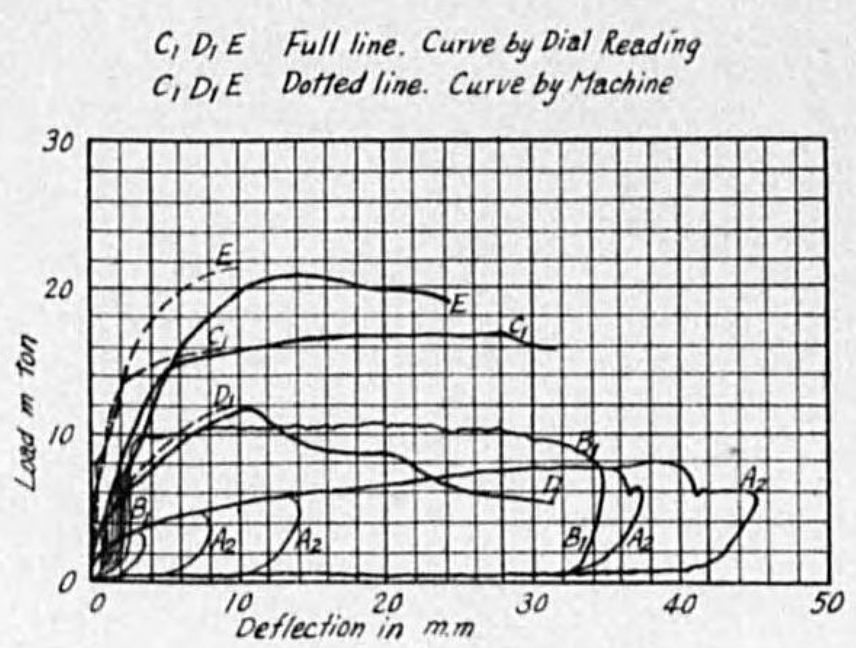


Fig. 20. Load deformation diagram of Joined Beams

yet it does not bear up the load at its maximum, whereas, that of steel and reinforced concrete combined shows that it possesses together the strength peculiarity of each.

Especially, when reinforced concrete is applied to a steel joint for obtaining rigidity then reinforced concrete works very effectively.

In the stress-strain diagram, the curves *C* and *E* show marked stiffness as compared with the curves *A* and *D*.

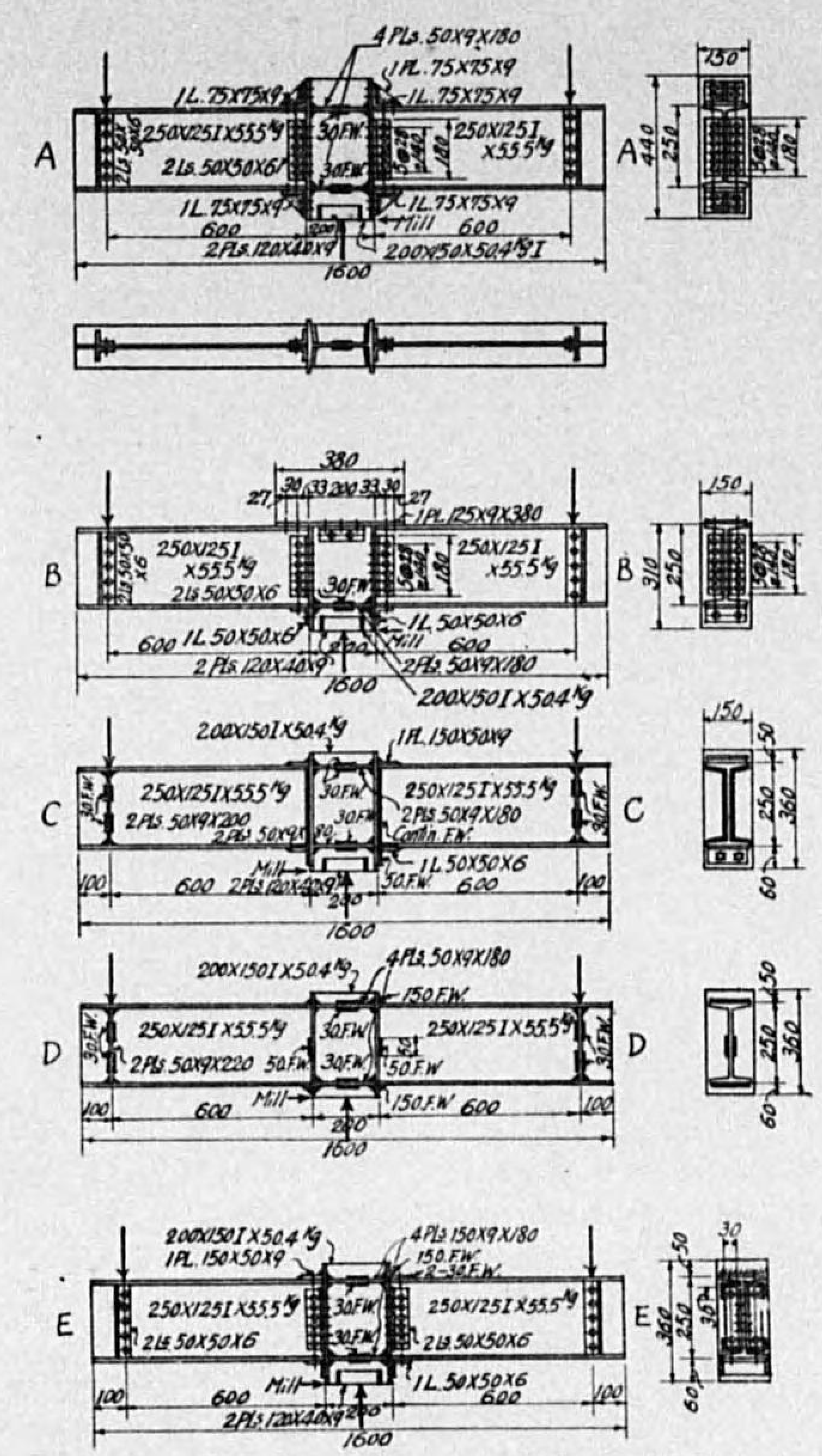
In these experiments, it is regrettable that the reinforced concrete beams failed through the slipping of the bars at the end supports, and in consequence, the expected full strength of reinforcement could not be developed. If the bars had been prevented from slipping, a far greater load could have been sustained.

After all, the test shows that bond plays a very important part in reinforced concrete structures.

(6) Beam to Column Connections (II).

(1931 Experiments, Laboratory of Waseda University)

In these experiments, joints with a ribbed top *L*, those with a through-plate on the tension-side, and those with electric welds, were tested for the purpose of finding their strength and rigidity. Test pieces are classified as *A. B. C. D. E.*, as shown in Fig. 21 (*A*), two pieces being prepared for each type. All specimens were tested as double cantilevers.



Note ; Rivet 8φ. Rivet hole 10φ. Welding 9x9mm fillet

1930 test by Dr. T. Naito & Mr. A. Tsuruta

Fig. 21 (A) Detail design of Structural Test pieces

Calculation of the Strength of Test Specimens.

Specimen *A*. The rivet dia. was 8 mm. and the rivet hole 10 mm. It was assumed that the rivet filled up the hole.

When the test piece bends, the lower part of it will be compressed, and at the same time the upper part will suffer tension, causing shearing stress on the rivet.

At the web the joint of the *L* suffers tension, so its modular ratio *e* becomes large. Here we assume that the ratio is 27 as in the previous case. The effect of web *L* being negligible, the equivalent area will be as small as

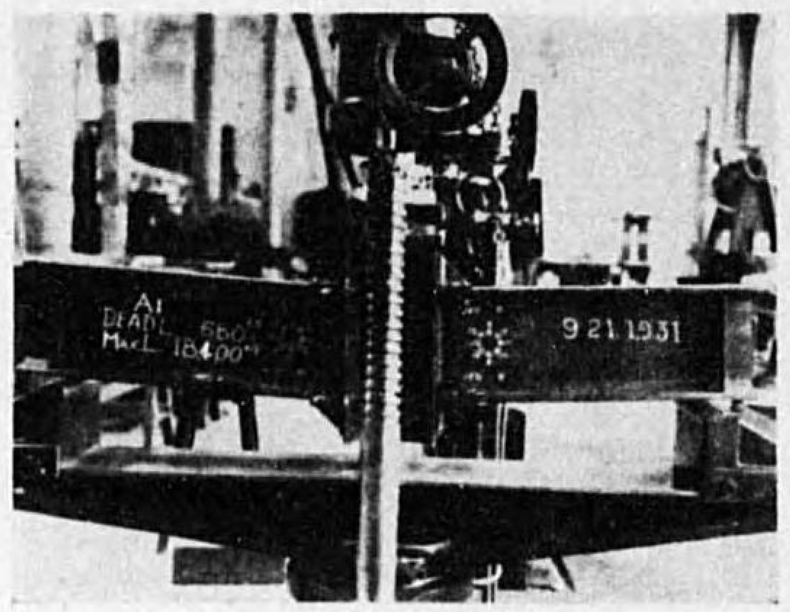


Fig. 21 (B) Testing method

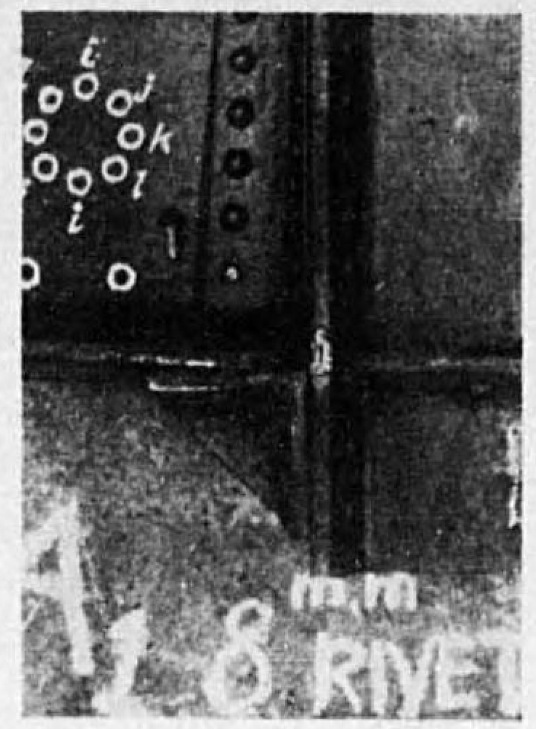


Fig. 21 (C) Test piece A_1

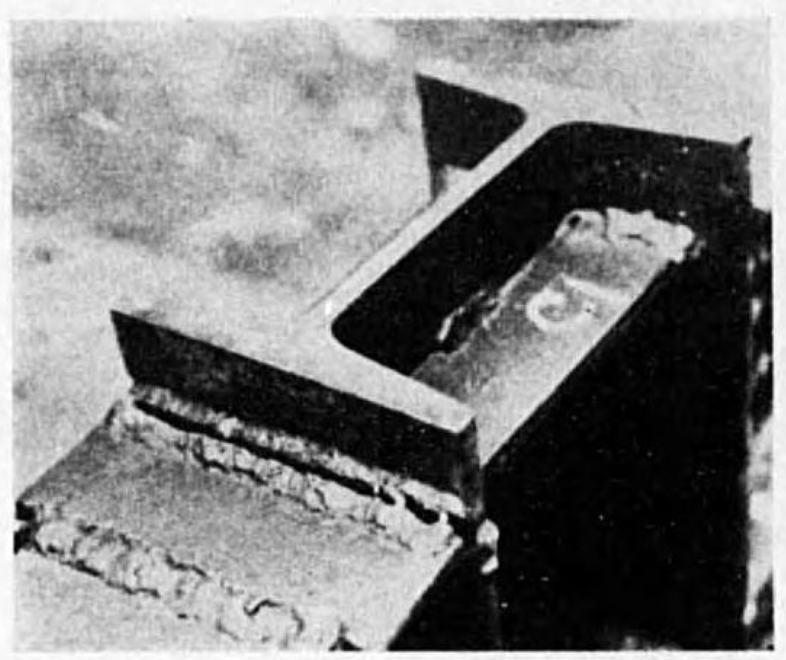


Fig. 21 (D) Test piece C_1

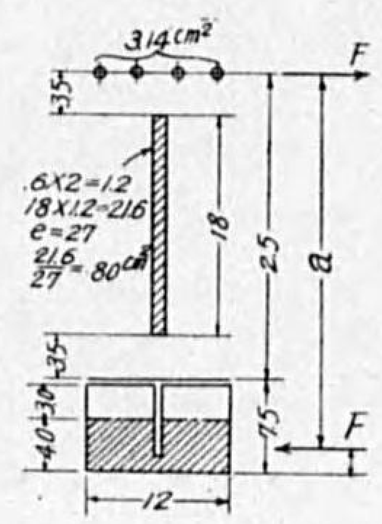


Fig. 22. Test piece

$\frac{18 \times 1.2}{27} = .80\ \text{cm}^2$. It can safely be said that only the compression on the angle surface and the shearing force on the tension side rivets will resist moment, as shown in Fig. 22. The neutral axis will, therefore, be located at a point 4 cm. from the edge of the L abutting to the column (Fig. 22).

$\therefore\ R.M. = F.a$ here F equals the shearing strength of 4 rivets.

$F = 3.14\ \text{cm}^2 \times 3{,}500\ \text{kg/cm}^2 = 11\ \text{t} \qquad a = 25 + 7.5 - 2.0 = 30.5$

$\therefore\ R.M. = 11 \times 30.5 = 3.36\ \text{t.m.}$

and $\quad B.M. = \frac{P}{2} \times .6 = .3P \qquad \therefore\ P = 11.2\ \text{t}$

The test loads were 18.4 t and 16.7 t, the average being 17.5 t

Specimen *B*. An ordinary L being placed on the compression side, the compression concentrates upon the bottom flange of the I beam. Therefore, making the arm length equal to 15 cm, and neglecting the effect of the web L, the following is obtained :—

$R.M. = F \times 25 = 3.14 \times 3{,}500 \times .25 = 2.77\ \text{t.m.}$

$M = .3P \qquad \therefore\ P = 9.2\ \text{t}$

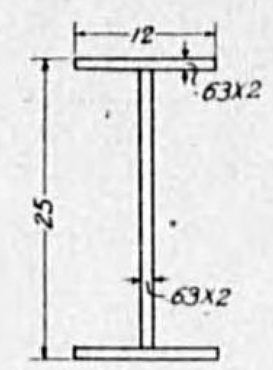


Fig. 23.

Actually, the test loads were 12.5 t and 14 t, the average being 13.25 t.

Specimen *C* As the joint was all welded, this was supposed to be an I section as shown in Fig. 23.

$$I = (12 \times 2 \times 1.26) \times 12.5^2 + \frac{1.26 \times 22^3}{12} = 4{,}750 + 1{,}110$$

$$= 5{,}860 \qquad S = \frac{5{,}860}{12.5} = 470$$

Assuming the strength of weld to be $3{,}000\ \text{kg/cm}^2$,

$R.M. = 470 \times 3{,}000 = 14\ \text{t}$

$M = 0.3P \qquad \therefore\ P = 46.6\ \text{t}$

Actually, the test loads were 39.55 t, 45.15 t, the average being 42.33 t.

Specimen *D*. Only the top and bottom flanges were welded.

$13 \times 0.63 \times 3{,}000 = 24.5\ \text{t}$

$R.M. = 24.5 \times .25 = 6.15\ \text{t.m.}$

$M = .3P \qquad \therefore\ P = 20.5\ \text{t}$

The test loads were 23.25 t and 27.7 t, the average being 25.5 t.

Specimen *E*. The welded part was about 7 cm. with two rivets.

Then, $\quad F = 7 \times .63 \times 3{,}000 + \frac{3.14}{2} \times 3{,}500 = 13 + 5.5 = 18.5$

$\therefore\ R.M. = 18.5 \times .25 = 4.6\ \text{t.m.}$

$P = 15.3\ \text{t}$

The test loads were 16.45 t and 21.65 t, the average being 19.5 t. This result shows that the sum of the strength of the rivets and welds gives the strength of the specimen. The result is shown in Table 5.

Table 5. Bending Resistance of Beam to Column Connections.
(Test of 1931)

Specimen	Expected breaking load	Test load	Test load / expected load
A_1	11.2 t	18.4 t	1.55
A_2		16.7	
B_1	9.2	12.5	1.43
B_2		14	
C_1	46.6	39.55	.9
C_2		45.15	
D_1	20.5	23.25	1.24
D_2		27.7	
E_1	15.3	16.45	1.25
E_2		21.65	

Except for specimen C, all the others sustained 20–50% larger loads than the calculated and it seems that the strength of rivet joints is usually larger than the calculated values.

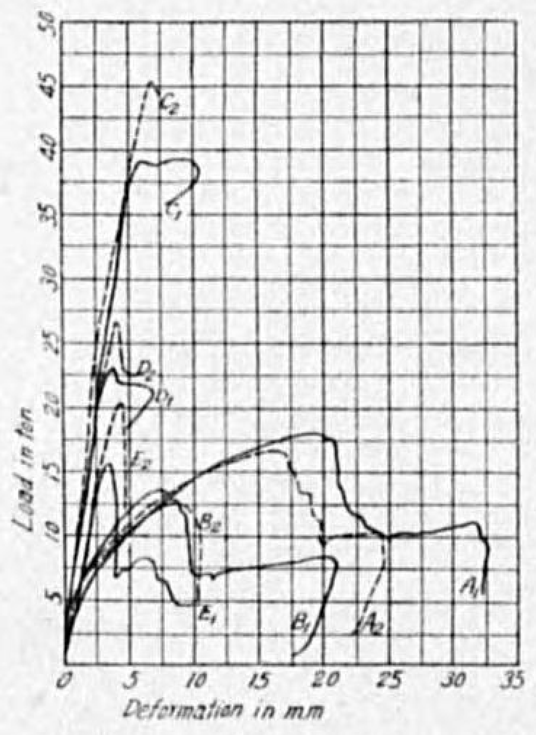


Fig. 24.

Concerning the Rigidity of Connections.

Fig. 24 which shows the stress strain diagrams of these five specimens ($A. B. C. D. E.$), indicates that the welded ones proved striking stiff. The riveted pieces suffer larger deformation, and in point of stiffness, riveted ones bear no comparison with the welded ones. The welded pieces are indeed very stiff, and the failure of the joints takes place abruptly. This is a phenomenon similar to that appearing in the tension test, but if the safety factor 4 be given to a planned load, it will be amply safe.

Concerning the Design Calculation of Connections.

The foregoing tests show that the connections of this kind may be calculated principally by regarding the flange stresses as a couple and regarding the end reaction as resisted by the rivets of the connecting web angles of the joint.

This mode of analysis will afford safe and easier calculation. When weld is added, the joint will be so much strengthened.

(7) Beam to Column Connections (III).

(Beam with Bracket End).

(1930 Experiments, Laboratory of Waseda University).

In the quake-resisting design of high buildings, great importance ought to be attached to the connections of beam to column, because the connections are subject to large bending moment produced by lateral force. In the case of the design of Tokyo Sumitomo Building, the author, with an expert of the company, contrived a great number of different details and tested them with half sized specimens.

Figs. 25–30 show the details of the test specimens :—

Specimen A.

The beam was connected with a plate through the column, and it had a ribbed top-angle.

Specimen B.

This was the same as A, except that it had a plate welded at the top instead of a ribbed top-angle.

Specimen C.

The beam had a bracket at the bottom and a ribbed angle and weld at the top.

Specimen D.

This was the same as C, but it had no weld at the top.

All was designed approximately with the same strength.

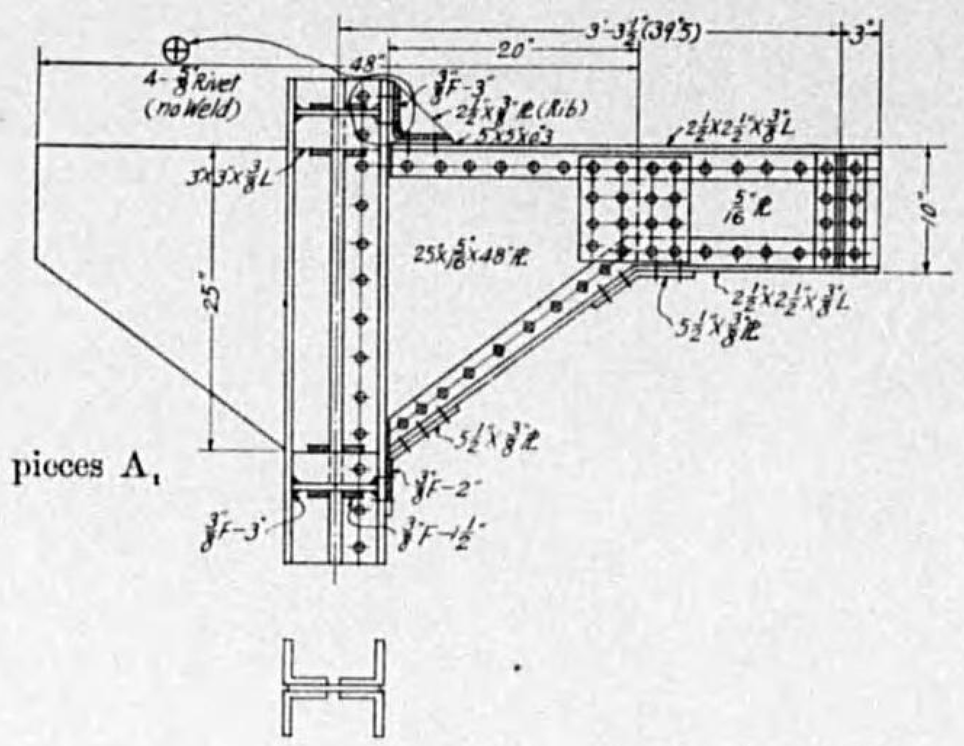


Fig. 25. Test pieces A_1

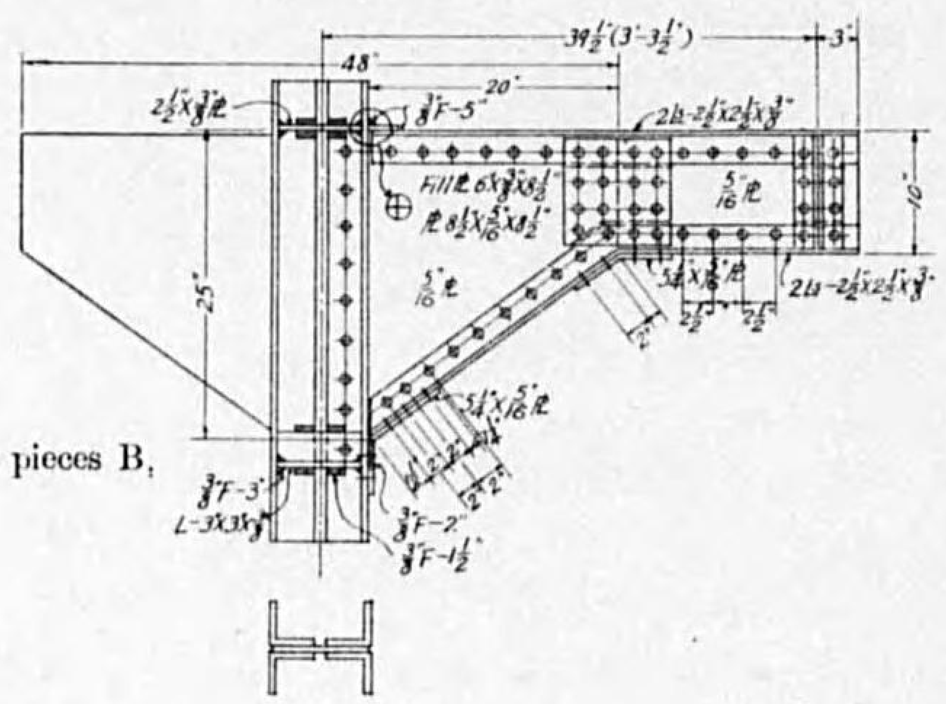


Fig. 26. Test pieces B_1

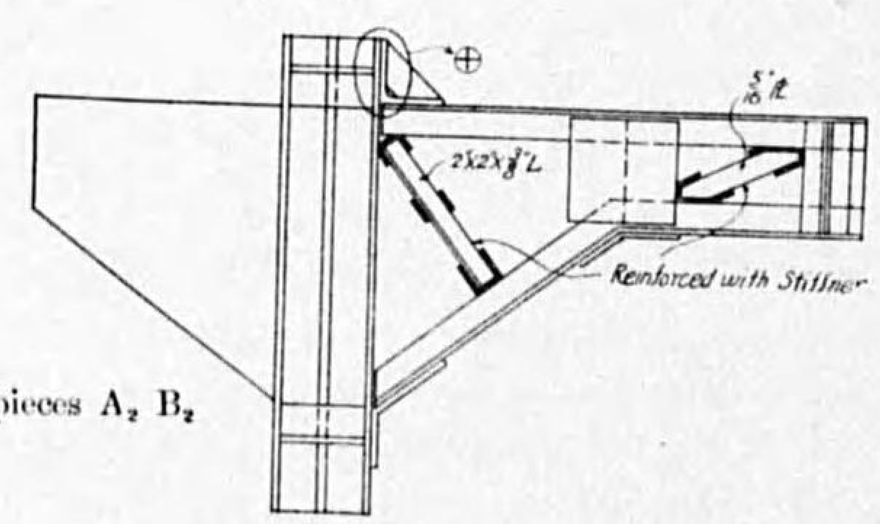


Fig. 27. Test pieces A_2 B_2

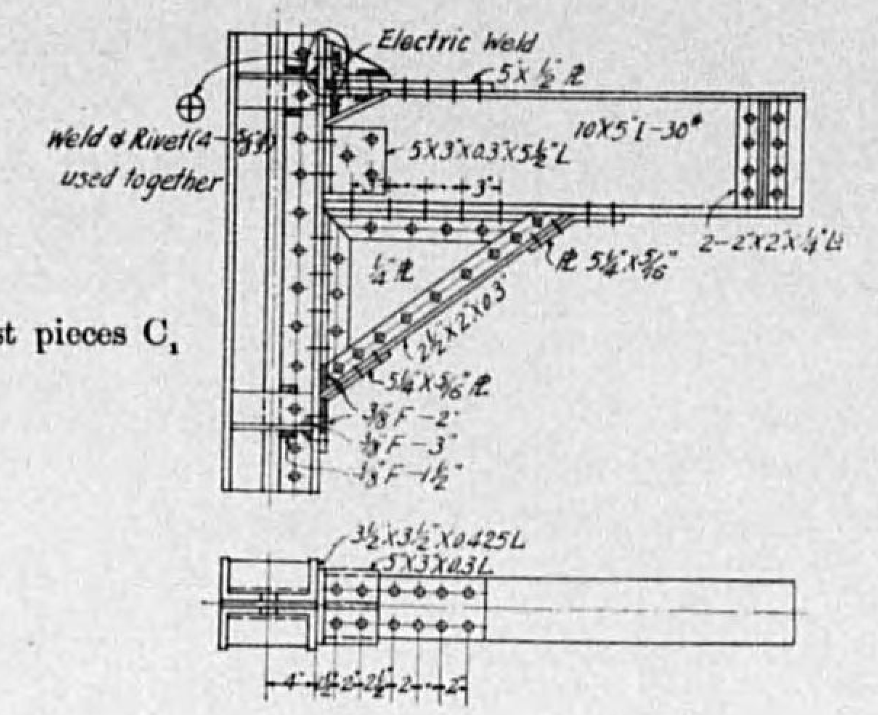


Fig. 28. Test pieces C_1

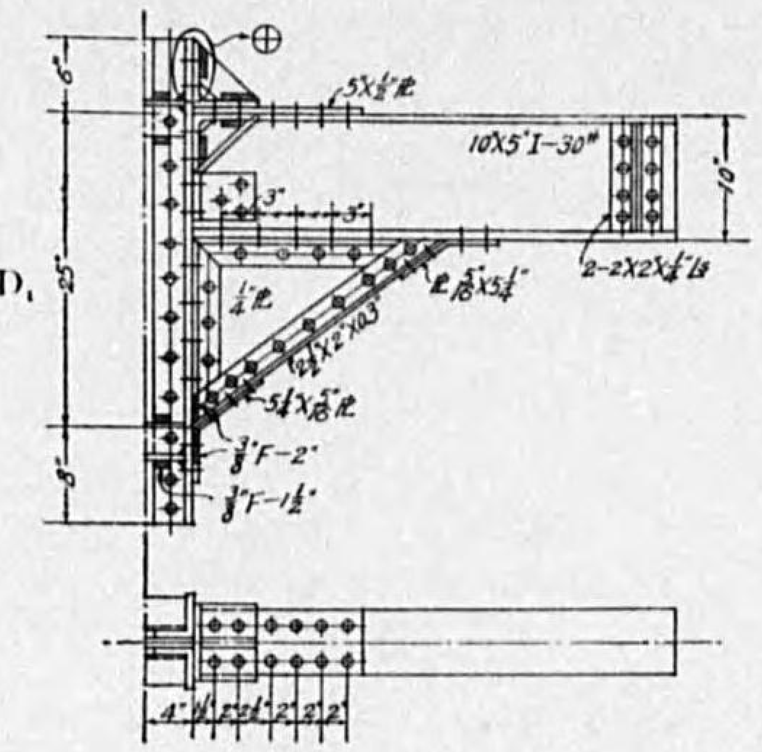


Fig. 29. Test pieces D_1

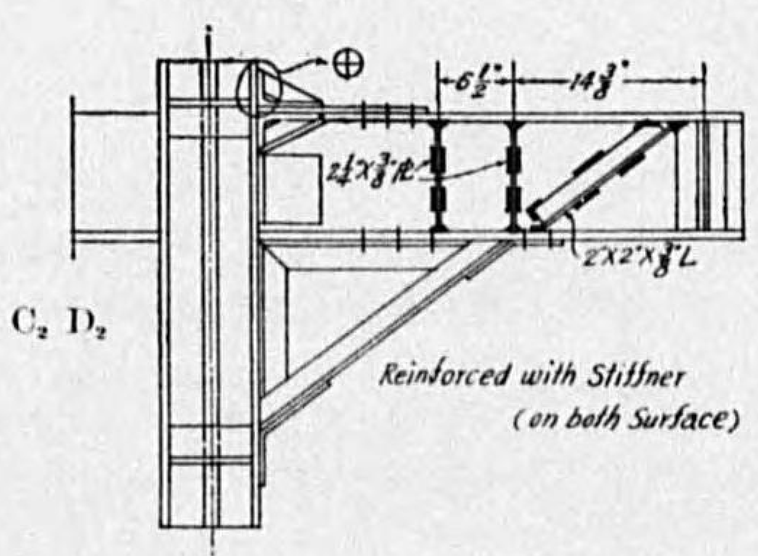


Fig. 30. Test pieces C_2 D_2

Calculation of Joint Strength and Test Load.

Specimen *A*.

The top angle having a rib to impart sufficient strength, the four shearing rivets are assumed to work. Therefore, the section which resists the moment may be assumed as shown in Fig. 31 (rivet hole being 11/16″).

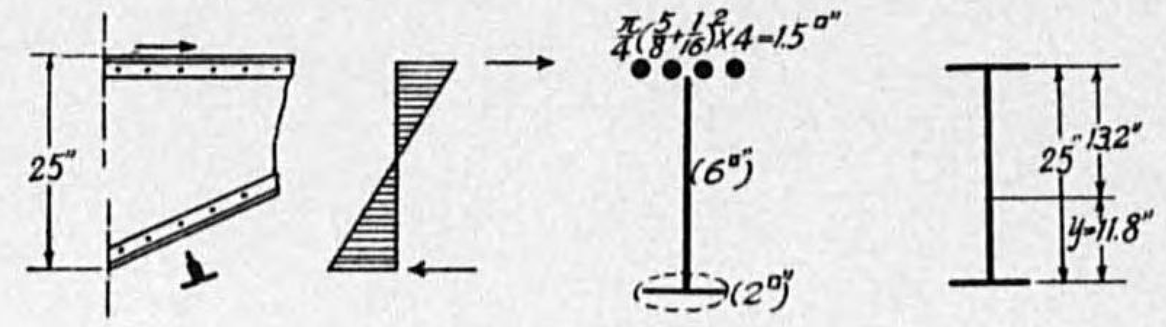


Fig. 31. Specimen A.

$$I=863. \qquad M=f\frac{I}{y}=12{,}000\times\frac{863}{13.2}=790{,}000 \text{ in. lbs.}$$

$$B.M.=35.5''\times\frac{P}{2}=790{,}000 \qquad \therefore\ P=20\text{ t}$$

Assuming the safety factor as 4, the breaking load will be 80 t. The actual maximum test loads were 79.5 t 78.5 t, the average being 79 t, and failed by buckling of the web plate.

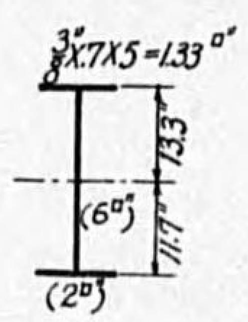


Fig. 32. Specimen B.

Specimen B.

Taking the weld area at the top into consideration and calculating similarly as for specimen A (Fig. 32),

$$I=824 \qquad S=62.5$$

Assuming the strength of the weld in tension as 11,000 lbs./sq. in.,

$$R.M.=680{,}000 \text{ in. lbs.}$$

$$\therefore\ P=17.7\text{ t.}$$

If the safety factor be 4, then the breaking load will be 70 t. The test loads were 69.5 t and 73.5 t, the average being 71.5 t, and the joint failed at the weld.

Specimen *C*.

Eight rivets at the top flange were stressed by shear, the area being 3 sq. in.

Assuming the breaking shearing strength of the rivet as 45,000 lbs/sq. in., and taking the arm length as 25″ (neglecting the web),

$$R.M.=(45{,}000\times 3)\times 25''=3{,}400{,}000$$

$$\therefore\ P=\frac{2\times 3{,}400{,}000}{35.5\times 2{,}240}=83\text{ t.}$$

Specimens C_1 and C_2 were subjected to test loads of 80 t and 86.5 t. The former failed by buckling, while the latter did not fail as the stiffner was added before testing.

Specimen *D*.

This is the same as *C*. $P=83$ t.

Test loads were 73 t and 74.5 t, and they buckled, but did not fail.

The following are the results tabulated :—

Table 6. Bending Resistance of Beam Connections with Bracket Ends.

	Expected breaking load	Test load	Manner of failure
A_1	80 t	79.5	Buckled (not broken)
A_2		78.5	„
B_1	70 t	69.5	Weld part broken
B_2		73.5	„
C_1	80 t	80	Buckled not broken
C_2		86.5	„
D_1	83 t	73	Buckled not broken
D_2		74.5	„

Test specimen *B* was welded only, so that it proved to be the weakest. Notwithstanding this fact, it bore up a heavier load than the calculated load.

The others failed by buckling, and could not develop full strength. If the web is properly protected against buckling, the connections will

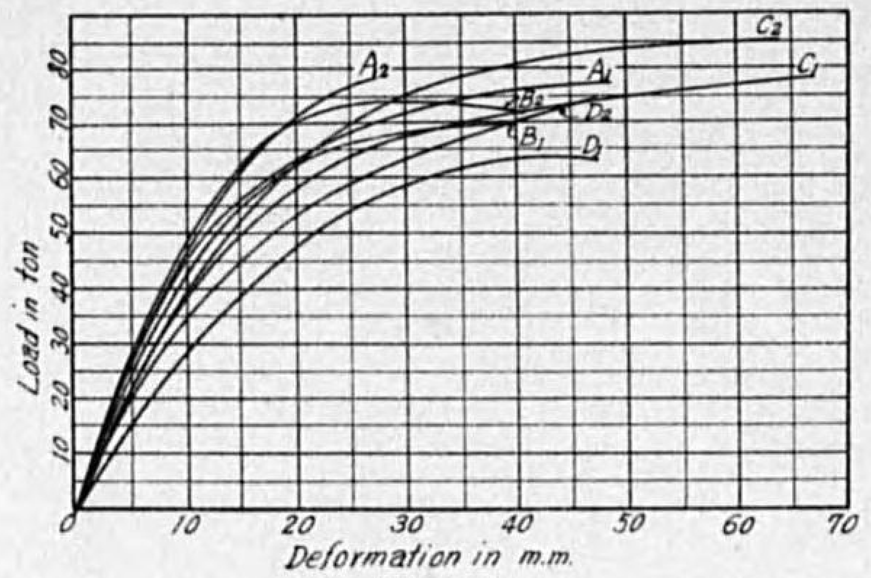


Fig. 33.

show further resistance. In general, the connections will withstand a calculated load. From the tests, it can safely be said that the calculation of strength of joints as mentioned above is one of the most proper methods.

The connection of beam to column (when it has no through plate) may be calculated, by the above method as it sustains the moment principally with the top and bottom flange stresses.

Concerning the Rigidity of Joints.

Judging from the curves in Fig. 33, it is noticed that specimens *A* and *B* are the most rigid, undergoing the least deformation : next comes *C*, and *D* shows comparatively larger deformation. The reason for the above is this : both in *A* and *B*, a through plate forms the web, and the top ribbed angle or weld adds strength, therefore, the joint undergoes very little elongation. So the joint is extremely rigid. Now in *C* and *D*, the top rivets are sheared at two places and the joint plate is elongated. Naturally the joint undergoes large deformation.

In conclusion, the connections can safely be designed and calculated as described above. The results of the actual tests are on the safe side, and check with the calculation satisfactorily, thereby proving its reliability.

For producing rigid joints, ribbed top angle must be used, restricting the deformation of the connection angle to a minimum. The application of welds to connections or the combined use of welds and rivets will achieve good effects.

Furthermore, a steel joint embedded in reinforced concrete will have

a strength equivalent to the sum of the strength of the steel joint and reinforced concrete structure, considered separately. It is quite effective and favorably adapted to practical construction.

The foregoing statements are verified by the author's experiments.

Tests on the Strength of Weld Joints Strengthened by Reinforced Concrete.

BY

Inst. **Akira Tsuruta.** *Kōgakushi.*

(1) Introduction.

The purpose of this investigation is to study experimentally the degree to which the all-welded connection of steel beam and column is strengthened by reinforced concrete encasement.

Few investigations have been made in this direction generally, which may be attributed to the fact that the so-called steel concrete construction is a distinct development following the disasterous earthquake of 1923 in Japan. Quite logically, the study of this type of construction has been pursued with vigor in Japan, however.

I acknowledge deep gratitude to Dr. T. Naito for his kind suggestions during the present investigation and to the Japan Steel Products Co., Ltd. for the contribution of steel test specimens. I also thank several students, who worked in cooperation with the author, for their assistance.

(2) Strength of Welds.

The strength of welds is subject to variation with the quality of the materials used and with the skill and technique of the welder. A qualification test was performed under conditions which were as nearly identical as possible with the actual conditions of the main experiment in order to determine the weld strength in the present investigation.

The qualification test was made with four types of joints (see Fig. 1.) which were conveniently chosen by referring to both "Standard Tests for Welds"[1] and "Vorschriften fur die Ausfuhrung geschweisster Stahlhochbauten."[2]

[1] American Bureau of Welding: Standard Tests for Welds (revised, 1931).
[2] Der Minister für Volkswohlfahrt: Vorschriften für die Ausfuhrung geschweisster Stahlhochbauten. (Juli, 1930)

(a) Tension Tests of Butt Welds (see Fig. 1 (a)).

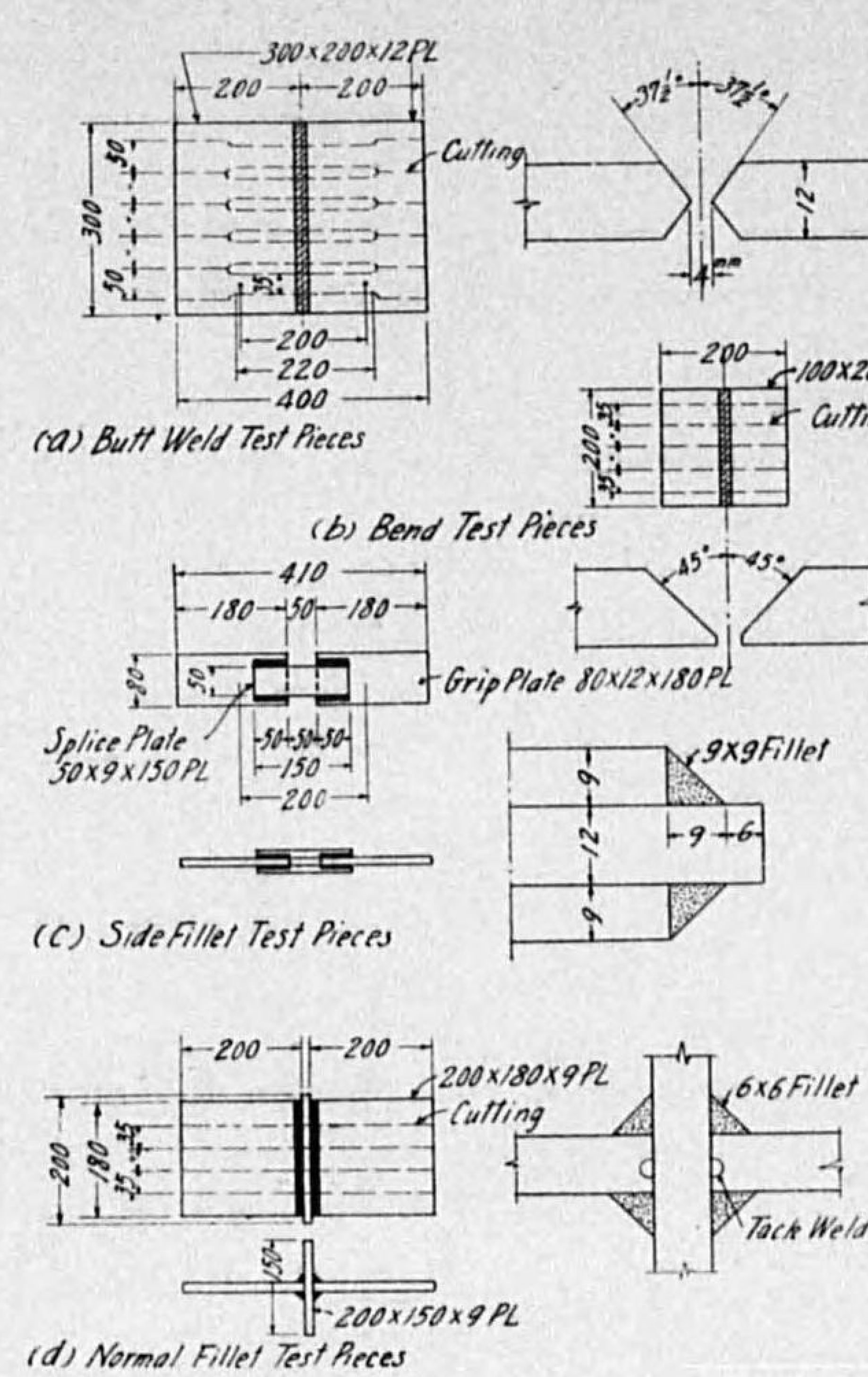


Fig. 1. Qualification Test Pieces.

The tests were made with two sheets of an open double-vee butt joint welded in two different positions, that is welded in the flat and vertical positions. The reinforcement was removed from these two sheets, and each of them cut into seven coupons. The middle five coupons thus obtained were tested. The average value of the strength obtained is shown below :—

Flat Welding
35.64 kg/mm²
(50,680 lbs/in²)
Vertical Welding
29.12 kg/mm²
(41,410 lbs/in²)

(b) Bend Tests of Butt Welds.

The test coupons used for this type of tests were made in the manner similar to the previous coupons mentioned, except that these were of an open single-vee butt joint welded in the flat position (see Fig. 1 (b)).

The test coupons were tested but no clear results were obtained owing to the fact that the method employed in these experiments seemed inappropriate.

(c) Tests of Side Fillet Welds.

Fig. 1 (c) shows the test specimens. These were welded in the flat position, the size of the fillets being 9 mm × 9 mm × 50 mm.

The result showed that the shearing strength through the throat of the fillet was 29.83 kg/mm^2 (42,410 lbs/in^2) which is the average value of the two specimens.

(d) Tests of Normal Fillet Welds.

The test was performed to determine the strength of the fillet welds subjected to a force in the normal direction.

The test specimen consisted of three plates which formed a cross as shown in Fig. 1 (d). When the welding was finished in the flat position, the specimen was cut into five pieces of which inner three were tested.

The test showed that the average strength of the fillet at its throat was 30.85 kg/mm^2 (43,870 lbs/in^2).

Perhaps there are many causes to account for the fact that the strength of the weld used in the present investigation was about 15% lower than the strength of the welds generally used in Japan.

(3) Strength of Concrete and Reinforcing Bars.

Assuming the expected strength of four week concrete to be about 150 kg/cm^2, the fineness modulus of the aggregates was measured to obtain the proper proportion of the concrete mixture. The proportion of the concrete was determined to be in the ratio of 1.2 : 2 : 4. The water-cement ratio is 65% by weight and the slump of 20 cm (8 in) on the average. The concrete thus made was tested simultaneously with the principal experiment when the specimens were three months old. The average value in compression for the concrete was found to be 198 kg/cm^2 (2,816 lbs/in^2).

The determination of the strength of the reinforcing round bars was accomplished by testing four specimens. The average value of the tensile strength was 48.1 kg/cm^2 and the elongation 28.4%.

(4) Design of Specimens to Test the Welded Joints Strengthened by Reinforced Concrete.

The author has adopted the following notation to be applied to the specimens: *R*, *D*, and *DR*. Specimen *R* stands for reinforced concrete specimen; specimen *D* for the steel specimen joined by welding; specimen *DR* for the combined specimen strengthened by placing *D* in *R*. Two specimens of each kind were prepared for the purpose of obtaining average results.

The following are the assumed stiengths in the design of the specimens:—

Strength of Weld (9 mm × 9 mm fillet, throat 6.36 mm)

Tensile strength 3,000 kg/cm^2 (42,670 lbs/in^2)

Shearing strength 2,850 kg/cm^2 (40,530 lbs/in^2)

Strength of Concrete.

Compressive strength 150 kg/cm^2 (2,133 lbs/in^2)

Strength of Reinforcing Bar.

Tensile strength 4,800 kg/cm^2 (68,270 lbs/in^2)

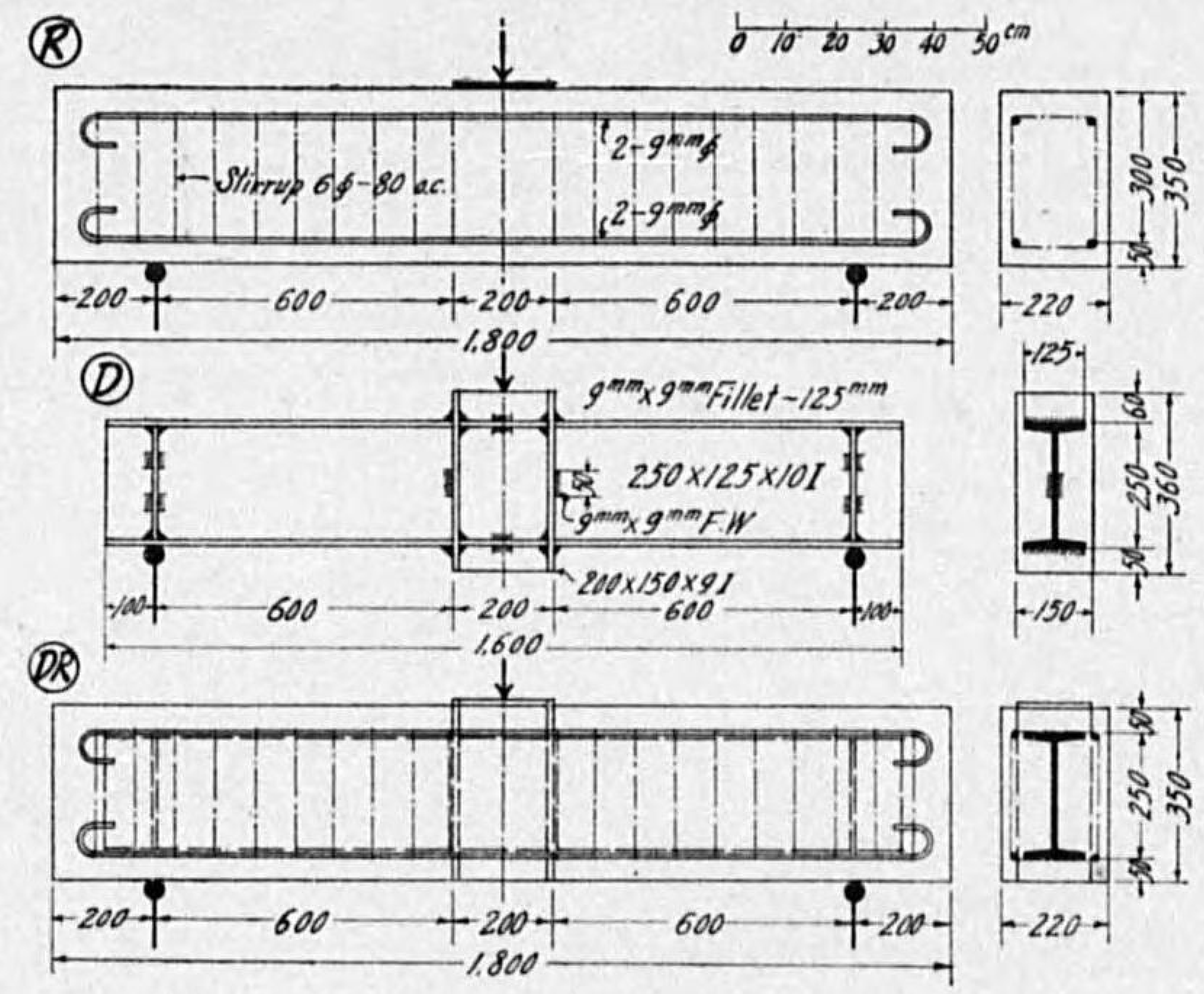


Fig. 2. Specimens for principal testing.

Fig. 2 shows the specimens which were designed with due consideration for the capacity of the testing machine in the laboratory of Waseda University.

The testing method in the photograph was adopted with the load at the center and simply supported at the ends. Neglecting the self weight of the specimens, the expected breaking load was calculated as follows :—

Specimen *R* : The reinforced concrete beam formula was adopted, and the specimens were designed to be broken by the failure of the reinforcing bars. Accordingly, the resisting moment of the bars $(R.M_r.)=1{,}750$ kg. m. (12,660 ft. lbs.) and the load $(P)=1{,}800$ kgs. (12,790 lbs.)

Specimen *D* : Assuming that the fillet weld of the top and bottom parts of the flange are capable of taking bending moment, and also assuming that the welds of the web are able to bear shearing, the resisting moment, $R.M.=5{,}950$ kg. m. (43,000 ft. lbs) and the load $P=19{,}800$ kg. (43,660 lbs.)

Specimen *DR* : Assuming that the summation of the strength of *D* and *R* separately is the strength of specimen *DR*, the ultimate load will be, $P=5.8+19.8=25{,}600$ kgs. (56,450 lbs.)

(5) Experiment.

The following table shows the comparative results of the experiment. According to these results, addition of the experimental loads on

Table 1. Results of Experiment.

Specimens		Allowable load in kg. (lbs.)	Experimented Breaking load in kg. (lbs.)	$\frac{\text{Breaking } L}{\text{Allowable } L}$	Fracture
Mark	Remarks				
R_1	Reinforced concrete	1,400	7,040 (15,520)	5.03	Slip of bars.
R_2		(3,087)	7,050 (15,550)		
D_1	Steel, joined by welding	5,300	23,250 (51,300)	4.81	Broken weld.
D_2		(11,690)	27,700 (61,100)		
DR_1	Combined, *taking *D* in *R*	6,700	37,300 (82,300)	5.48	Broken weld & slip of bars.
DR_2		(14,777)	36,150 (79,700)		

Note : Allowable load of *DR* is assumed as the sum of those of *R* and *D*.

specimens *R* and *D* gives, 7,015 kg+25,475 kg=32,2520 kg. (71,700 lbs.) However, it is to be noted that the experimental load on *DR* is 36,725 kg (80,980 lbs), and the ratio of the two values is

$$\frac{36{,}725}{32{,}520} \fallingdotseq 1.13$$

In short, the specimen *DR* is stronger by about 13% than the separate addition of the specimens *R* and *D*.

The specimens failed in the following manner :—

R : The slip of the reinforcing bars.

D : Failure of the weld.

DR : Failure of the welded part first, then the slip of the reinforcing bars.

Fig. 3 shows the relation between the load and deflection of the three specimens. It is seen that *DR* failed soon after it reached the maximum load (36,725 kg.) although it stood about 7,000 kg. load, being held by reinforced concrete.

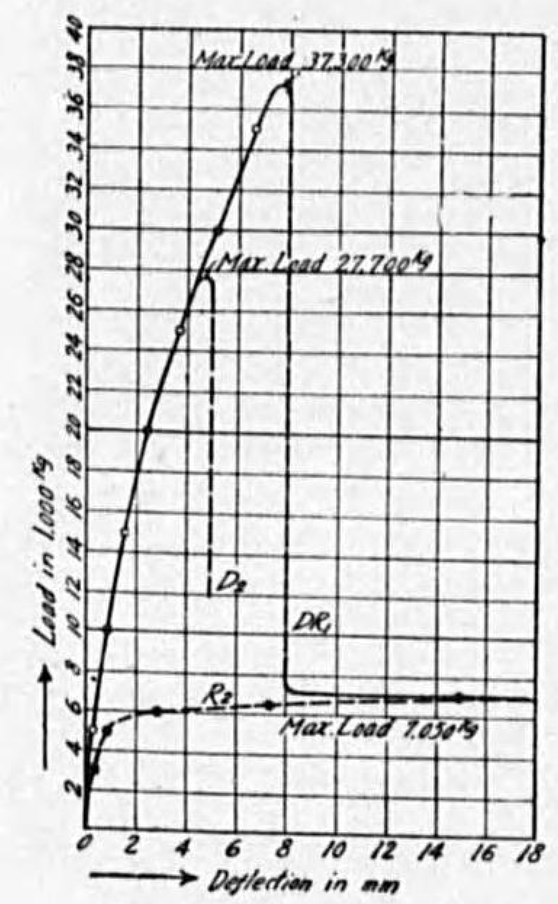


Fig. 3. Load-deflection Curves.

(6) Conclusions.

The following is a brief summary of the results of the experiments :

(a) The strength of the welded joint encased in reinforced concrete was greater than the sum of the strengths of the welded joint and reinforced concrete beam, considered separately, by 13%.

(b) The weld of the specimen *DR* was broken and yet reinforced concrete showed some resistence.

(c) The load-deflection curves for *D* and *DR* were much the same, showing almost straight lines before they were broken.

From the results mentioned above, the strengthening action of reinforced concrete on the weld joint was fairly effective. In the light of the present experiments, it will be safe in practical calculation to adopt the method of summing up the strength of the component parts which make up the member. However, the above may not be true in all cases when conditions vary greatly from these under which the present investigation was conducted.

The answer to the following items must be found by further studies :

(a) The strength of the weld joint covered by plain concrete.

(b) The variation in the strength obtainable when the proportion of the weld and reinforced concrete is changed.

(c) The phenomena taking place at the elastic limit.

(d) The investigations as to angular variation of the joint and other problems.

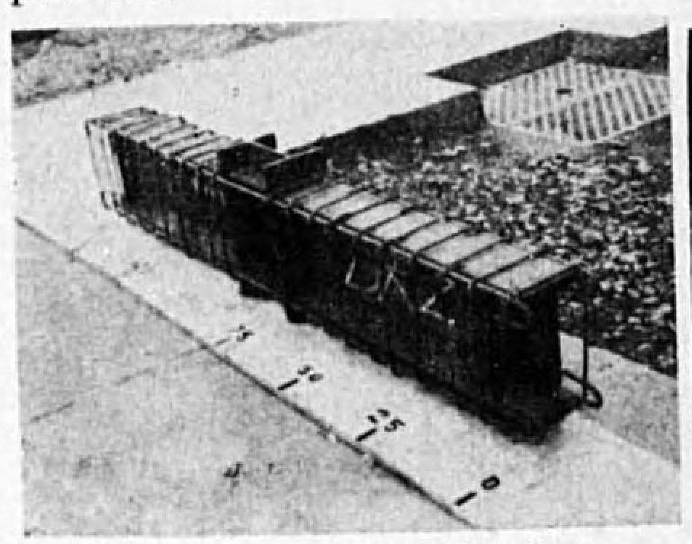


Fig. 4. Specimen *DR* before concreting.

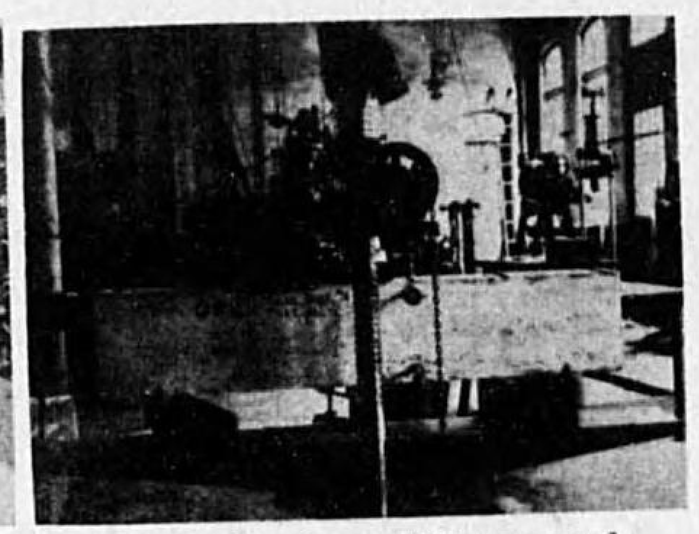


Fig. 5. Specimen *DR* amounted the testing machine.

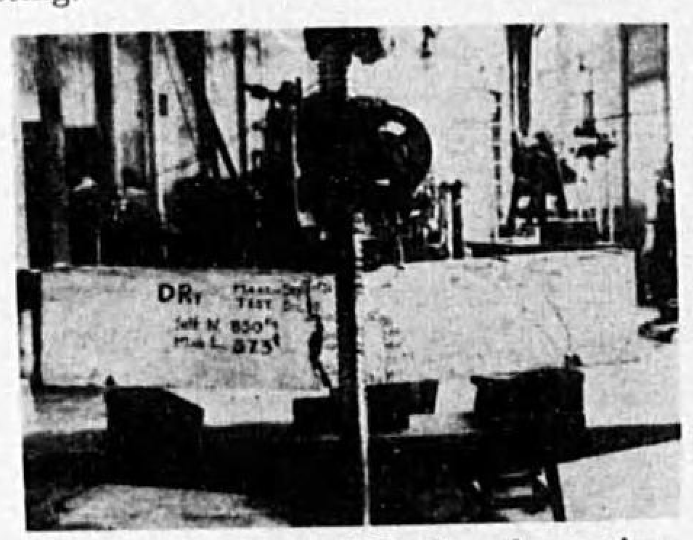


Fig. 6. Specimen *DR* after the testing.

On the Properties of the Metal Deposited in Arc Welding with Domestic Electrodes

BY

Inst. **Akira Tsuruta.** *Kōgakushi.*

The author presents in this paper some observations from the study of the metal deposited in arc welding with domestic electrodes which are commonly used in building construction in Japan. The items studied were the measurement of physical properties and the possible effects of annealing on the physical properties. Ordinary structural steel was tested in parallel to furnish comparative values with the metal deposited by electrodes.

The electrodes used in the test were coated electrodes, and designated *A*, *B*, and *C*, which were furnished by three representative manufacturing companies. The test conducted were :—tension test, impact test in which Charpy impact machine was used, and repeated impact test in which Mohr repeated impact machine was used.

The test pieces were obtained solely from the metal which was deposited in many layers on a steel plate by shaving and milling. (See Fig. 1) The test pieces of structural steel were obtained from the common round bars milled to size and dimension to conform with the above specimens.

Tables 1 and 2 show the test results. The following are the summary of the conclusions :

1. The tensile strength of the deposited metal is about 47 kg/mm² without annealing and about 43 kg/mm² with annealing. The yield point is about 60% of the ultimate strength, the modulus of elasticity being about 20,000 kg/mm². The elongation is from 10 to 15% in 100 mm. gage. The properties are similar to those of mild steel with the exception of the elongation which is less.

Table 1. Results of Tension Test.

Specimen	Mark	Method	Description of specimens: Diameter (mm)	Description of specimens: Area (mm²)	Stress at yield point (kg/mm²)	Maximum load (kg)	Load at failure (kg)	Stress at max. load (kg/mm²)	Elongation 100 mm gage (%)	Reduction in area (%)	Modulus of elasticity (kg/mm²)
A	1	without	10.15	80.4	33.47	3715	3250	46.2	8.95	22	20,500
	2	annealing	10.25	82.8	31.52	3795	3330	45.8	10.90	21	20,100
	11	annealed	10.20	82.0	24.39	3490	2720	42.6	12.20	24	20,300
	12		10.20	82.0	23.29	3470	3030	42.3	14.80	27	18,950
B	1	without	10.50	86.9	33.25	4065	3720	46.8	12.15	29.5	19,190
	2	annealing	10.25	82.8	33.69	3872	3570	46.8	10.00	18.5	19,620
	11	annealed	10.20	82.0	25.36	3526	3225	43.0	10.55	14	20,020
	12		10.20	82.0	26.09	3615	3190	44.1	15.25	29	20,900
C	1	without	10.25	82.8	35.02	3840	3345	46.4	8.25	29.5	20,800
	2	annealing	10.50	86.5	38.97	4380	4115	50.6	7.80	16	20,200
	11	annealed	10.25	82.8	28.14	3730	3420	45.0	15.75	39.5	21,100
	12		10.26	83.5	23.77	3435	3025	41.1	16.45	43.5	19,950
M (Mild Steel)	1	without	10.20	82.0	32.56	3579	2560	43.6	27.40	65.5	21,500
	2	annealing	10.30	83.0	30.60	3535	2510	42.6	31.25	62.5	21,300
	11	annealed	10.25	82.8	32.60	3253	2380	39.3	30.90	64.5	—
	12		10.20	82.0	23.17	3120	2180	38.0	30.40	65.5	20,950

Table 2. Results of Impact and Repeated Impact Test.

Impact Test							Repeated Impact Test						
Specimen	Mark	Method	Description of Specimen: Width (mm)	Thickness (mm)	Area (mm²)	Impact energy (kg. m.)	Specimen	Mark	Method	Description of Specimen: Diameter (mm)	Area (mm²)	Impacts per minute	Σ impacts to failure
A	3	without	10.8	9.00	97.1	2.90	A	5	without	13.0	132.7	110	4534
	4	annealing	10.4	9.45	98.4	4.69		6	annealing	,,	,,	100	3615
	13	annealed	10.3	8.10	83.5	0.498		15	annealed	,,	,,	100	8435
	14		10.2	8.25	84.2	2.938		16		,,	,,	100	3330
B	3	without	10.7	8.80	94.3	4.96	B	5	without	,,	,,	110	5496
	4	annealing	10.4	8.75	91.0	0.92		6	annealing	,,	,,	110	7871
	13	annealed	10.2	8.10	82.7	2.19		15	annealed	,	,,	85	4695
	14		10.2	8.30	84.7	2.07		16		,,	,,	95	5892
C	3	without	10.8	9.05	97.8	2.74	C	5	without	,,	,,	100	2450
	4	annealing	10.6	9.00	95.4	3.30		6	annealing	,,	,,	100	4578
	13	annealed	10.2	8.35	85.2	2.85		15	annealed	,,	,,	100	2775
	14		10.2	8.55	87.3	1.58		16		,,	,,	100	3460
M (Mild Steel)	3	without	10.1	8.35	84.0	22.5	M (Mild Steel)	5	without	,,	,,	112	12260
	4	annealing	10.1	8.35	84.0	21.4		6	annealing	,,	,,	120	7125
	13	annealed	10.05	8.00	80.4	21.5		15	annealed	,,	,,	122	7955
	14		10.0	8.10	81.0	22.4		16		,,	,,	100	6812

2. The resistence of the deposited metal to impact is from 2 to 4 kg. m. which is considerably less than the value for mild steel which is about 22 kg. m.
3. The result of the repeated impact test indicates that some of the deposited metal has practically the same strength as mild steel. However, where a mild steel fails at 8,000 blows, the probable average failure for the deposited metal may be around 4,000 blows.

Table 3 shows for reference the results of the chemical analysis of the core bar and the deposited metal used in the present experiment.

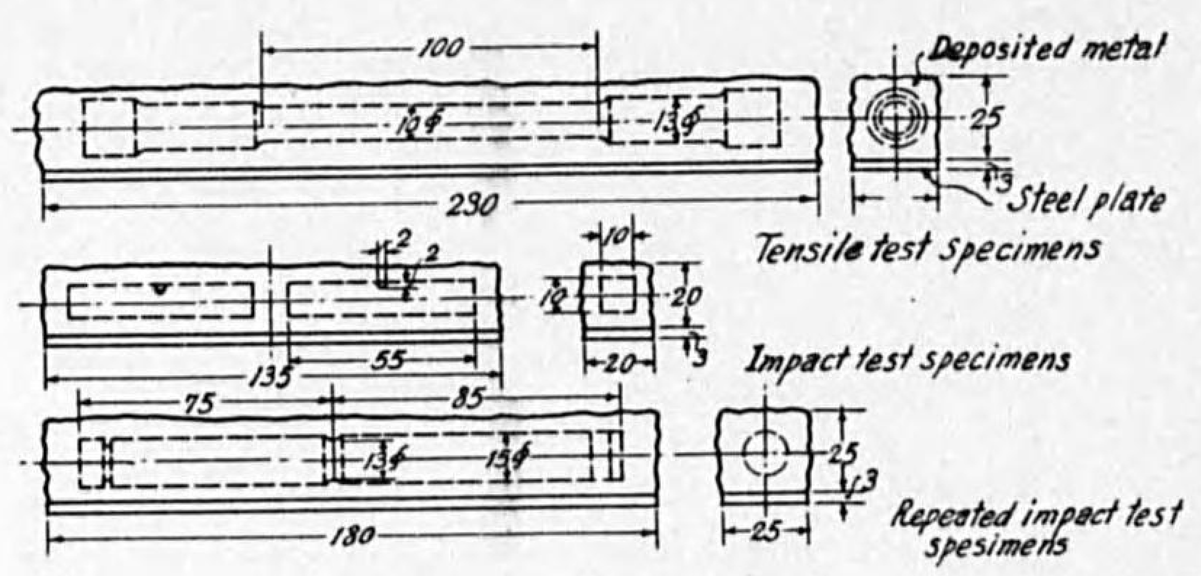


Fig. 1. Test Specimens.

Table 3. Chemical analysis of core bar and deposited matal.

Specimens	Material	Carbon C (%)	Silicon Si (%)	Manganese Mn (%)	Sulphur S (%)	Phosphorus P (%)
A	core bar	0.133	0.021	0.440	0.002	trace
	deposited metal	0.163	0.153	0.465	0.005	”
B	core bar	0.139	0.025	0.420	0.003	trace
	deposited metal	0.143	0.093	0.410	0.005	”
C	core bar	0.055	0.009	0.400	0.007	trace
	deposited metal	0.049	0.106	0.160	0.002	”

Studies as to Frost Action on Buildings in Japan.

BY

Prof. **Kyoji Yoshida.** *Kōgakuhakushi.*

Preface

In drawing up plans for modern structures and their styles based upon modern science it is of importance, the author considers, that not simply the studies of building forms, but also material, construction, execution, and preservation, and those of scientific work together with practical experiences should be prosecuted.

True, the sphere of the studies as to architecture has been so extensive, that without the synthesized studies of the physical and chemical problems pertaining to architecture, perfection can never be attained.

In the present paper the author treats a subject abstracted from the studies of freezing effect made in 1932, the studies of this phenomenon are apt to be made light of in Japan, and he aims at appealing to the professional world for reviews.

Now, in view of foreign-style buildings built one after another both in town and country, attention must be paid to freezing effect, to which buildings in colder parts of this land are subject.

Especially, with the arising of Manchukuo, building industry there has markedly been thriving. And in consequence, a number of requests has of late been made with regard to testing the frost resistance of the building materials to be used in the land; then the studies of frost action are in urgent need.

The testing methods of frost action are, however, comparatively complicated, and a rather long lapse of time is required for them.

In America and European countries a lot of propositions has been offered with regard to judging freezing effect on building materials, and yet general conditions of the land of ours are different from those of theirs, so these propositions are not to be applied here as they are.

Accordingly, the author has advanced with simplicity a proposition as to a frost-action judging in an endeavour to afford testing methods better adapted in Japan, making the freezing effect tests with solid carbon dioxide (Dry Ice) as freezing chemicals, and at the same time adopting practical examples and theoretical process simplified.

In addition, some items worthy of experts' attention to construction against frost action are summarized, together with the notes of the chief frost damage done to building materials in Japan.

1933.

I. General Remarks

Freezing effect is a phenomenon much affecting the durability of buildings. Excepting the districts where temperature does not fall below $0°C$, in colder parts of a land, due importance should be attached to this phenomenon. In the mainland of Japan, however, builders are apt to be less careful of structure and of selecting building materials. And in consequence, frost action often does unexpectedly bigger damage to buildings. The following are the oriental lands and districts where they have days when an average temperature is below $0°C$, and the months when they have such a temperature :—

Tokyo and Nagoya districts	Jan.—Feb.
Kyoto districts	Dec.—Feb.
Osaka districts	Jan.—Feb.
Mainland in general	Dec.—Feb.
Hokkaido	Nov.—Apr.
Korea	Nov.—Mar.
Manchukou	Nov.—Mar.
Siberia	Oct.—Apr.

Besides, on account of geographical and topographical features in warmer districts temperature sometimes falls below $0°C$.

If temperature be below the freezing point, frost action will not do so much with the degree of coldness as with the number of days having temperatures below $0°C$, and with the times of rising and falling of temperature. In short, frost action to buildings is ascribable to the change of the volume of water when it is frozen.

The volume of any material contracts by the fall of temperature. Since the volume of water begins expanding when temperature is $4°C$ and that is frozen at $0°C$, and also specific gravity of water at $0°C$ is 0.99987 and that of ice at the same degree of temperature is 0.91673, then the volume of ice is $0.99987 \div 0.91673 = 1.0903$: the increase of volume is 9%. The value of pressure then produced remarkably differs according to laboratory workers,[*1, 2, 3] and yet several latest investigations synthesized show about 2,000 kg/sq. cm.. General building materials cannot stand this pressure. Such being the reason, in order to do away with stress produced when the free water contained in a material freezes, we have to leave about 10% margin of the free water to a void capacity.

Now, regarding the freezing of the free water in a material the following items must seriously be considered :—

(1) In many cases the free water in a material contains chemicals, and in consequence freezing point is generally more or less below $0°C$.

(2) Some formation of void of a material makes the free water in it come out of the surface, and there it is frozen.

(3) The volume of water expands, when frozen, in a confined space of a material, and the expanded bulk destroys the material texture. Therefore, this phenomenon has directly much to do with building materials, with the states of water absorbed among the parts of a structure, and with material strength, specially with the ratio between void capacity and free water, and above all, water absorption has serious concern with the said phenomenon.

* These numbers relate to the references on page 68.

The exterior parts of a building subject to freezing effect are these: wall, pillar, basement, roof, window-sill, floor surface, moulding, water supply and dranage systems, and other parts. And the interior parts: any part directly connected with the exterior parts, especially the parts exposed to wetness or connected with water such as toilet, kitchen, cellar, basement, bath, and other parts.

The following are regarded as the results of freezing effect :—

(1) crack, exfoliation, disintegration.
(2) decrease of strength of a material.
(3) change of formation, accelaration of weathering.
(4) fading of colour and lustre.

2. Examples

(1) Roofing Tiles

Freezing effect differs markedly according to burning temperature of

Fig. 1. Detail Fig. 2. General view.
Frost damaged rooting tiles facing the east for 12 years after installation.

tiles, to their form, and to the surface finish. And also the degree of frost damage is dependant upon the location where a material is applied, that is to say, the north side, the north-east side, and the east side bring about the worst effect, next the west side, and the south produces the least effect.

State of Effect

(a) In the first place cracks take place, which is followed by partial exfoliation, and then the expanded bulk of free water breaks materials.

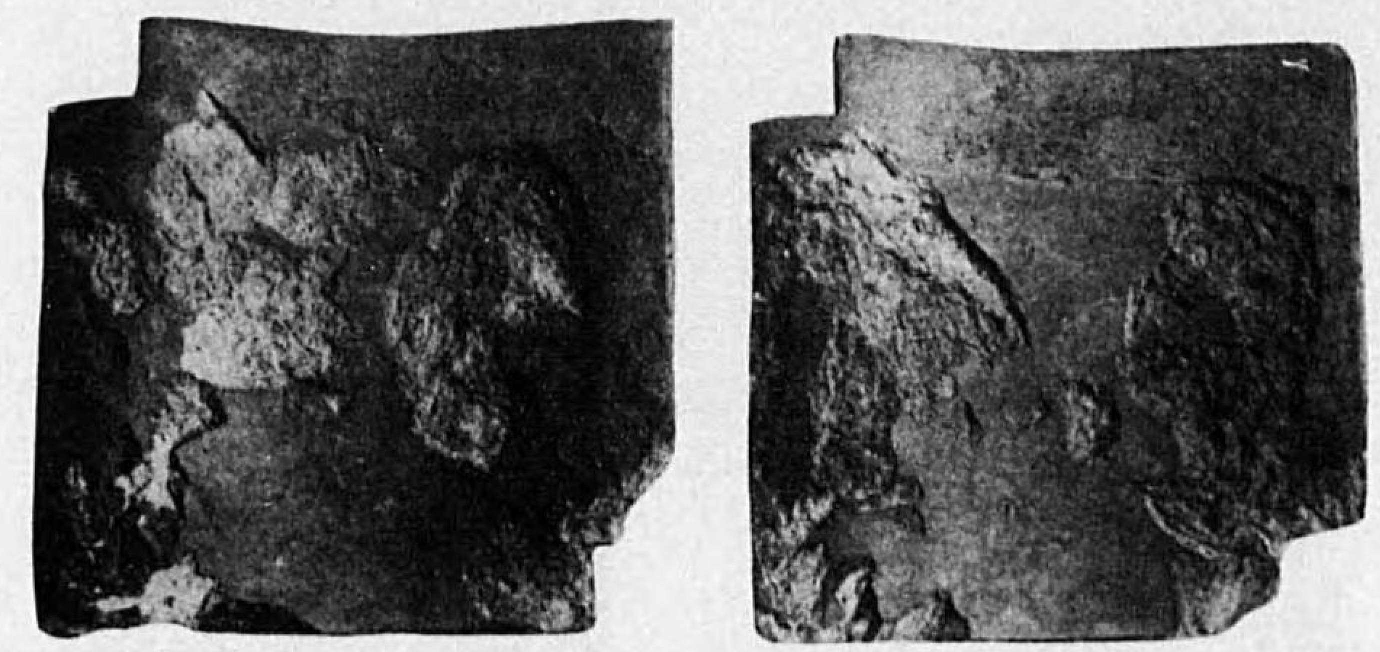


Fig. 3.
Showing frost damage to the surface, lap parts show little damage and rear side less damage.

(b) Stratiform exfoliation begins parallel with the surface of a tile.
(c) Any flaw in a tile prior to setting the tile causes much absorption of water, resulting in earlier breakage.
(d) Detail carvings and exterior corners are more subject to freezing effect.
(e) The portion of laps of roofing tiles in Tokyo suffer less frost damage, both in Hokuriku and San-in districts water between tiles freezes in winter, and then sometimes tiles are displaced.

(2) Wall Tiles

The tiles on the interior walls equipped with no heating system as well as the exterior tiles suffer freezing effect, especially glazed tiles suffer worse effect, and some surfaces of them fall off partly. The outer

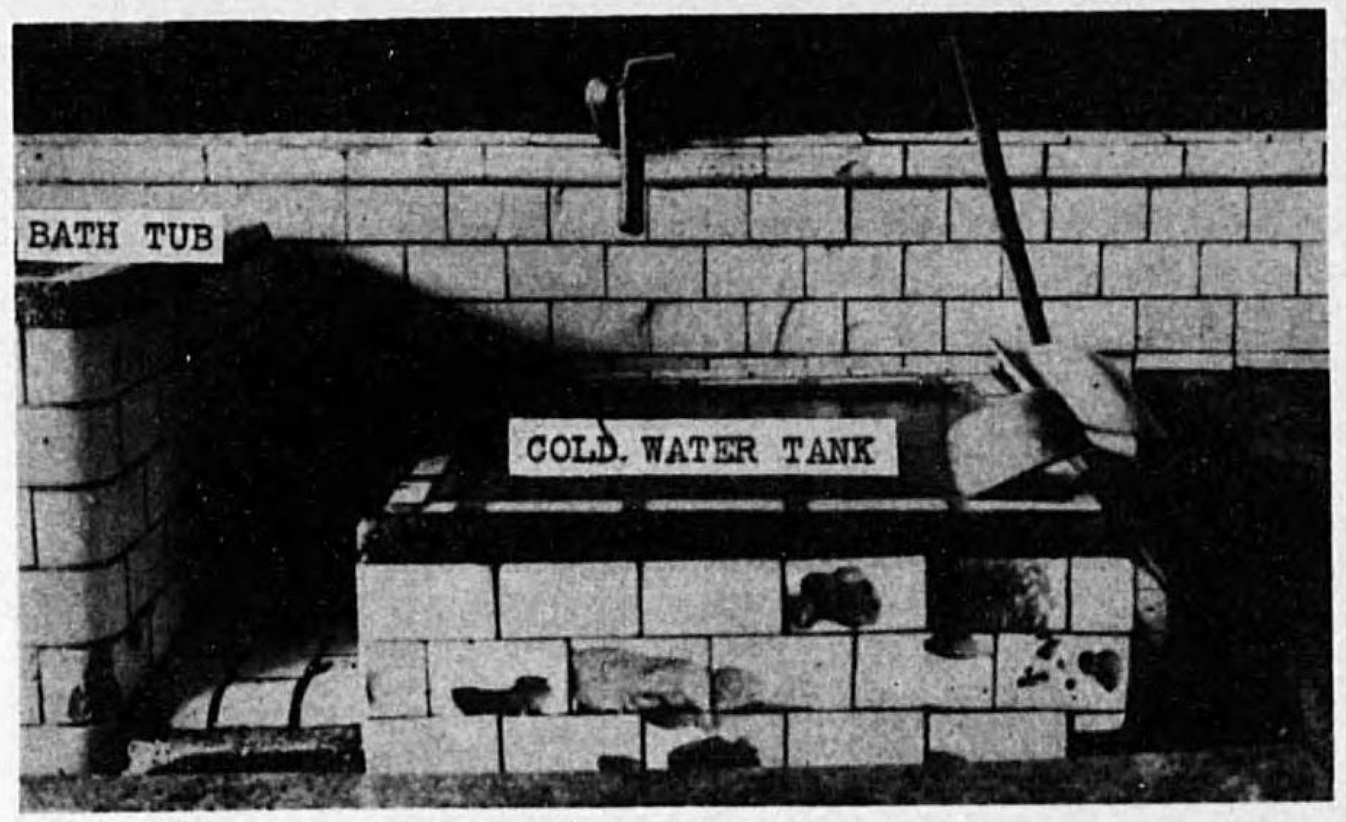


Fig. 4. Frost damaged tiles on a wall in a bath room of a hotel in Nikko.

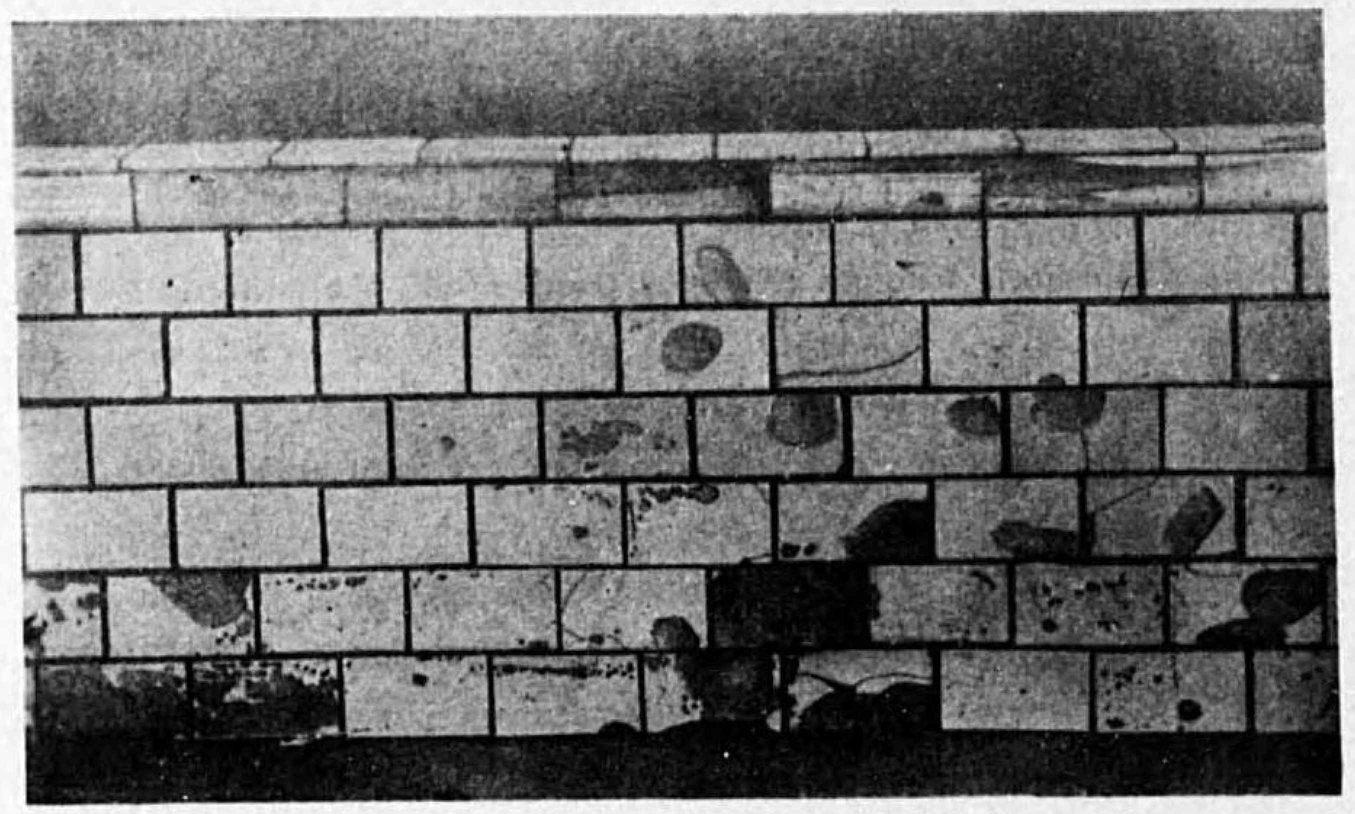


Fig. 5. Another part of the same in Fig. 4.

wall-tiles facing the north, the north-east, and the east suffer intense damage.

(a) Interior Walls

Much damage is done to the tiles applied to the places having much to do with water such as bath, toilet, and kitchen. The author witnessed much damage done to the tiles of the bath room of a certain hotel in Nikko. A hot-water bath tub suffers less freezing effect as compared with a cold water tank.

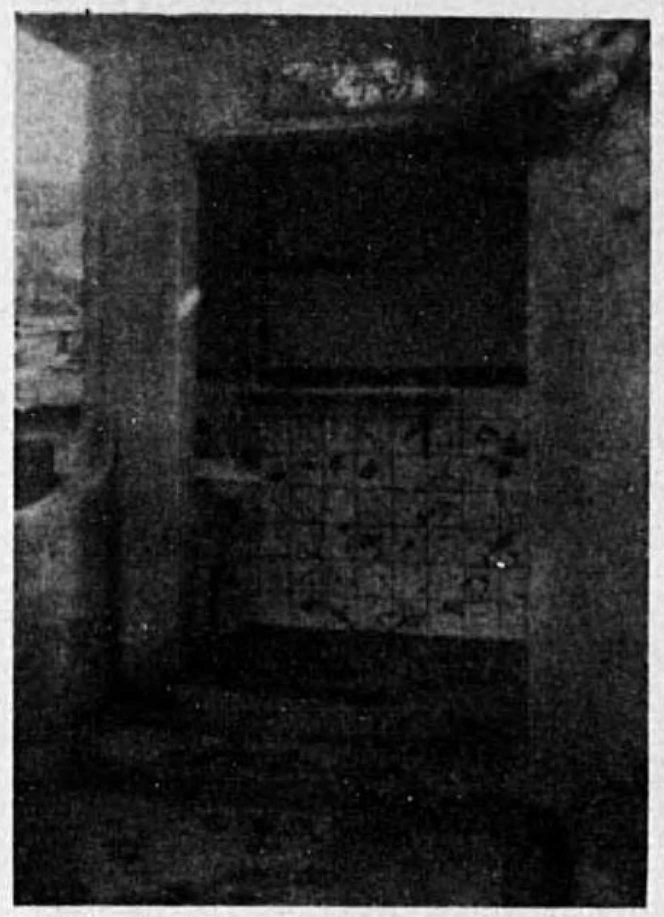


Fig. 6. Frost damaged wall tiles at a place devoid of protection.

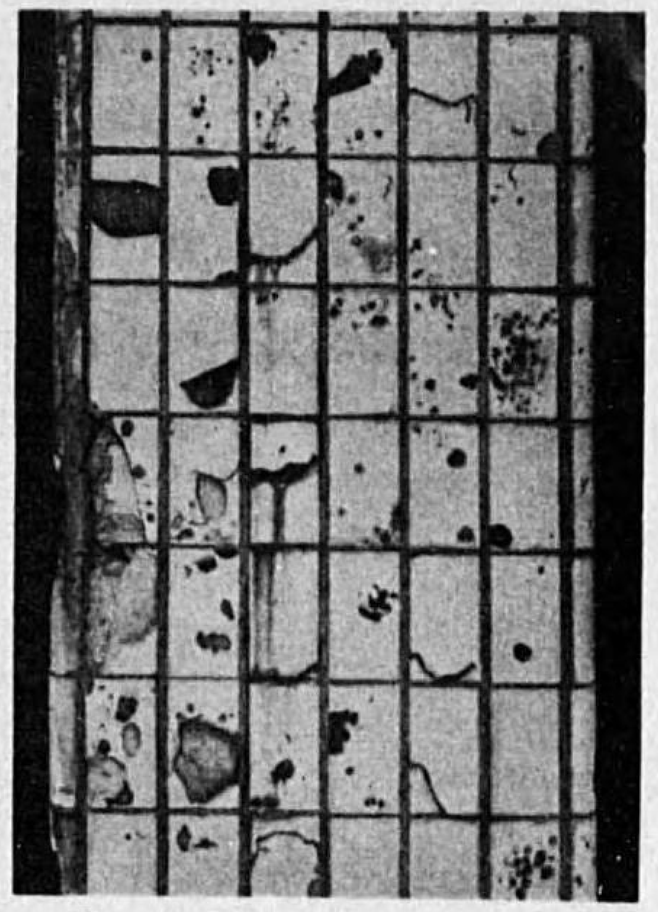
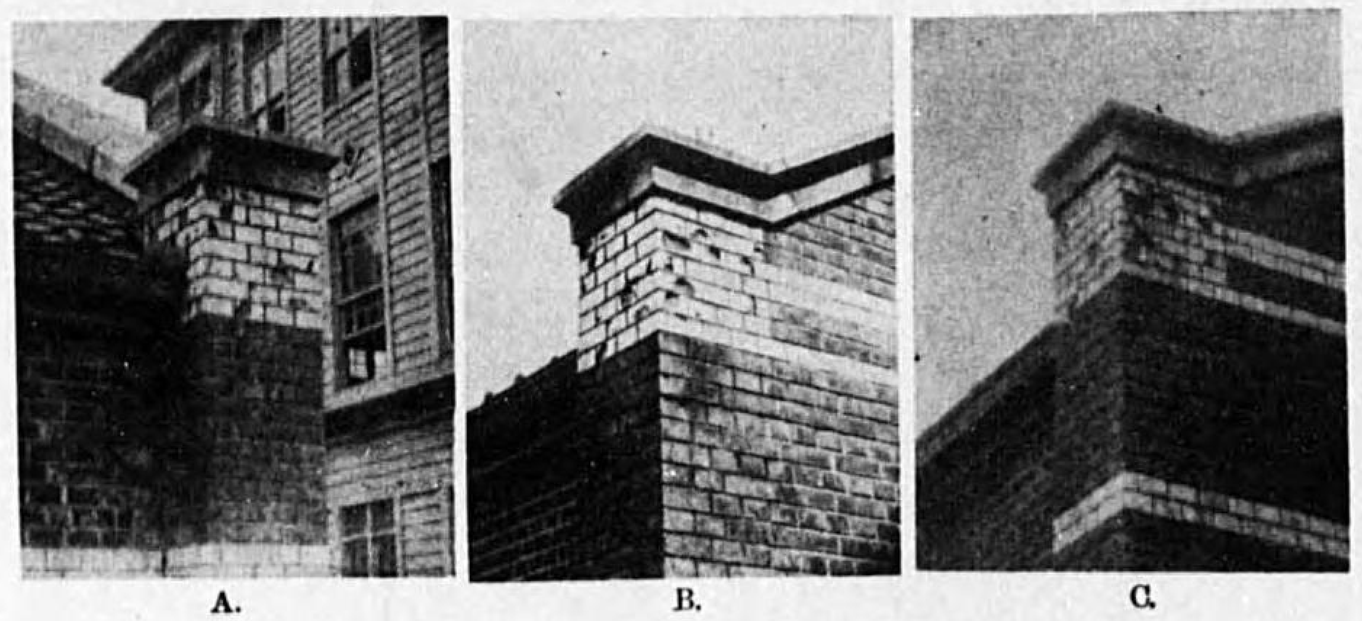


Fig. 7. Frost damaged tiles on the surface of gate posts.

(b) Exterior Walls

The exfoliation of the surfaces of glazed tiles hanging on the exterior walls is generally observed. The cause of this damage can never be attributable only to freezing effect.

Shop fronts, show-window sills, gate posts and general outer parts of a building disclose exfoliation. In the early stage of freezing effect the process is quick, and afterward it is slow. The author examined for five years intervals the process of frost damage to one corner of a school building of Waseda University, and in the later part of these periods little change was observed.



A. B. C.

Fig. 8. Shows the progress of the frost damage to the exterior wall tiles.
A: Shows the state in 1921.
B: Shows the state in 1926.
C: Shows the state in 1931.

The frost damage to tiles and other similar materials is not simple in its cause ; in hanging them careless hammering produces much impact, which breaks the formation of tiles. Thus, the tiles bring about a cause of much frost damage to be done afterward.

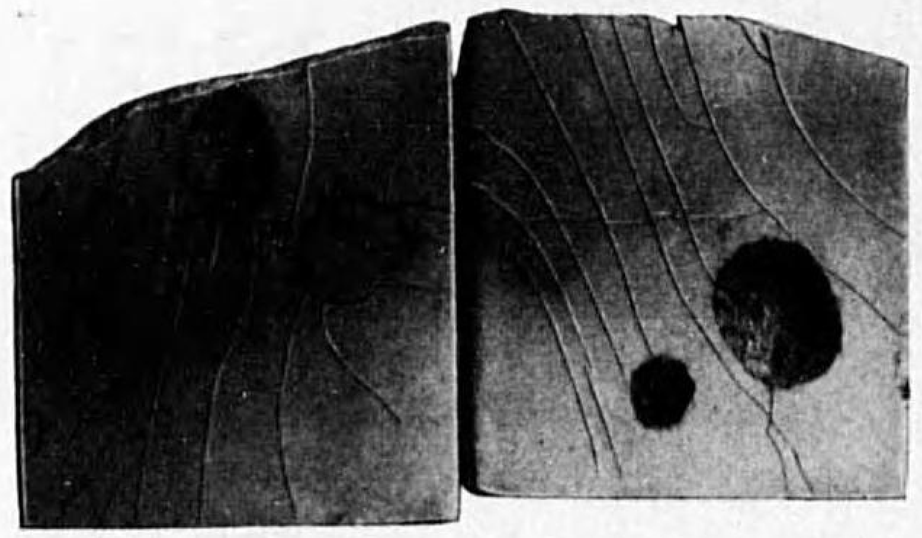


Fig. 9. Frost damage to glazed tiles caused by the water absorption of body.

It is a hasty conclusion to regard the cracks in glazed tiles as the results of freezing effect ; in general, cracks are attributable to the change of temperature, and to the volume change of body caused by absorption.

From spring till the end of summer minute cracks often appear on tile surfaces. These cracks are not so much the results of freezing effect as an inducing cause of the effect. So with terra-cotta.

(3) Building Stone

The nature of building stone is complex on account of variety being manifold. Then, it often is difficult to determine which is the result of freezing effect after examining the weathering condition of a material. In the case of sandstone and tuff, these parts and things disclose exfoliation : the angular point of an exterior corner, detail parts of carvings, a capital, the carved tip of a stone lantern, and the surfaces of a pavement. Hard stone absorbs less water, yet the crystallized parts or the cracked portions of it suck water in the stratiform way. When water thus finding its way in hard stone freezes, the material suffers frost damage. While soft stone, which sucks much water, quite often crumbles when the water in it freezes. In the weathering of building stone, if stone once undergoes chemical action it will suffer easily freezing effect, if stone suffers freezing effect it will easily undergoes chemical action. Therefore, as a result, we must examine both freezing and chemical action put together in the weathering consideration.

(4) Concrete and Cement Mortar

Concrete and cement mortar resist, after hardening, comparatively intense coldness. But during setting and ealier stage of hardening these materials bring about, if exposed to the temperature below $0° C$, astounding results of frost damage. It is, however, difficult to point out real degree of freezing effect, hardening and the increase of strength in age being slow.

A wall rendered with cement mortar or a Jinzo-seki (artificial stone) wall often disclosed cracks, through which water find its way into it, and freezes therein. Then, we see quite often exfoliation of the rendered wall.

(5) Plaster and Glazer's putty

That freezing effect during plastering execution is big has generally been known, and yet in the normal state no big damage is done to a wall after hardening.

It is, however, natural that a wall should suffer freezing effect owing to the freezing of the free water in the wall.

In plastering, free water will remain in the wall. If the wall in this state suffer frost action, the wall surface will be deprived of lustre along the trowel traces, the surface layer will be in pluverization. And later on, the surface will peel.

When glazer's putty is applied, owing to a crevice or crack made by the freezing of water finding its way there, some portions of the putty fall off.

(6) Metal Pipes and Valves

Consequent upon the stress due to the internal water frozen, iron and lead pipes and valves may break.

Fig. 10. A cock broken by freezing when the thickness of it is uniform.

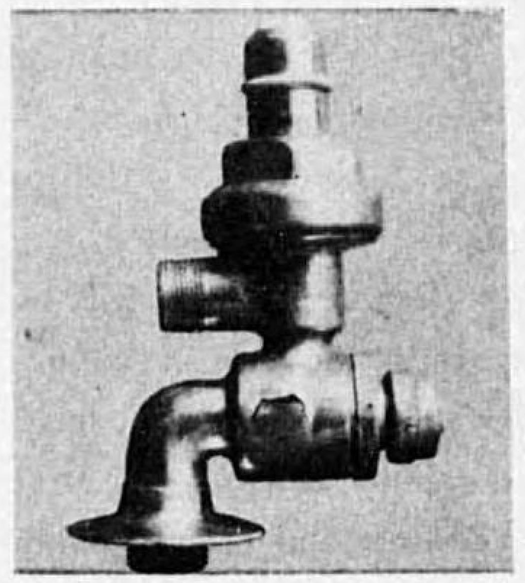


Fig. 11. A flushing valve broken by freezing when the thickness of it is uneven.

(7) Execution and Construction

The degree of frost damage in many cases depends upon the methods of construction and execution. In general, joints and contacts of materials are needed to be closed together, and in consequence, there takes place capillary attraction inducing water to come in. When the water therein freezes the crevices appearing will all the more be large. Such a phenomenon is often seem, for instance, in a metal flashing. Sometimes water permeats into the layers of an asphalt roofing; when the water therein freezes, the layers are often stripped off, and occasionally the

layers will break owing to asphalt growing fragile by the effect of the cold.

3. Freezing Tests and Judgements

In order to presuppose the actual results of frost damage to a building, it is necessary to see the resistance degree to freezing effect by the test of a short period of time. The repetition of freezing and thawing brought about by the change of temperature acts upon a material, and there we see freezing effect. However, when there is water in a fissure or crack of a material, the freezing of only one time will sometimes do damage to it. While, a texture saturated with water suffers frost after some repetition of the water freezing. Such being the case, it is a little difficult to presuppose the degree of damage to a material by the freezing test of a short lapse of days. A variety of methods for the purpose has hitherto been deviced, but the author has yet found no appreciable method of a standard to be adapted for practice in Japan. The following are the freezing tests and judging methods in the past the author knows of :—

(1) Tests nearer to natural freezing

The test methods are to produce the almost expected results of natural freezing effect. In the first place, a material with free water in it must be frozen, and thawed, next, we repeat the alternate freezing and thawing 25, 40 or 50 times.[4] Or, we make this kind of tests continuously until the material breaks owing to freezing. The times required for the disintegration of the material show the degree of resistance, and another way is to see the resistance by the weight decrease obtained after the tests.

(2) Freezing Tests with Chemicals

The tests are made to presuppose the state of frozen breakage through the pressure caused by the crystallisation of chemical compounds. Mr. Brard[5] was the first to use saturated solution of sodium sulphate, sometimes solution of 28%, 14% being used, often potassium sulphate and sodium chloride used.[6 7]

This kind of tests often produces the results more intense than those of natural freezing, or sometimes no appreciable result is yielded. The results are to be expressed by the change of weight and by examining the appearance of the test material.

(3) Judgement by Saturation Coefficient

When water freezes its volume expands by 9%. Suppose, therefore, there were a theoretical relation $\frac{A}{P} \leq \frac{100}{109} \fallingdotseq 0.91$ between A and P, where A is saturated value of water absorbed and P the capacity of pore space, there would be no stress. And the frost action judging test through how much void was filled with water were made by Profs. Bauschinger[8] and Hirschwald.[9] Prof. Hirschwald called the ratio between water absorption and porosity as saturation coefficient, that is, Sättigungskoeffizient.

$$\text{Saturation Coefficient} = \frac{\text{water absorption}}{\text{porosity}}$$

According to Prof. Hirschwald, theoretical value 0.9 is too big. As the results from a lot of investigations based upon a variety of building stone, he says that the value of 0.8 is enough. He says further, there exists 0.8 as an exception, in accordance with, the state of pore arrangement and the softening caused by the saturated water in a texture. The saturation coefficient is named as Krëuger's[10] ratio, and as the ratio 0.80–0.85 should be adapted. Schaffer[3] advises the test makers in general, like a proposal by Ledue, that they should not depend solely upon saturation coefficient for judging the degree of frost action, but also ought to take natural freezing tests into consideration. Prof. Hirschwald makes judging tests of freezing effect by the relation of saturation coefficient, and the ratio of strength obtained when a material dries and saturates. In 1905, adapting the relative strength of building stone obtained when it is dried and wetted, Dr. Heinrich Seipp[11] studied freezing judging.

As is mentioned above, much importance is attached to saturation coefficient in judging the degree of freezing effect, yet it is difficult to find the limit of measurement between water saturated volume and void capacity. The volume of saturation can be measured by immersion, but to measure the real void capacity is quite difficult. The free water content in a material changes according to absorption, to the lapse of time

after saturation, to the arrangement of pores, to the size of pores, to the situation where a material is applied, and to the atmospheric condition. The measurement of void capacity differs according to the authorities, and in consequence, different results are to be produced.

As a means of judging the resistance to freezing, McBurney[12] proposes to adopt Schurecht's criterion instead of saturation coefficient (Krëuger's ratio). Mr. Schaffer[3] criticizes that Schurecht's criterion is devoid of the studies for the totally enclosed pores which can not easily be saturated. Schurecht's criterion is shown in 48 hours absorption divided by 48 hours absorption plus 5 hours boiling. Palmer's[13] criterion is saturated absorption in cold water divided by 5 hours boiling absorption. By both suggestions, boiling absorption is treated as the foresaid void capacity.

(4) Proposition of Frost Action Judging of Building Research Station of England

The following is a proposition[14] offered by the Building Research Station of England :—

The volume change of a material undergoes is measured ; the stress required for the material transformed by freezing through stress/strain of the material is calculated, then the strength of the material is compared with the result. Thus, freezing effect is judged.

This proposition of judging tests must be quite rational. But there must be some room, so the author thinks, for further consideration as to the possibility of applying Young's modulus even in this case.

4. The Author's Tests for Freezing Effect

Field tests being about the most reliable results for frost damage, the test results obtainable for convenience' sake in the laboratory being apt to be different from those of field tests. And it is not advisable to make tests with the crystallisation of salt, because some unknown factors are often conducted in the crystallisation. Thereupon, the tests with the procedure to freeze the free water in a material seem proper, yet a phenomenon of freezing disintegration bears a complicated relation with a few other properties, resulting, in many cases, in the difference of the test results from field tests.

The building materials with definite market forms produce quite different results of freezing effect according to their forms, then, tests should be made in two ways, that is, a test with a piece of a material, and that with a market form. And, the difference of the surface finish of specimen

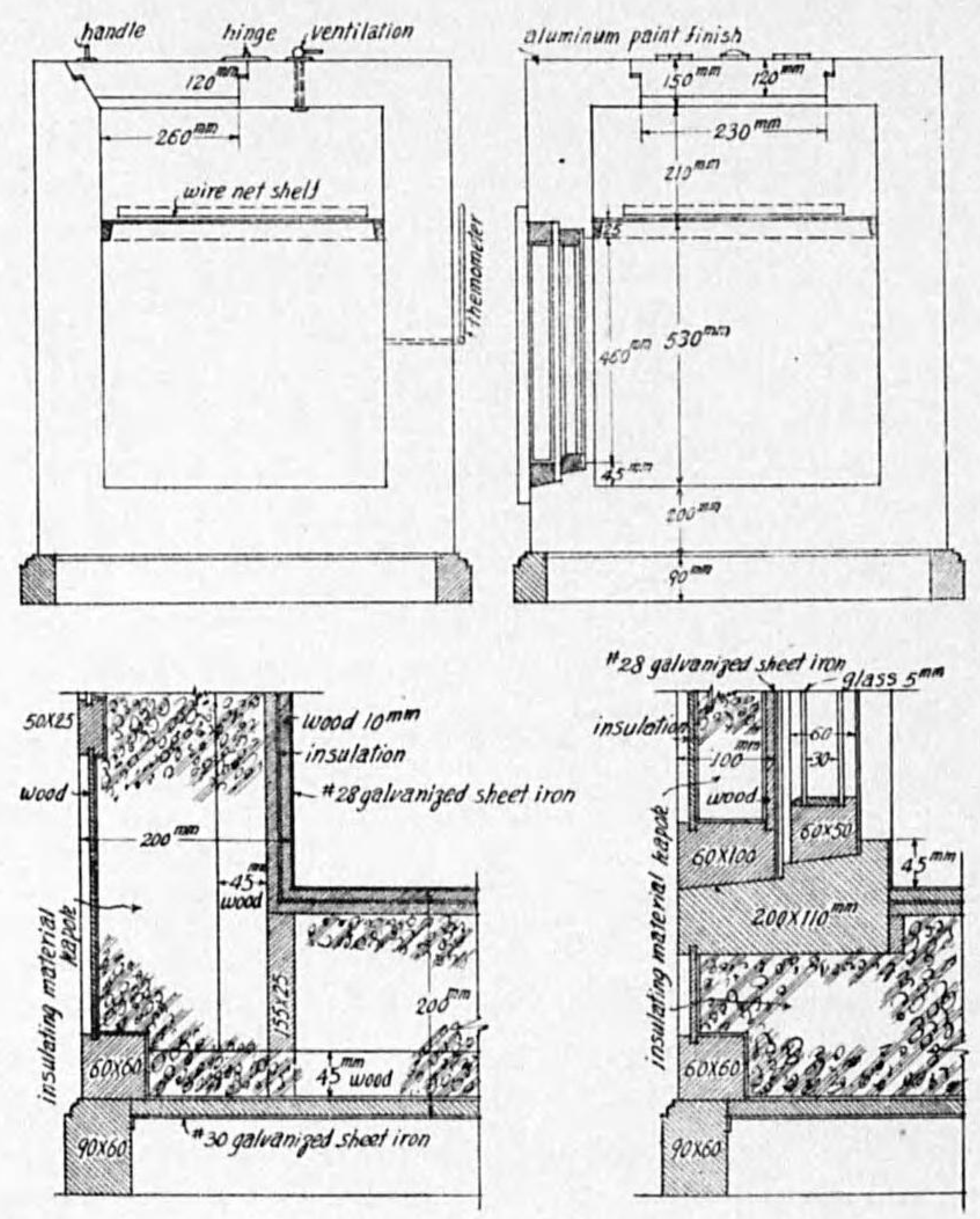


Fig. 12. Low Temperature Freezing Box.

acts much upon the results; in general, the test with a piece specimen yields more intense results. After all, for judging material itself, building stone, cement, concrete, and other similar materials should be tested chiefly with a piece specimen. And in the point of the tests with market forms, due importance ought to be attached to such materials as clay products, this being a better function to manufacturing.

(1) Test Apparatus and Procedure

As is shown in Fig. 12, the author used a low temperature testing box, and as freezing chemicals solid carbon dioxide (Dry Ice) was applied.

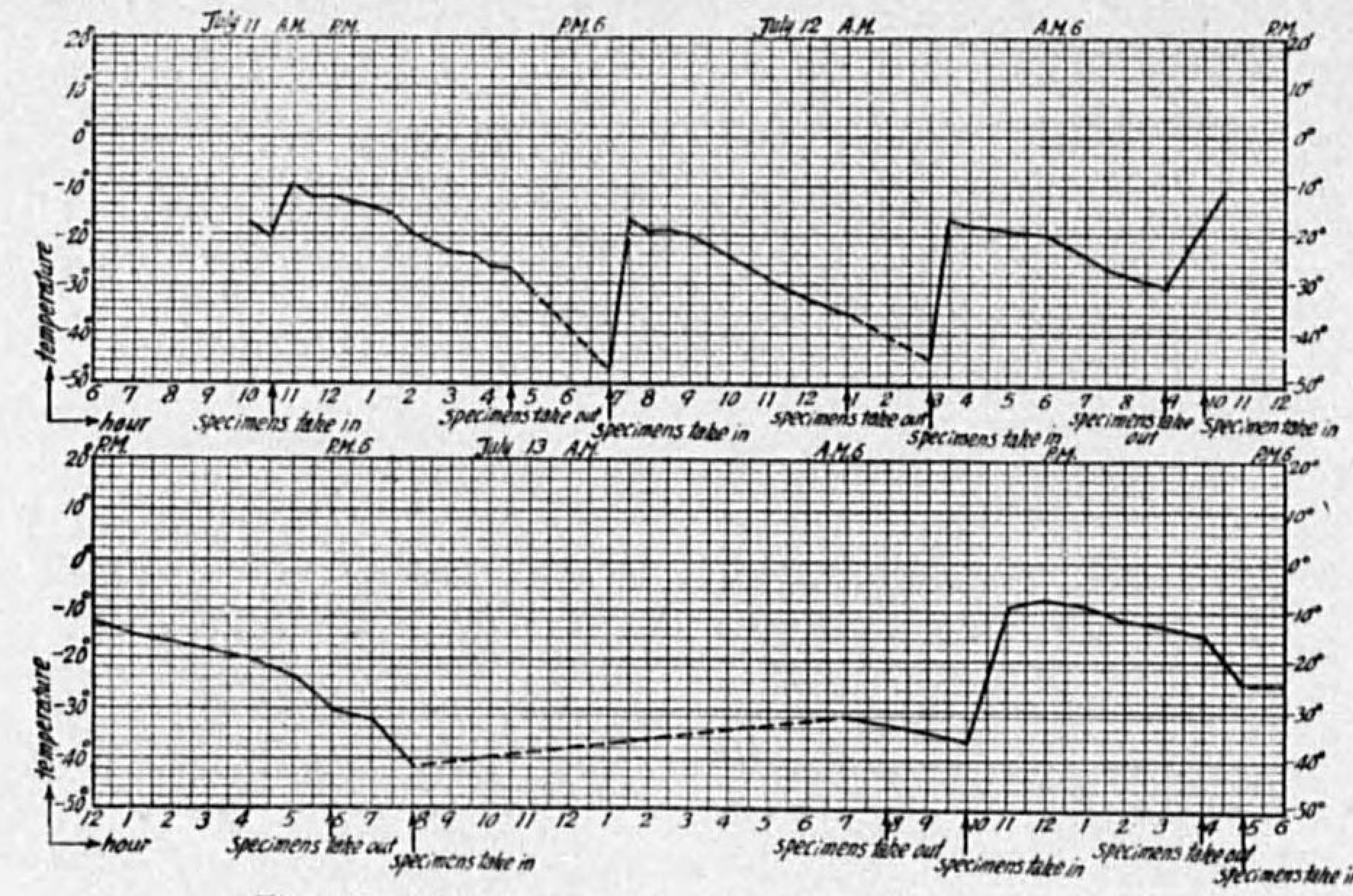


Fig. 13. Charts expressing the temperature change in the freezing box during test.

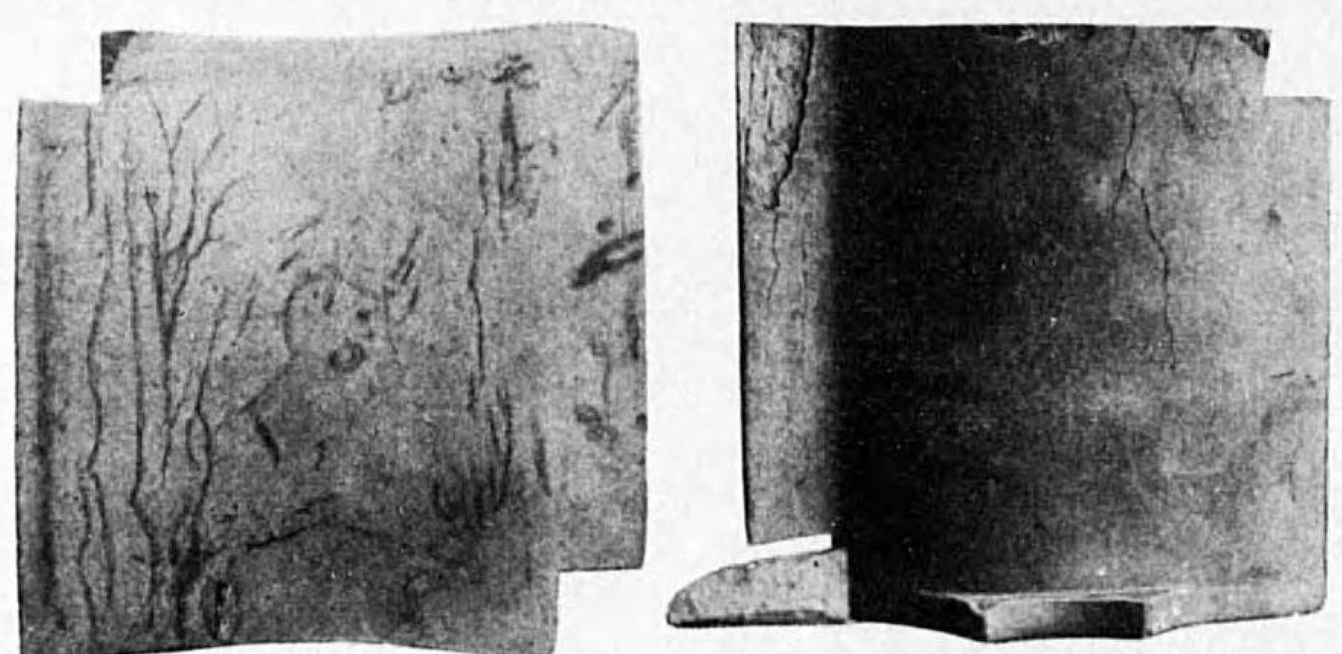
(Dotted line indicates connection of the lines though not practically measured)

Though the lowest temperature $-60°C$ was obtained, yet we could not do away with temperature deflection brought about by taking in and out the test materials. The temperature in the box during tests was fluctuating between $-10°$ and $-48°C$. And the lapse of times required for keeping the specimen in the box was indefinite, as we had to wait for the material to freeze perfectly.

(2) Test with Fragments

Immersing specimens in water for 96 hours to make them absorb water perfectly, we transferred them into the freezing box. The pieces were deposited therein for one or two hours exposing to the temperature -20— -30° C. Assuming it was sufficiently frozen, the pieces were taken out and put into water of normal temperature with much care, and made them thaw therein; the pieces were again put into the freezing box, thus, freezing and thawing were repeated fifty times as one cycle. Then, we measured the decreased weight of the specimens different from the original in the dry state. The loss of weight by freezing in percent was computed, and the changes of the outside appearance were observed.

Fig. 14. A piece specimen of roofing tile tested by salt crystallisation, showing the side view that has many stratiform cracks.



A. B.

Fig. 15. Specimen of a roofing tile after freezing and thawing test.
A. Immersion thawed specimen.
B. Boiling thawed specimen.

(3) Test with Market Forms

Here, tests were made with roofing tiles of clay products, wall tiles,

Table 1. Results of Test with Fragments.

No.	Kind of Materials	Weight of Specimen			Weight decrease by freezing $\frac{W-w}{W} \times 100\%$	Porosity %	Water absorption %	Specific gravity
		Before freezing W gr.	After freezing w. gr.	Difference $(W-w)$gr.				
1	Marble	39.60	39.60	0	—	1.6	0.48	2.721
2	Granit (Inada-ishi)	35.10	35.00	0.10	0.216	0.79	0.49	2.613
3	Andesite (Komatsu-ishi)	39.8)	39.80	0	—	7.8	3.18	2.684
4	Tafaceous Sand Stone (Boshu-ishi)	24.30	19.10	5.20	21.3	15.3	7.18	2.537
5	Tuff (Ōya-ishi)	32.20	15.50	16.70	51.8	34.3	23.86	2.209
6	Tuff	30.00	29.70	0.30	1.0	10.1	4.62	2.646
7	Fine grain Sand Stone	31.00	30.90	0.10	0.322	19.6	9.68	2.554
8	Brick	21.80	21.70	0.10	0.455	15.8	7.78	2.414
9	Roofing Tile French type	75.70	75.60	0.10	0.131	20.0	10.7	1.90
10	Roofing Tile Spanish type	64.60	64.50	0.10	0.154	24.0	12.8	1.84
11	Roofing Tile Spanish type	92.85	92.80	0.05	0.0538	15.0	7.6	1.99
12	Japanese Roofing Tile (Sanshu)	46.85	46.80	0.05	0.106	35.0	21.1	1.68
13	Japanese Roofing Tile (Enshu)	61.20	61.10	0.10	0.163	30.0	17.8	1.73
14	Japanese Roofing Tile (Saitama)	63.40	63.30	0.10	0.157	21.0	11.3	1.88
15	Terra-cotta No. 1	35.80	35.70	0.10	0.279	18.0	9.2	1.92
16	Terra-cotta No. 2	53.40	53.40	0	—	19.0	10.0	1.90
17	Wall Tile	27.60	27.60	0	—	12.0	5.4	2.21
18	Scratched Wall Tile	44.25	44.25	0	—	6.0	3.1	2.12
19	Cement Tile	38.55	38.55	0	—	15.0	7.0	2.15

and natural slates, all of them being formed into market size. The specimen, which was dried to constant weight, was immersed in water for 96 hours for perfect absorption. Then, it was put into the freezing box for 5–6 hours, after that it was taken out, and was thawed. Weighing the specimen and observing the appearance possibly changed, the specimen was frozen and thawed alternately. Making this process repeated again and again, the author formed an estimate both of the weight and the appearance changed.

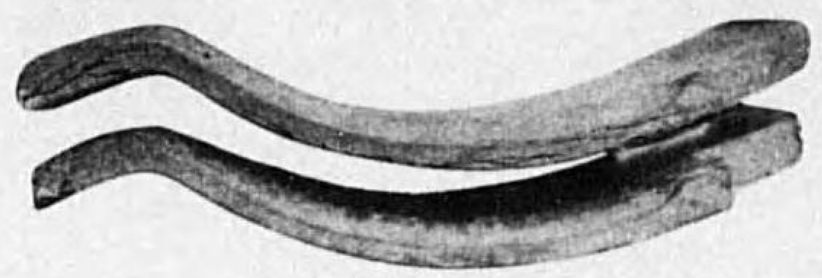


Fig. 16. Specimen of roofing tiles after freezing and thawing test, showing the side view.

The author adapted two systems for thawing the frozen specimen ; one is to thaw the frozen specimen in water of normal temperature, and left it as it was for absorption : the other is to boil it for 1–3 hours for absorption after thawing it in water. When the two different results thus produced were compared with each other, the result through the latter system suffered more intense frost action.

Now, in the case of the boiling process the difference of the temperature to which a material is exposed, is as much as 130° *C*, and in consequence, the results of freezing seem to be strikingly intensified owing to the thermal expansion.

(4) Test Results

The loss of weight by freezing in percent shown in these freezing results affords the value of much the same figures for a variety of materials. As for building stone, there are manifold textures and crystals of minerals, then the range of the loss of weight by freezing in percent is quite wide, ranging from what is larger than clay products to what is too small to measure decreasing ratio. The sorts of stone in the same line as tuff generally are subject to freezing effect.

According to the results of freezing tests of building stone obtained in "*Eizen-kanzaikyoku*" of the Department of Finance, comparatively

Table 2. Results of Freezing and Thawing Tests. (Thawed in water)

Testing Schedule				Weight changes		
				Japanese Roofing Tile (Kawara)		
Date	Time	After Freezing or Thawing	Remarks	Sanshu kg.	Kyoto kg.	Jigawara kg.
			Original wt. Before testing.	2.27	2.30	2.45
			Original dried wt.	2.26	2.30	2.45
July, 11, 1931,	a.m. 10- 0		Water absorbed 96 hrs. immersion	2.67	2.65	3.01
	p.m. 4-30	Freezing		2.66	2.65	2.99*
	p.m. 7- 0	Thawing	in Water, 2 hrs. 15°C	2.70	2.65	3.02
July, 12, 1931,	a.m. 1- 0	Freezing		2.70	2.67	3.00
	a.m. 3- 0	Thawing	in Water, 2 hrs. 14.5°C	2.70	2.65	3.03
	a.m. 9- 0	Freezing		2.80	2.70	2.99
	a.m. 10- 0	Thawing	in Water, 2 hrs. 15°C	2.70	2.67	3.01
	p.m. 6- 0	Freezing		2.67	2.66	3.00
	p.m. 8- 0	Thawing	in Water, 2 hrs. 16.5°C	2.70	2.66	3.02
July 13, 1931,	a.m. 8- 0	Freezing		2.70†	2.66	3.01
	a.m. 10- 0	Thawing	in Water, 1 hr. 15°C	2.72	2.66	3.02
	p.m. 4- 0	Freezing		2.96*	2.66	3.00
	p.m. 5- 0	Thawing	in Water, 1 hr. 15°C	2.70	2.65	3.01
July, 14, 1931,	a.m. 9- 0	Freezing		2.68*	2.66*	3.00
	a.m. 12- 0	Thawing	in Water, 3 hrs. 16°C	2.70	2.70	3.00
July, 15, 1931,	a.m. 9- 0	Freezing		2.68	2.64	2.98*
	a.m. 11- 0	Thawing	in Water, 2 hrs. 15°C	2.69	2.64	3.01
July, 16, 1931,	a.m. 9- 0	Freezing		2.69	2.65	2.98*
	a.m. 11- 0	Thawing	in Water, 2 hrs. 16°C	2.71	2.66	3.02
July, 17, 1931,	a.m. 9- 0	Freezing		2.67*	2.65	3.01
	p.m. 0-30	Thawing	in Water, 2.5 hrs.	2.69	2.65	3.00
July, 19, 1931,	a.m. 10- 0	Freezing		2.75	2.66	3.03
	p.m. 0-30	Thawing	in Water, 2.5 hrs.	2.70	2.65	3.01
July, 21, 1931,	p.m. 3-30	Freezing		2.71	2.65	3.04
			Final dried wt. after Testing	2.26	2.36	2.43

Legend. † cracks appeared.
* weight lost remarkably.

Table 3. Results of Freezing and Thawing Tests. (Thawed in boiling water)

Testing Schedule				Weight Changes		
				Japanese Roofing Tile (Kawara)		
Date	Time	After Freezing or Thawing	Remarks	Sanshu kg.	Kyoto kg.	Jigawara kg.
			Original wt. before testing			
			Original dried wt.			
			Water absorbed 96 hrs. immersion.			
July, 12, 1931,	a.m. 1- 0	Freezing		2.70		2.99
	a.m. 3- 0	Thawing	2 hrs. boiling	2.73		3.02
	a.m. 9- 0	Freezing		2.73		3.00
	a.m. 10- 0	Thawing	1 hr. boiling	2.78		3.03
	p.m. 6- 0	Freezing		2.75		3.02
	p.m. 8- 0	Thawing	2 hrs. boiling	2.78		3.03
July, 13, 1931,	a.m. 8- 0	Freezing		2.78		3.00
	a.m. 10- 0	Thawing	2 hrs. boiling	2.80		3.05
	p.m. 4- 0	Freezing		2.78		3.02
	p.m. 5- 0	Thawing	1 hr. boiling	2.82		3.04
July, 14, 1931,	a.m. 9- 0	Freezing		2.80		3.02
	a.m. 12- 0	Thawing	3 hrs. boiling	2.80		3.05
July, 15, 1931,	a.m. 9- 0	Freezing		2.78†		3.02
	a.m. 11- 0	Thawing	2 hrs. boiling	2.82		3.04
July, 16, 1931,	a.m. 9- 0	Freezing		2.79†		3.02
	a.m. 11- 0	Thawing	2 hrs, boiling	2.82		3.06
July, 17, 1931,	a.m. 9- 0	Freezing		2.80		3.03
	p.m. 0-30	Thawing	2.5 hrs. boiling	2.76		3.02
July, 19, 1931,	a.m. 10- 0	Freezing		2.79		3.05
	p.m. 0-30	Thawing	2.5 hrs. boiling	2.83		3.07
July, 21, 1931,	p.m. 3-30	Freezing		2.64		3.08
		Final dried wt. after testing		2.289		2.45

† cracks appeared.

bigger absorption is observed in the materials such as tuff and andesite, while loss of weight by freezing is little ; marble shows less absorption, while comparatively bigger is loss of weight by freezing ; such a material as granit shows a little bigger absorption by the side of marble, and yet in comparison with other stone materials, the former shows the least loss of weight by freezing. Being similar to the author's tests some of tuff, show remarkably big ratio both in absorption and weight decrease.

Some roofing tiles, when they suffer frost action, partially exfoliate, and other tiles wholly exfoliate. So in two different points, testers should examine them ; as to roofing tiles, Japanese types show comparatively bigger loss of weight by freezing as compared with French or Spanish types, which are widely adopted in Japan. Loss of weight by freezing seen in some tiles and terra-cotta is too small to be measured.

Generally speaking, the faces of glazed tiles and the other similar ones, when they suffer frost action, exfoliate in the round or oval shape. Even some tiles appearently pure from freezing disclose under the microscope marked fissures on their glazed surface. And when the glazed face of a tile is magnified as large as thirty times, there air-cells are revealed, this inducing the freezing effect.

Fig. 17. Shows the cracks on the glazed wall tile before freezing test.

Fig. 18. Shows the cracks on the glazed wall tile after freezing test.

The fissures on the surface of glazed tiles come from the difference of moisture and thermal expansions of glaze and body, so no estimate can be formed indiscriminately with freezing only.

If tester examine loss of weight by freezing with water absorption combined, there will be seen a definite relation generally exsisting between materials belonging to a class.

Fig. 19. Specimen of concrete in frozen state.

A material, which has a relation far from the definite one, appears to suffer wonderfully intense frost action.

The freezing tests of concrete the author made were based upon field work ; the degree of freezing effect was examined by changing the curing of concrete and the following are one of the experimental results :—

(i) Cured in Water

A note with regard to the specimen.

A : Cured in water normally.

B : Twenty-four hours after a specimen had been made, it was taken off the mould. Freezing it in the freezing box, then it was cured in water.

C : The specimen made freeze after four weeks' curing in water.

Age 33 days ; mixing ratio 1 : 2 : 4 in volume ;
water cement ratio 65% ; slump 18.5 cm.
Form and dimension of specimen : cylinder, dia. 15 cm. and hight 30 cm. ;

No.	Weight of specimen in kg.	Compressive strength kg./sq. cm.	Mean
A_1	12.65	188.4	
A_2	12.75	183.3	188.0
A_3	12.70	192.4	
B_1	12.70	104.1	
B_2	12.70	156.2	137.1
B_3	12.65	151.1	
C_1	12.65	191.3	
C_2	12.75	209.4	199.0
C_3	12.70	196.4	

The result of series *B* shows the decrease of strength by 27% as compared with that of series *A*. The result of series *C*, as compared with that of series *A*, shows an increased strength. Such being the results, the author can safely observe that owing to freezing there will be no decrease of strength in series *C* and much decrease in series *B*.

(ii) Cured in the air

A note with regard to the specimen.

A′ : Cured in the laboratory basement.

B′ : Cured in the working room in the basement floor.

C′ : Cured outdoors.

For *A′* and *B′* there was not a day of temperature below 0° *C*. The lowest temperature for *C′* was -5° *C*. After the first one week every day nocturnal-temperature was below 0° *C*.
Age 29 days ; mixing ratio 1 : 2 : 4 in volume ;
water cement ratio 65% ; slump 18.4 cm.
Form and dimension of specimen : cube, edge length 15 cm. ;

No.	Weight of specimen in kg.	Compressive strength kg./sq. cm.	Mean
A'_1	4.925	213	219.0
A'_2	4.950	225	
B'_1	5.000	214	198.5
B'_2	4.925	183	
C'_1	4.850	195	167.0
C'_2	4.825	159	

The result of series *B′* shows the decreased strengh by 10% in comparison with that of series *A′*, and the result of series *C′*, as compared with that of series *A′*, shows decreased strength by 24%. From these results that freezing brings about the decrease of strength in the air curing is fairly judged.

In short, as to freezing after the hardening of concrete there is nothing worthy of mentioning, only during setting and in the early stage of hardening, concrete suffers frost action. The author thinks some day in the future there will be minute studies concerning frost damage to concrete.

5. Judging of Frost Action

The author measured water absorption of materials by water immersion, by under pressure, and by vacuum treatment, and tried to contribute to judging the frost resistance of materials from the relative relation brought about by the said measuring methods.

Measurement of Water Absorption

(1) Water Absorption by Water Immersion

The measurement of water absorption is to immerse a dried specimen in water for a certain limited lapse of time, and to measure the increased weight of the specimen after it absorbs water sufficiently. In general, the absorption by water immersion shows a bigger value in comparison with that of the absorption in moisture. As a matter of fact, so long as a material is not in water, the absorption in the air shows value less than that of the absorption by water immersion.

As the results of experiments, the author established the unit of 24 hours for the absorption measurement by water immersion for general materials, though such materials as roofing tiles require only one or two hours as to get the constant.

(2) Water Absorption by under Pressure

What we call water absorption by under pressure is this; a specimen is immersed in water, and hydraulic pressure is applied. By this method, however, the pores in the material are forced to be filled up with air in the compressed form. Therefore, when pressure is eliminated, the air therein expands, and it naturally drives out the water in the material to a certain degree. So, the results by this method are often indefinite owing to the state of pores in the material. In some cases, as high as 150 atmospheric pressure was applied,[9] but the author learned 5 atomospheric pressure was quite enough for the purpose.

Table 4. Comparison of Water Absorption.

No.	Material	Wt. of specimen after drying (gr.)	Volume of specimen (c.c.)	Absorption by water immersion (24 hrs): Absorbed water (gr) Wi	Absorption by water immersion (24 hrs): Filled up space with water (%)	Absorption by under pressure (5 atm): Absorbed water (gr.) Wp	Absorption by under pressure (5 atm): Filled up space with water (%)	Absorption by vacuum treatment: Absorbed water (gr.) Wv	Absorption by vacuum treatment: Filled up space with water (%)	$\frac{Wi}{Wv}$	Remarks
1	Marble	42.8	13.9	0.1	0.72	0.1	—	0.1	—	1.00	$Wi \doteqdot Wp \doteqdot Wv$
2	Granit	40.4	13.5	0.2	1.48	0.2	—	0.2	—	1.00	$Wi \doteqdot Wp \doteqdot Wv$
3	Andesite	35.8	13.7	1.3	9.49	1.6	11.68	2.5	18.25	0.52	
4	Tuff	23.4	13.5	4.7	34.81	5.1	37.78	5.7	42.22	0.825	
5	Sand Stone	30.5	13.0	2.5	19.23	2.9	22.31	3.7	28.46	0.676	
6	Wall Tile	16.7	6.3	1.0	15.87	1.3	20.63	1.8	28.57	0.556	
7	Brick	179.6	88.2	15.0	17.01	16.7	18.93	21.8	24.72	0.688	
8	Japanese Roofing Tile (Sanshu)	108.3	41.5	18.3	44.10	19.4	46.75	22.4	53.98	0.817	
9	" (Kyoto)	117.4	63.8	18.1	28.37	20.1	31.50	23.2	36.36	0.780	
10	" (Jigawara)	166.7	103.9	42.6	41.00	44.6	42.93	46.3	44.56	0.920	
11	1 : 3 Cement mortar	83.2	33.9	2.2	6.49	3.3	9.73	4.3	12.68	0.512	
12	1 : 2 : 4 Concrete	82.5	33.8	3.3	9.76	4.1	12.13	4.6	13.61	0.717	

Jigawara is the product near Tokyo.

(3) Water Absorption by Vacuum Treatment

A dried specimen is placed under vacuum state, all the air occupying the pores is got rid of, then water free from any gas in solution is run in to fill up pores with water.

Of the three methods mentioned above, this vacuum treatment is the most rational measuring method for water absorption ; almost all the crevices, cracks and pores are filled up with water. And as compared with the water absorption by under pressure and that by immersion, this method by vacuum treatment shows the biggest value. (Fig. 20, Table 4.) Therefore, the author wants to regard this value as the real void for judging freezing effect of a material.

In view of the indefinite absorption by under pressure, for judging frost-resistance the author disregarded the value obtainable by this pressure method, and calls the ratio of absorption by water immersion and that by vacuum treatment as ratio of absorptive void. The author judged

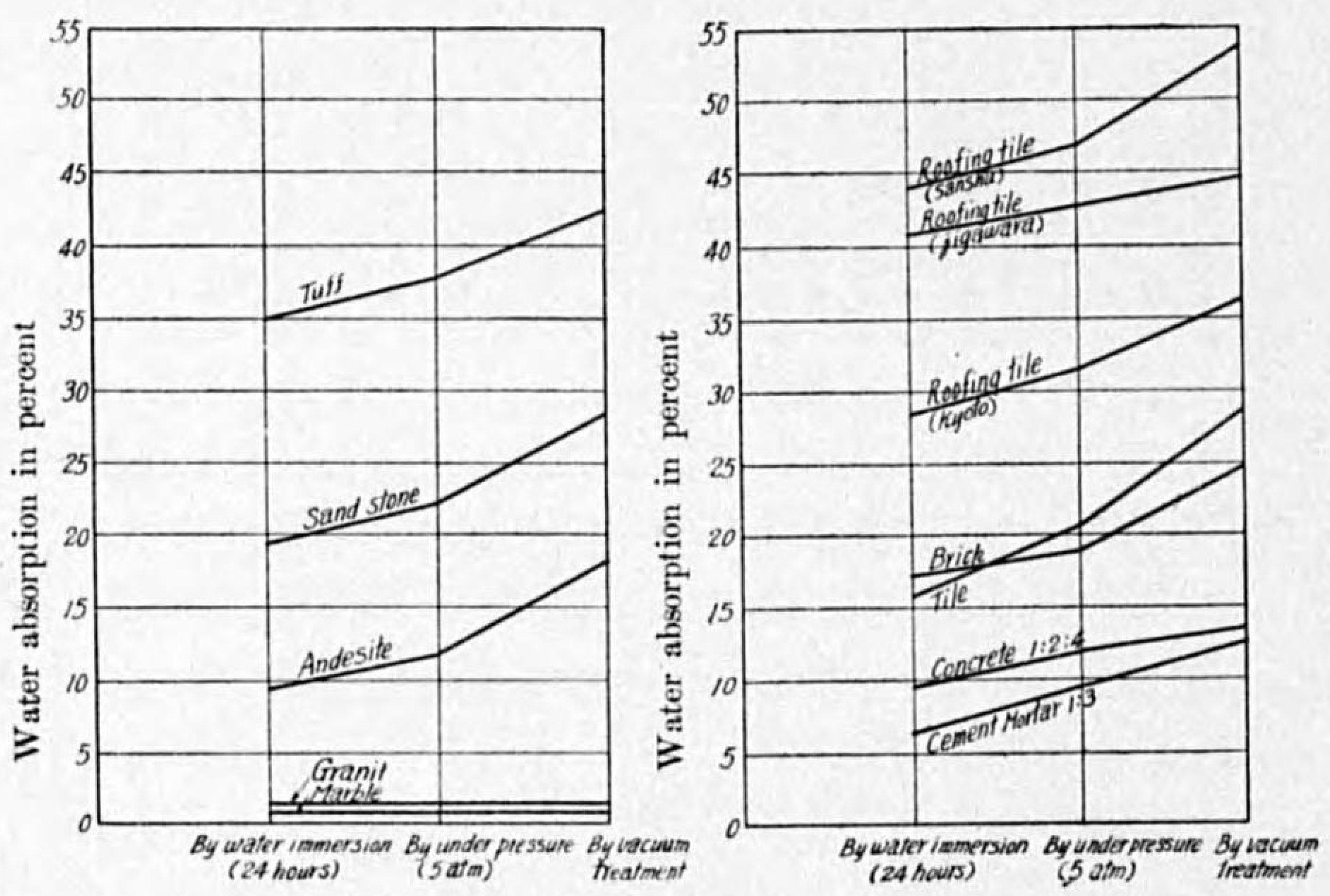


Fig. 20. Comparison of water absorption.

judged frost-resistance by the relation of said ratio and the absorption in volume percent by water immersion.

Considering the results of freezing tests as similar to actual freezing what are of higher ratio of absorptive void, that is, Sanshu roofing tiles or Jigawara (Japanese roofing tiles produced near Tokyo), and tuff suffer frost action intensely. It is worth noticing that these materials absorb much water, sometimes amounting to more than 30%. Also, marble and granit have higher ratio of absorptive void, on the contrary they absorb extreemly little quantity of water, so they suffer less freezing effect.

Viewed in this light, the author considers it important to observe ratio of absorptive void together with absorption in volume percent by water immersion. Here, the author proposes that he should name the product of these two factors as the frost action judging value, that is :—

$$K=\frac{Wi}{Wv}\times\frac{Wi'}{V}=\frac{WiWi'}{WvV}$$

K : frost action judging value, (Yoshida's proposal)

V : volume of the specimen in c. c.

Wi/Wv : ratio of absorptive void.

Wi : quantity of water absorbed by water immersion in gr.

Wi' : quantity of water absorbed by water immersion in c.c.

Wv : quantity of water absorbed by vacuum treatment in gr.

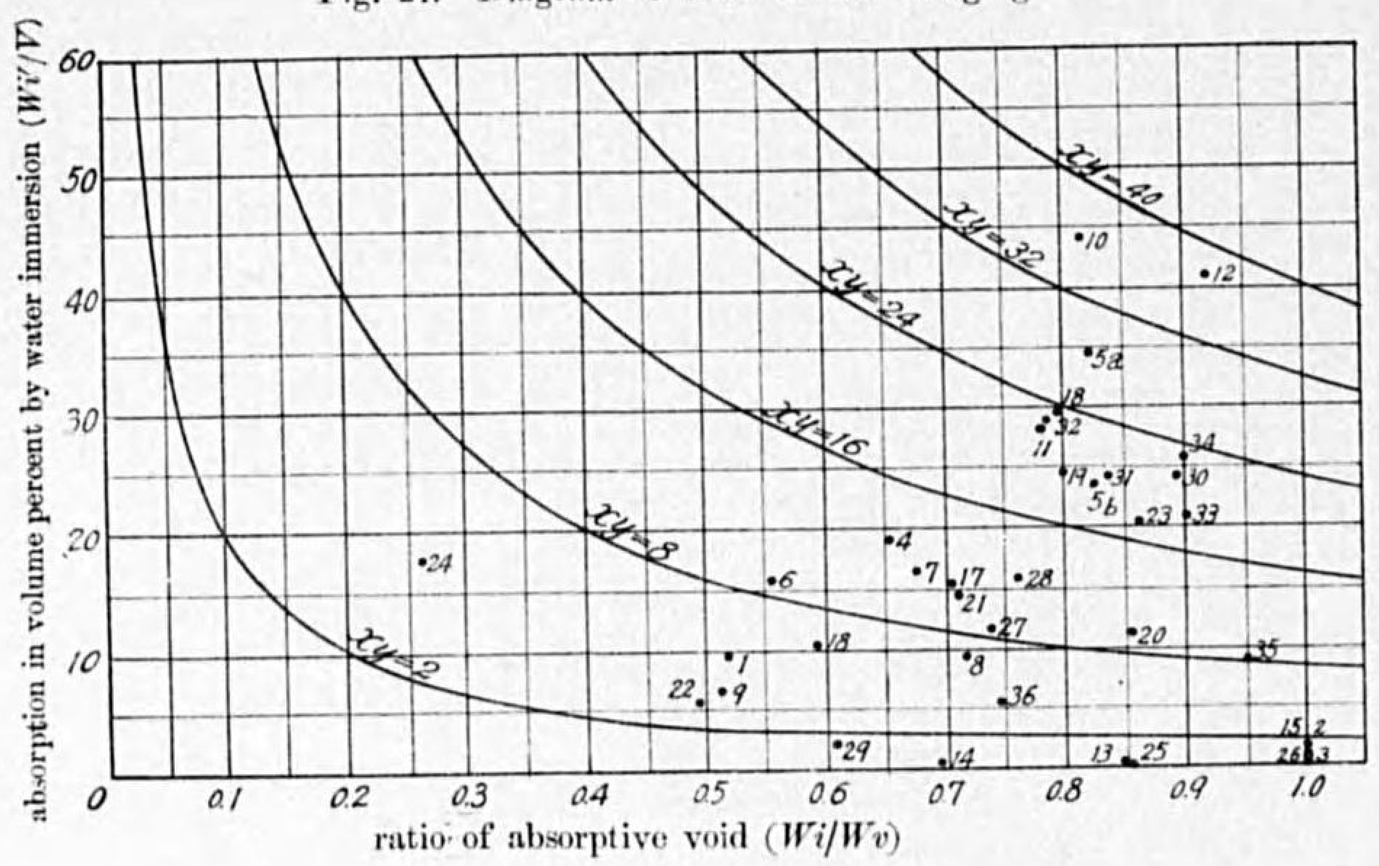
In this case, Wi (gr.) and Wi' (c.c.) can be considered as the same value.

So, $$K=\frac{WiWi'}{WvV}=\frac{Wi^2}{WvV}$$

Take ratio of absorptive void in abscissa and absorption in volume percent by water immersion in ordinate, and the value of K will be expressed in hyperbolas. By giving a lot of value to K, series of hyperbolas will be obtained. (Fig. 21). Plotting the value K on the diagram, the degree of freezing effect shall be judged. The less is the value of K, the more is the frost resistance.

The author examined the possibility of the application of frost action judging value plotted in the diagram by comparing the freezing tests nearer to natural freezing with the author's frost-action judging value. And the

Fig. 21. Diagram of Frost Action Judging.



1. Andesite
2. Granit
3. Marble
4. Sand stone
5*a*, 5*b*. Tuff
6. Tile
7. Brick
8. Concrete 1:2:4
9. Cement mortar 1 : 3
10. Roofing tile (Sanshu)
11. „ (Kyoto)
12. „ (Jigawara)
13. Granit (Mannari)
14. „ (Mikage)
15. „ (Inada-ishi)
16. Sand stone (Tako-ishi)
17. Sand stone
18. Tafaceous Sand stone (Boshu-ishi)
19. Tuff (Ōya-ishi)
20. Tuff (Shikishima-ishi)
21. Sand stone (Hinode-ishi)
22. Andesite (Yokonezawa)
23. Marble
24. Pumice stone (Kōka-seki)
25. Dolomite (Kansui)
26. Marble
27. Tile A
28. „ B
29. „ C
30. „ D
31. Roofing tile (Kyoto)
32. Brick
33. 35% neat Cement
34. 40% neat Cement
35. 40% 1:2 Cement mortar
36. Concrete

author has found that the author's value, so far as the author's experiment goes, can perfectly be applied.

6. Conclusion

In order to keep building material safe from frost action the causes of damage must be examined, on the one hand the frost action examples, and the freezing tests should be made to see that there is no mistake, and on the other, experts ought to take good care of construction and execution.

It is needless to remark that the test results play an important parts in judging the degree of frost action, the test results being often different from those produced by actual frost action.

Care must be taken of a material, whose absorption is big, because in general such a material is much subject to freezing effect. While a material having less absorption ratio and showing better test results is alarmingly subject to frost action owing to the free-water frozen in the partial fissures.

This being the reason, experts have to be careful not only of selecting the sorts of material, but of classifying it too individually. Some experts give narrow limits to the studies of freezing tests of materials and to judging it ; no building material discloses damage through one time freezing, but it gives way to freezing effect by some repetition both of freezing and thawing. The degree of frost action differs according to the formation of pores in a material, and the state of water absorption.

The author has advanced a proposal that the exact degree of freezing effect should be measured and judged by a diagram showing the relation of the ratio of absorptive void and the absorption in volume percent by water immersion, that is a frost action judging value.

Though the materials safe from frost action are selected by the tests needed, but if experts are not careful of construction, execution, and of applying the materials, none the less they suffer frost action. In short, there will be an intense stress when water freezes, therefore in construction we must leave so much margin as to prevent, as far as can be, materials from absorbing water, and to eliminate resistance to stress, namely ; (1) not to make materials and parts of a structure absorb water ; (2) to leave the most margin of space possible not related to water absorption ; (3) to facilitate the running-out of absorbed water and moisture ; (4) not to make capillary attraction work, which will accelerate water absorption.

References

1. E. R. Buckley : Building and Ornamental Stones. (1898)
2. J. E. Marsh : Stone decay and its preservation. (1926)
3. R. J. Schaffer : The weathering of natural building stone. (1932)
4. H. F. McMurdie : Results of freezing and thawing tests on sand-lime brick. Rock Products, Vol. 34, No. 12, (1931)
5. Brard : Annales de Chimie et de Physique. Vol. 38, p. 160, (1829)
6. W. A. McIntyre : Investigation into the durability of terra-cotta and faience. (1930)
7. M. Temin, W. Pigman and J. Tucker : The selection of durable aggregate for concrete. Rock Products. Vol. 34, No. 16, (1931)
8. J. Bauschinger : Investigation of the resistance to frost of natural and artificial building stones. Mitteilung Nr. 23. des Mechanische-Technischen Laboratoriums des Köngl. Technischen Hochschule in München. (1889)
   J. Schaffer : The weathering of natural building stone. (1932)
9. J. Hirschwald : Handbuch der bautechnischen Gesteinprüfung. (1912)
10. H. Kreüger : Utredning rorande klimatisk inveskan pa byggnadsfader, Ingeniörs Vetenskaps Akademien No. 24. (1924)
11. Heinrich Seipp : Die abgekürzte Wetterbeständigkeitsprobe der natürlichen Baustein. (1905)
12. J. W. McBurney : The water absorption and penetrability of brick. Proc. A.S.T.M. Part II. 1929.
13. L. A. Palmer : Some absorption properties of clay brick. Bureau of Standards, Research Paper No. 88, (1929)
14. R. E. Stradling : Effects of moisture changes on building materials. (1928)

Advantage of Hollow Draft Wall, Against Wood-Decay, Overheating, and Damp.

BY

Assist. Prof. **Saburō Soshiroda.** *Kōgakushi.*

Consequent upon the catastrophe in the Kwanto districts in 1923, the number of wood-frame hollow-wall building built only externally after the mode of reinforced concrete ones has been on the increase.

The cost of building of this kind being cheap in general, the number of them will surely increase in future.

As the external design of this kind of buildings is imitative of concrete construction, the structural spaces (under the floor, within the hollow-walls, and the roof) is entirely enclosed and in consequence this kind of an enclosed hollow wall building discloses two big defects, that is, Decay of wood frames and Overheating of rooms in summer. The important studies of this construction have, however, hitherto been overlooked. (See Fig. 1, 2.)

Laying stress upon these defects Mr. Kawamura, doctor of science, and Mr. Fujii, doctor of engineering, have

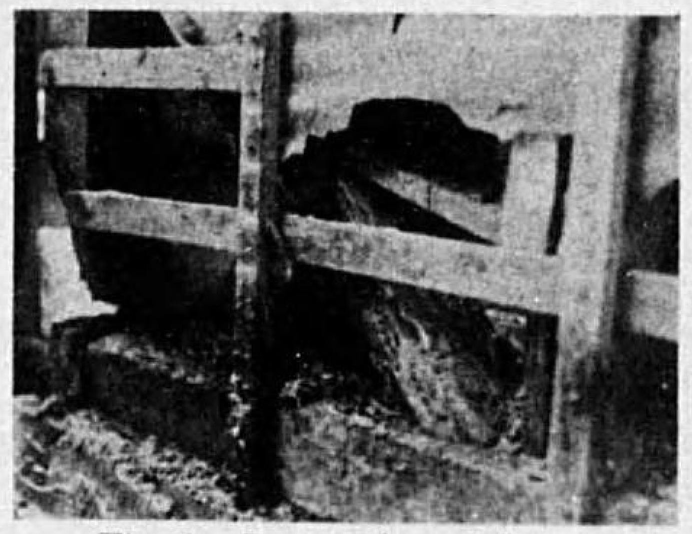


Fig. 1. Decay of wood frames of a school building.

Fig. 2. Decay of wood frames of a villa,

authoritatively mentioned in their respective essays (in which Dr. Kawamura dealing a subject of Decay of Wood-frames, Dr. Fuiji a subject of Over-heating in rooms) that in a land like Japan where temperature and humidity are higher in the summer time no building of this kind is positively recommended.

Notwithstanding the warning announcement made by these scholars no perceptive improvement has been contemplated.

Now, the essays of these doctors gave the author an impetus to taking up 'Hollow Draft Wall' in the present paper. The following is the system of a hollow draft wall construction: air is lead in the walls from the inlets either at under the floor or at upper the openings, and is exhausted from the outlet in the space between the roof and ceiling.

In this way the author attempted the simultaneous filling-up of the aforesaid two defects.

Wood-decay is ascribable in general to the dry rot or domestic fungi fecundated in the insufficient air ventilated spaces under the floor and within the hollow walls.

The best condition that fecundates domestic fungi in the enclosed places, is more than 80% constant humidity and about 20–30°C. constant temperature.

When outside air is sufficiently lead in the hollow walls, the air condition under the floor and within the walls will be similar to that of the outside. Then, it will serve the purpose of the wood-decay prevention.

Overheating in a room is due to the higher temperature caused by air convection in an enclosed hollow wall exposed direct to the sun in summer.

While after sunset when open air is gradually cooled, the temperature within the hollow walls does not go down soon, therefore the temperature in the room will remain longer rather hot.

Now, the hollow drafted system: when open air is lead in the floor and the walls, the difference of the temperature within the walls and abroad creates air circulation in the walls, and then the air within the wall is gradually exhausted.

Thus, after sunset the temperature both within the walls and in the room will soon go down.

During the winter season the inlets under the floor and the vents of the roof are closed, cutting off open air, and preventing both the inside of the wall and room from growing cold. In this season, both temperature and humidity are lower, therefore, the cutting-off of open air will bring about no fear of accelerating wood-decay.

To obtain the expected possibility and effective proof of this plan of the author's, two-houses for experiment were built in June, 1931, in the precincts of Waseda University, each covering an area of 3 metres square. And in these houses comparative experiments were made. (See Fig. 3, 4.)

Fig. 3. The experiment house of ordinally Construction.

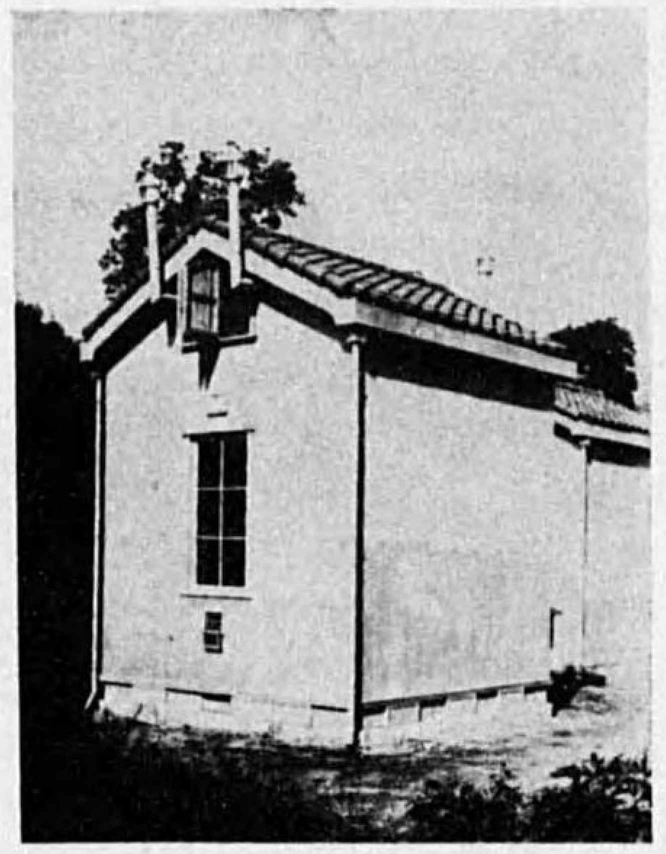


Fig. 4. The experiment house of Hollow Draft wall Construction.

For convenience's sake each of the temporary houses were built on the same condition, frame structure, ceiling hight, (3 metres) wall finish and openings, with each wall facing to four directions right.

Only one exception is the ventilation system: one house has ordinary enclosed (non-ventilated) walls, the other hollow draft walls.

The self-recording thermometres, the psychrometres, the dry-and-wet bulb thermometres, and the 1/10 thermometres were deposited in the room, in the open air in the shade, under the floor, in the space between the roof and the ceiling, and in the four hollow walls.

Temperature and humidity were measured in these places respectively, on condition that openings should be closed at a time and opened at another.

The air current in the inside of the walls was measured by the anemometer. The following is the synopsis of the results obtained from the experiments: A is a Hollow Draft Wall Construction, B an Ordinary Enclosed Hollow Wall, and Fig. 1–8 show the experimental results obtained in each season except winter, and 9–11 show those in winter.

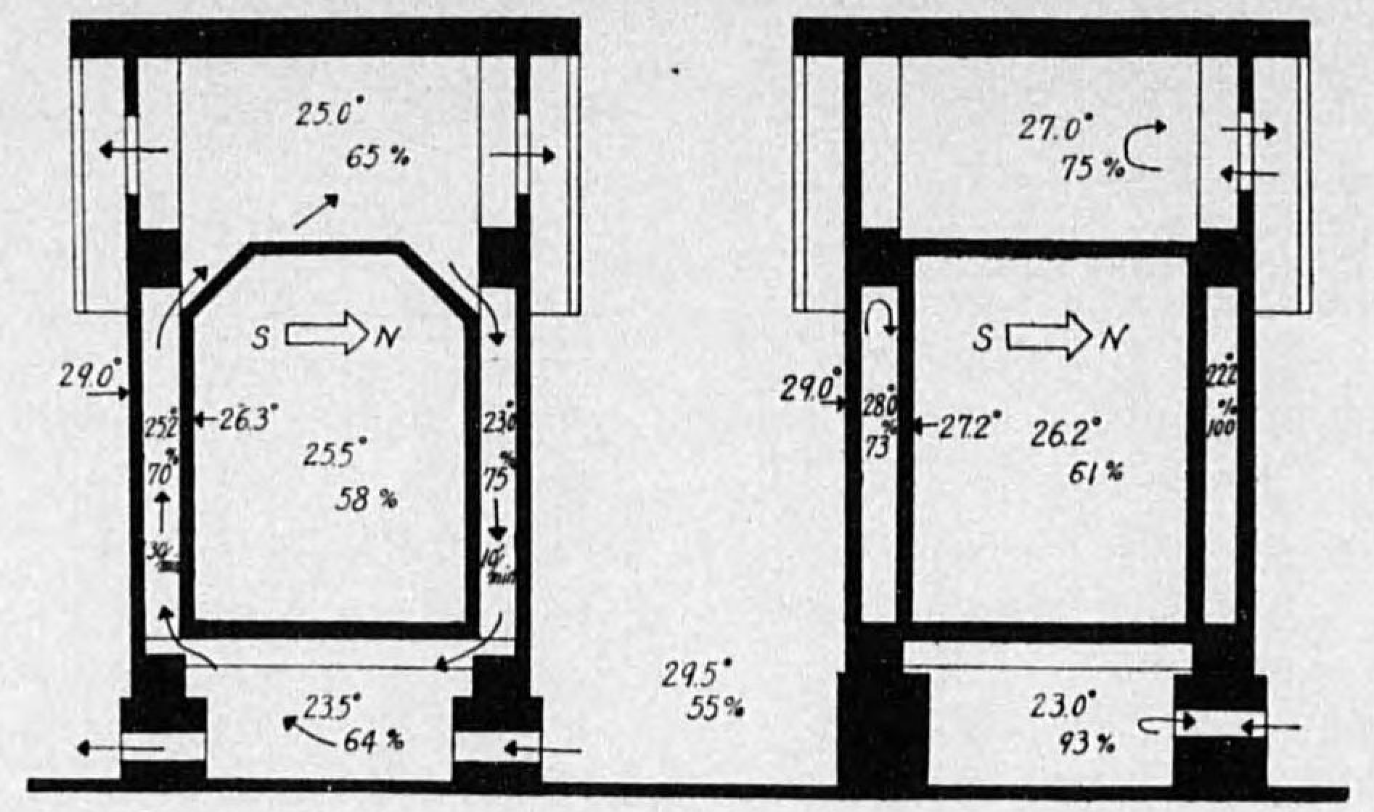
(1) The Ordinary Enclosed Hollow Wall is, prima facia, impervious to heat and humidity, yet as it was enclosed, contrary to expectation, both temperature and humidity go up higher.

While in the case of the Hollow Draft Wall, when exposed direct to the sun, the temperature in the wall goes up, and in consequence, the air current in it rises up and the humidity goes down.

And a big current is produced owing to the air convection circulating all around the room through the inside spaces of the roof, the opposite wall and under the floor, even in the parts not exposed to the sun. (See Fig. 5) This causes the temperature and humidity at all the places mentioned above to go down, and afforded it few opportunities to destroy wood. Here, it is needless to mention that the temperature of the room goes down.

The Fig. 5 shows the comparative experiments made in A and B houses at 1 p.m. on the 8th of September: in the case of B, the part of the floor inner-part of the walls, back-side of the roof were all enclosed, convection worked only in the inner part of the southern wall exposed to the sun, the temperature of the inside of the wall reaching 28° *C.*

Now A house experiment: As the arrow figure indicates, the aforesaid big current circulated around the room through the inner space,



"A" House "B" House

Fig. 5. The comparative experiments made in A and B houses. (11 a.m. 8th Sep.)

then the temperature there did not rise up. And the humidity at each place was lower than that of B. However, the temperature under the floor was slightly higher than that of B, which was due to the radiation of the sun and circulating air.

(2) The temperature under the floor of A is a little higher than that of B, when the weather is fine, but the humidity of A is by far lower. Fig. 6 shows the result of the measurement on the 8th of September (fine): the temperature was rather higher during the daytime, and yet the humidity of A was lower, and the maximum difference was 35%.

(3) The temperature of A under the floor, when it is cloudy and rainy, is higher than that of B, but the humidity of A is lower. Fig. 7 shows the result on the 10th of September (cloudy and rainy): the temperature of A was higher, but the humidity lower. The maximum difference between the two was 17%.

(4) The humidity of A within the walls, especially in the wall

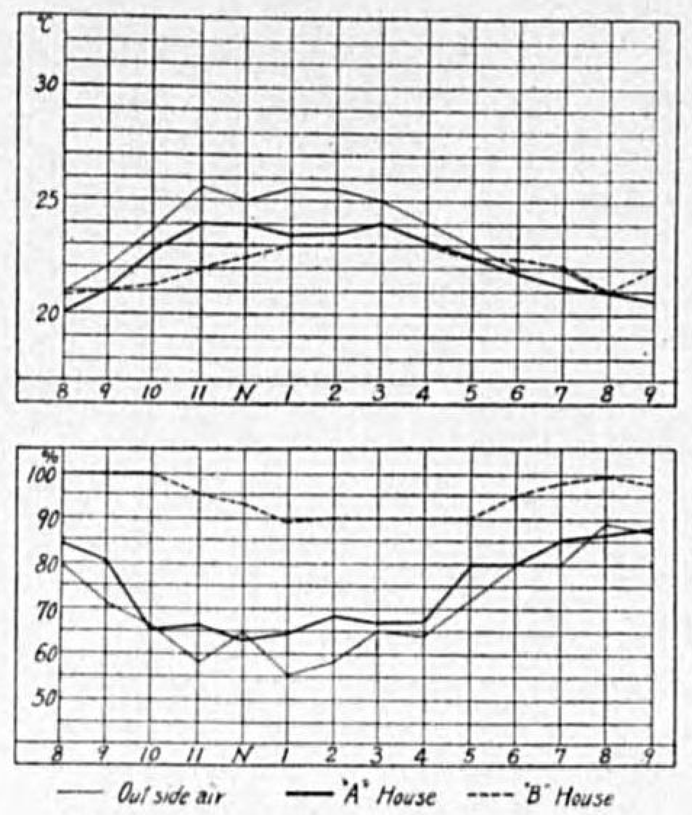


Fig. 6. The comparative results of temperature and humidity under the floors of A and B houses. (8th. Sep-fine)

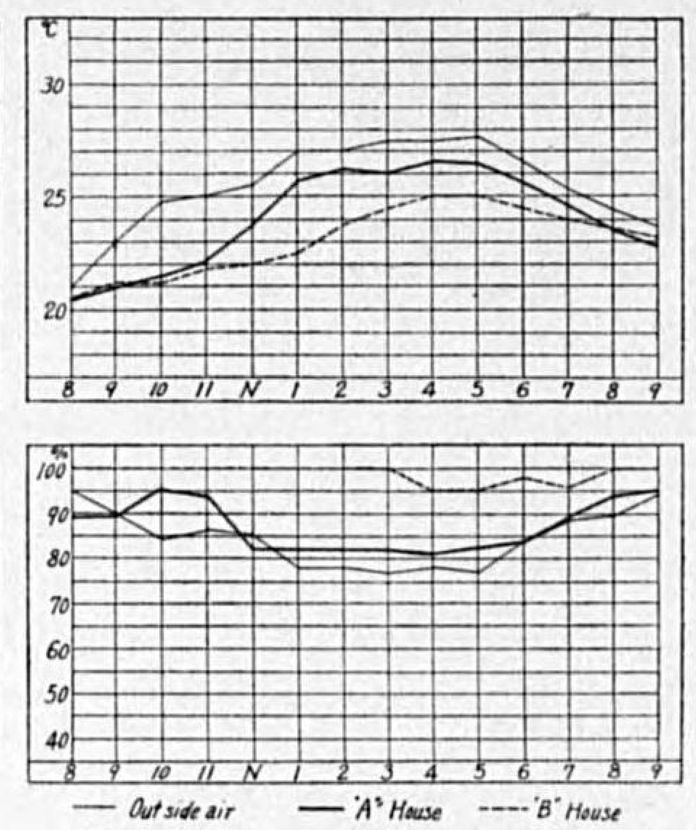


Fig. 5. The comparative results of temperature and humidity under the floors of A and B houses. (10th. Sep-cloudy and rainy)

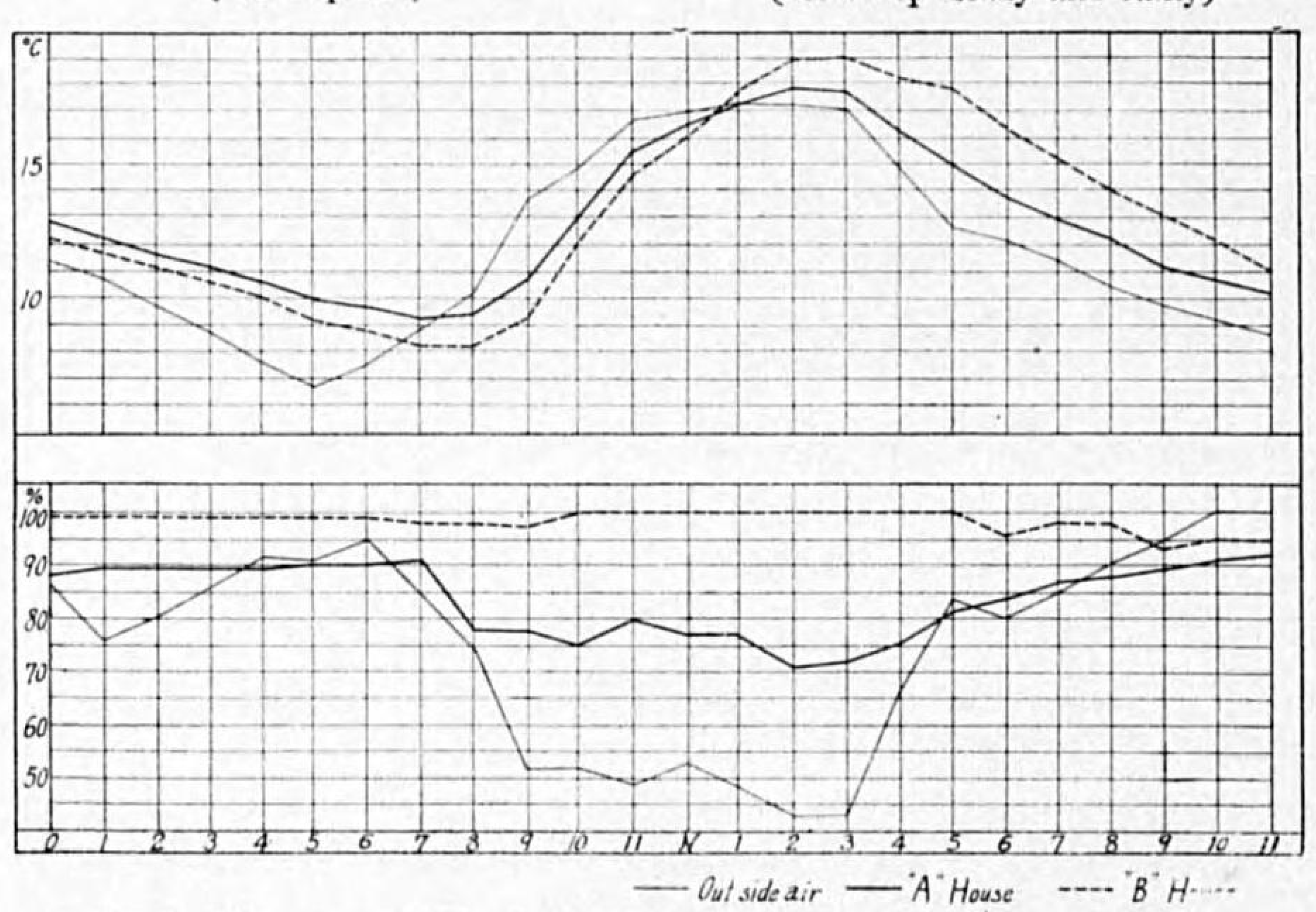


Fig. 8. The comparative results of temperature and humidity within the north walls of A and B houses (20th. Oct-fine)

facing the north (where humidity is usually higher as might be supposed), is by far lower than that of B. Fig. 8 shows the result on the 20th of October (fine): the temperature of A had been slightly higher from midnight till noon, yet in the afternoon that of A was lower.

The humidity of B was almost 100% (saturation condition), that of A being quite low. The maximum difference was 30%.

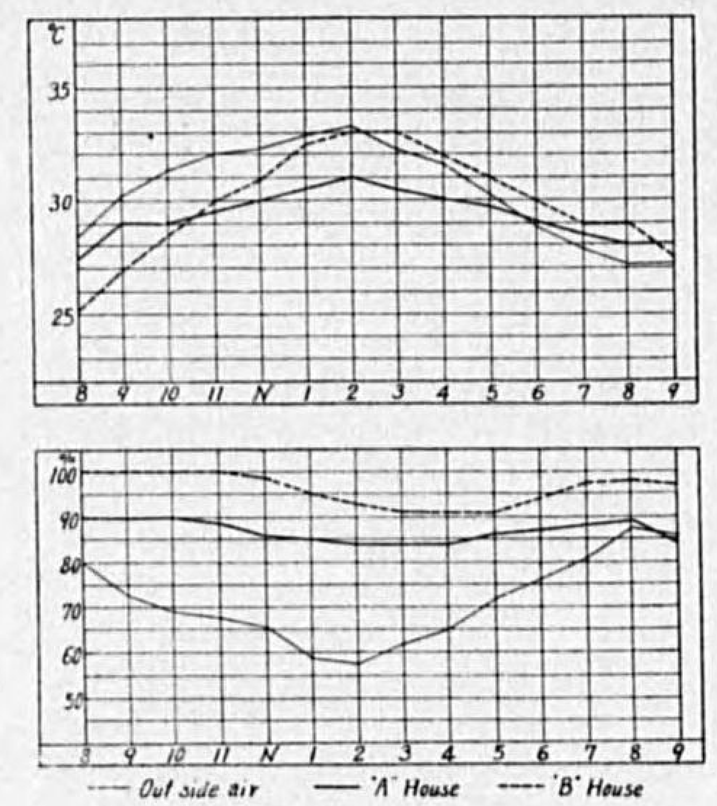


Fig. 9. The comparative results of temperature and humidity within the south walls of A and B houses. (8th. August-fine)

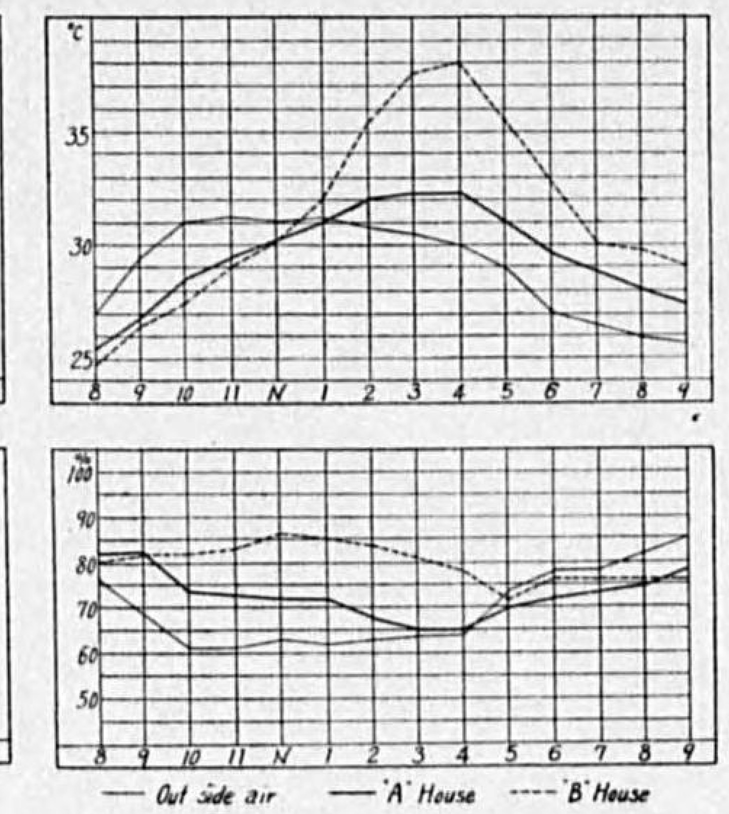


Fig. 10. The comparative results of temperature and humidity within the west walls of A and B houses. (5th. Sept.-fine)

(5) Both the temperature and humidity of A within the south wall are lower than those of B. Fig. 9 shows the result on the 8th of August (fine): the temperature of A during the daytime was lower than that of B, while the humidity of A was constantly lower.

(6) Both the temperature and humidity of A within the west wall are by far lower than those of B. Fig. 10 shows the result on the 5th of September (fine): the temperature of A was by far lower, the maximum difference being 6° *C*. And also the humidity of A was constantly lower, the maximum difference being 15%.

(7) Both the temperature and humidity of A in the room, when it

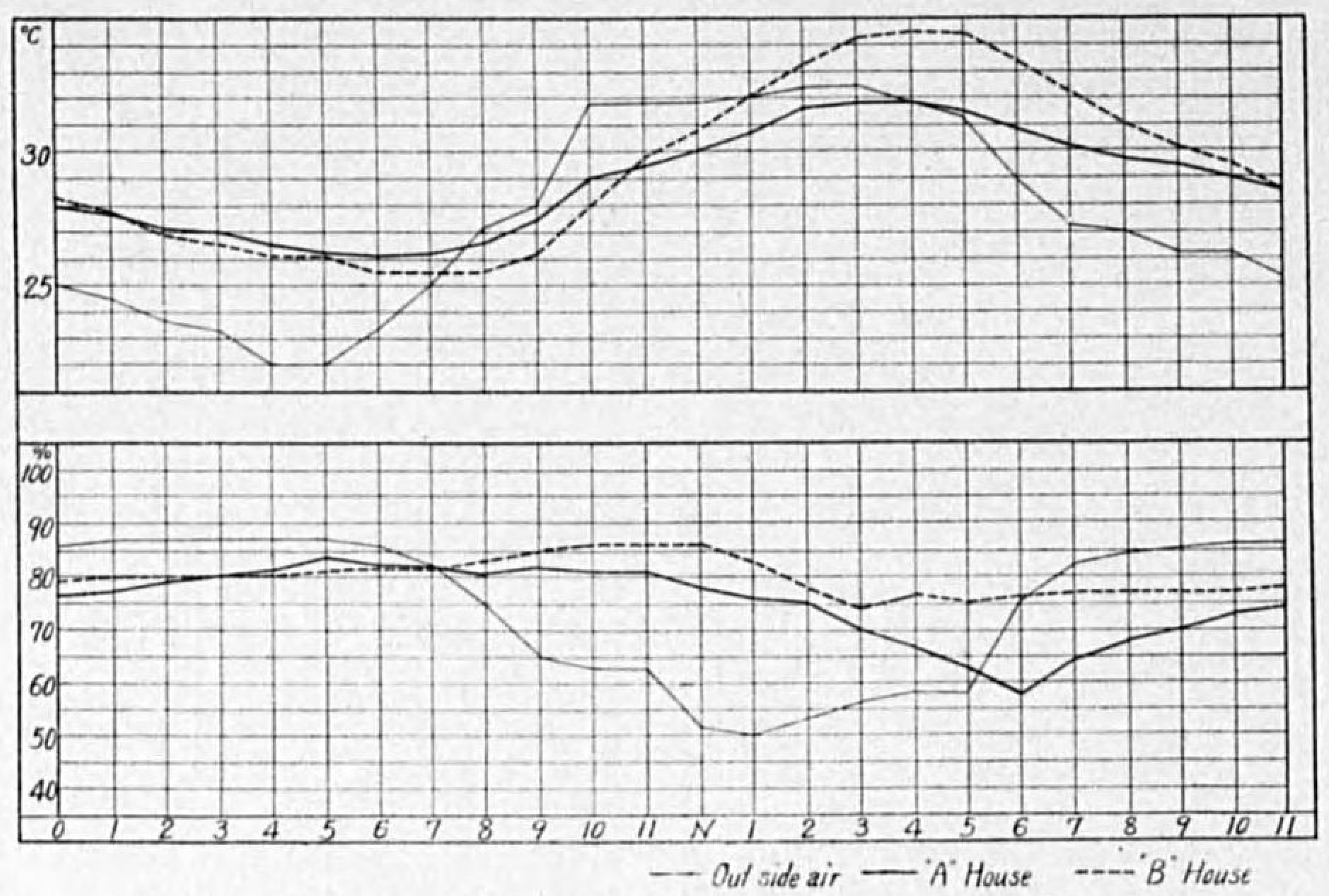


Fig. 11. The comparative results of temperature and humidity in the rooms of A and B houses. (22nd. August-fine)

is fine, are lower. Fig. 11 shows the result on the 22nd of August (fine): the temperature and humidity of A had been a little higher in the morning,but in the daytime they were much lower: the maximum difference was 20%. in humidity, 3° C in temperature.

(8) Both the temperature and humidity in the space between the ceiling and roof of A during the daytime of fine weather are lower. Fig. 12 shows the result on the 5th of September (fine): the temperature of A was higher in the morning, but during the rest

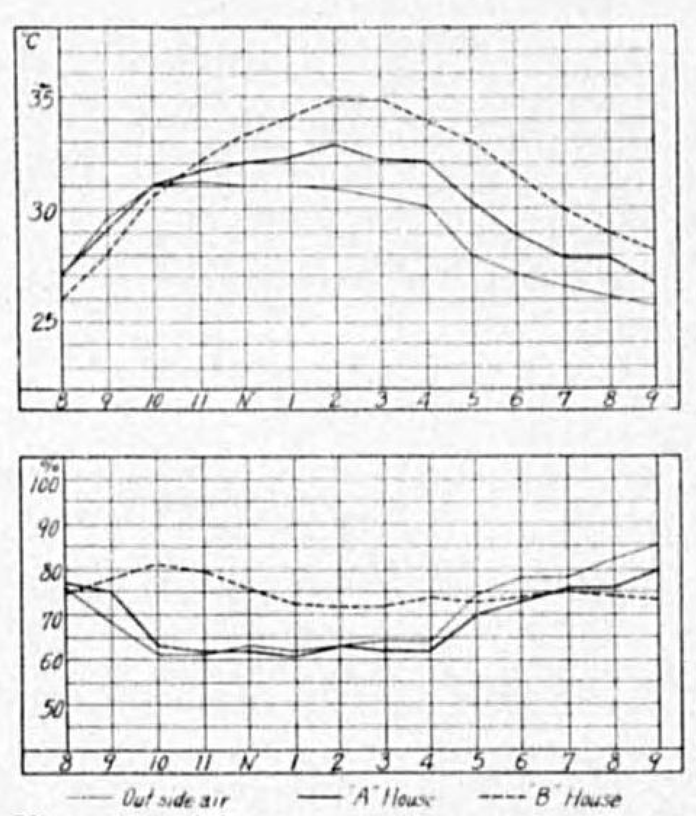


Fig. 12. The comparative results of tempera ture and humidity in the space between the ceilings and roofs of A and B houses. (5th. of Sept.-fine)

of the day that of A had been constantly lower. The humidity of A was lower in the daytime.

(9) In winter when the inlets under the floor and the outlets of the roof space were closed (what are mentioned below are on the same condition as this), and no heating systems were applied, the temperature of A was rather higher. Fig. 13 shows the result on the 22nd of December (fine): the temperature of A was rather higher.

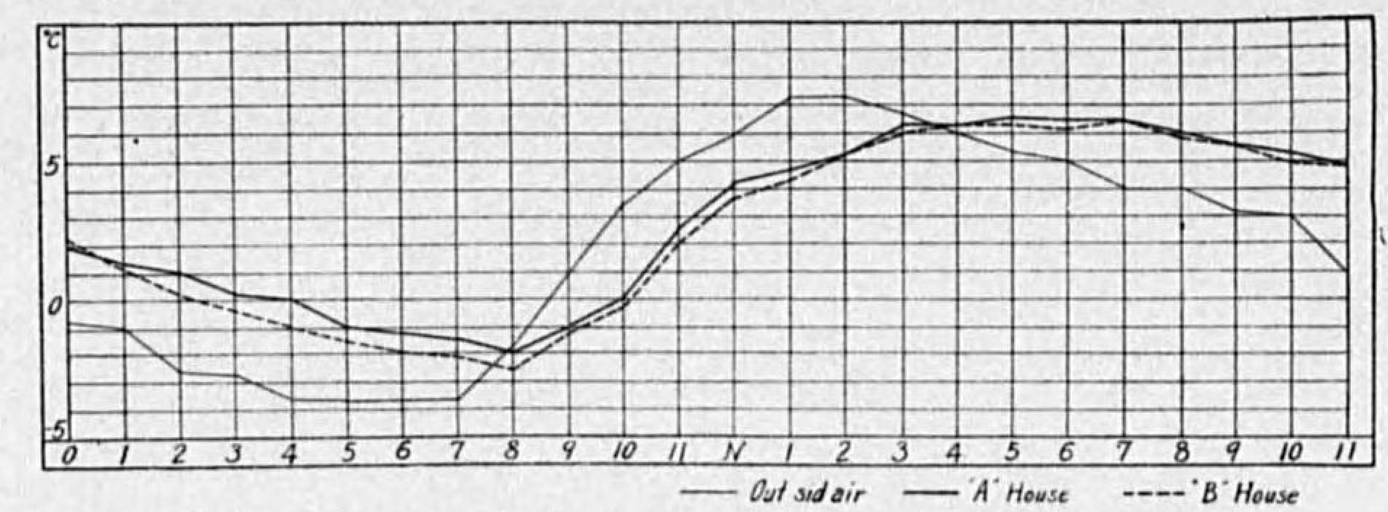


Fig. 13 The comparative results of temperature in the rooms of A and B houses. (22nd Dec.-fine. Non-heating)

(10) When 1 K. W. electric heat is given the temperature of A is higher in the morning, but lower in the afternoon. Fig. 14 shows the result on the 23rd of February (fine): from 9. a.m. till 2 p.m. the temperature of A was higher, the maximum

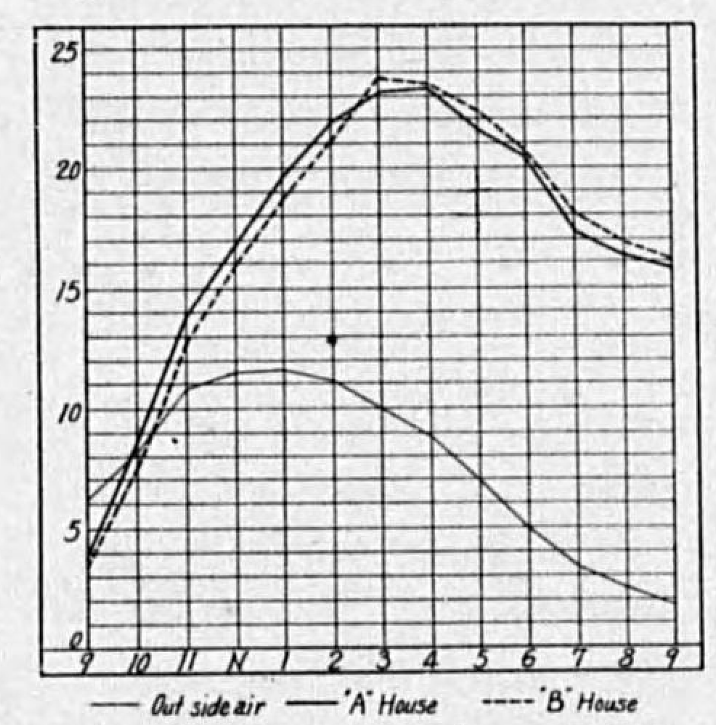


Fig. 14. The comparative results of temperature and humidity in the rooms of A and B houses. (22nd Dec.-fine. Non-heating)

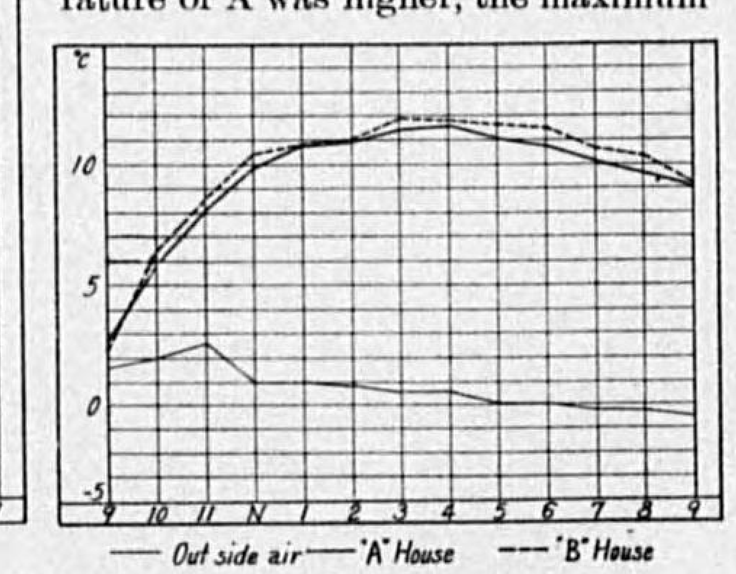


Fig. 15. The comparative results of temperature and humidity in the rooms of A and B houses. (25th Feb.-fine. heating)

difference being about 1° C.

(11) 1 K.W. electric heat is given, when it is snowy, the temperature of A is rather lower. Fig. 15 shows the result on the 25th of February (snowy): the temperature of A was lower all the day through, the maximum difference being about 0.6° C.

Recent Development of Architectural Acoustics in Waseda University.

BY

Assist. Prof. **Takeo Satow.** *Kōgakushi.*
Inst. **Sadao Iketani.** *Kōgakushi.*

1. Introduction.

For the past several years the study of architectural acoustics has been made in the Architecture Department of Waseda University, and we introduce some important results obtained in our recent studies.

2. Some Development of Reverberation Theory.

The process of sound growth, saturate and decay phenomena in a closed space, where a continuous sound source was conducted in, was discussed by W. C. Sabine, Jäger, Franklin, and many other authorities, while we have studied the said process by the growing, decaying and interrupted sound source in a room and have obtained the following formulae.(1)

(a) By the sound source $P(1-e^{-\beta t})$ and $Pe^{-\beta t}$

Growth, $$E_g=E_0(1-e^{-\mu t})+\frac{P}{V\beta-\frac{c\alpha S}{4}}(e^{-\beta t}-e^{-\mu t})$$

Satulate Value, $$E_0=\frac{4P}{cA}$$

Decay, $$E_d=E_0\frac{1}{(\beta-\mu)}(\beta e^{-\mu t}-\mu e^{-\beta t})$$

where μ is the damping coefficient of a room.

(b) By the sound source interrupted.

Let the sounded time t_1 and stopped interval t_2

Growth,

at $t=(t_1+t_2)2n$

$$E_g=E_0(1-e^{-\mu t_1}+e^{-\mu(t_1+t_2)}-\ldots\ldots-e^{-\mu\{(n-1)t_1+nt_2\}})e^{-\mu t_1}$$

at $t=(n+1)t_1+nt_2$

$$E_g=E_0(1-e^{-\mu t_1}+e^{-\mu(t_1+t_2)}-\ldots\ldots-e^{-\mu\{(n+1)t_1+nt_2\}})$$

Saturate value, $E_0=\dfrac{4P}{cA}$

Decay,

at $t=(n+1)t_1+nt_2$

$$E_d=\frac{E_0}{1-e^{-\mu(t_1+t_2)}}\cdot[e^{-\mu\{(n+2)t_1+(2n+1)t_2\}}-e^{-\mu\{(n+1)t_1+(n+1)t_2\}}]$$

at $t=2n(t_1+t_2)$

$$E_d=\frac{E_0}{1-e^{-\mu(t_1+t_2)}}[e^{-\mu\{(n+1)t_1+(n+1)t_2\}}-e^{-\mu\{nt_1+(n+1)t_2\}}]$$

The above formulae are based upon a viewpoint of the continuous change of sound energy density with respect to time, but more microscopically(2) the amount of sound energy accumulated by the great numbers of reflections (we call as average intensity of *virgin sound*, and it is denoted by $\varepsilon_0=\dfrac{Pp}{Vc}$) change in accordance with every p/c sec. interval, and this process can be considered as discontinuous regarding short interval of time. From that viewpoint we get the following formulae

$$E_g=\frac{\varepsilon_0}{\alpha}[1-(1-\alpha)^n] \quad \text{or} \quad \frac{4P}{cA}(1-e^{\frac{cS\log_e(1-\alpha)}{4V}t})$$

Also, according to each case,

(a) Source worked during t_1 time.

$$E_d=\frac{\varepsilon_0}{\alpha}[1-(1-\alpha)^{\frac{cS}{4V}t_1}](1-\alpha)^{\frac{cS}{4V}t_2}$$

(b) Source lasting $t=p/c$ sec.

$$E_d=\frac{\varepsilon_0}{\alpha}\alpha[1-\alpha]^{\frac{cS}{4V}t_2} \quad \text{or} \quad \varepsilon_0(1-\alpha)^{\frac{cS}{4V}t_2}$$

(c) Source lasting infinitely long.

$$E_d=\frac{\varepsilon_0}{\alpha}[1-\alpha]^{\frac{cS}{4V}t_2}$$

To get the Reverberation time T', according to the above cases,

$$T'=\frac{-4V\log_e\frac{\varepsilon_0}{\varepsilon\alpha}}{cS\log_e(1-\alpha)}-\frac{4V\log_e[1-(1-\alpha)^{\frac{cS}{4V}t_1}]}{cS\log_e(1-\alpha)}.$$

$$T'=\frac{-4V\log_e\frac{\varepsilon_0}{\varepsilon}}{cS\log_e(1-\alpha)}. \qquad T'=\frac{-4V\log\frac{\varepsilon_0}{\varepsilon\alpha}}{cS\log_e(1-\alpha)}.$$

To get the Sabine's time, let $E_0=\varepsilon\times10^6$

$$T'=\frac{0.05V}{-S\log_e(1-\alpha)} \quad \text{(Foot system)}$$

Then the reverberation formula by a certain sound source P

$$T_s=\frac{4V}{-cS\log_{10}(1-\alpha)}\left(\log_{10}\frac{4P}{c}-\log_{10}A\right)-\frac{4V\log_{10}[1-(1-\alpha)^{\frac{cS}{4V}t_1}]}{cS\log_{10}(1-a)}.$$

3. Some Corrections of The Transmission Measurement Theory of Building Materials.(3)

With regard to the measurement of sound transmission of building materials, a variety of method is presented. We have made several theoritical corrections for these measurement methods.

(a) At the Bureau of Standard, the total absorbing power of sound chamber change from α to α' by introducing the test piece, and from this results the Bureau has defined $k\dfrac{\alpha}{\alpha'}$ (where k=transmission coefficient of material) as a reduction factor. Of course, if it is aproximately $\alpha\fallingdotseq\alpha'$, there will be no room for discussion, but strictly speaking, some correction must be necessary as is mentioned above.

(b) At the River Bank Laboratories they get

$$10.\log_{10}\frac{\alpha_2S_2}{kW}=10\mu(t_1-t_2)$$

In this equation $\log_{10}\dfrac{\alpha_2S_2}{kW}$ of left side is called by the Laboratories as reduction factor, however, it must be corrected and

$$10.\log_{10}\frac{1}{k}=10\mu(t_1-t_2)-\log_{10}\frac{\alpha_2S_2}{W}$$

adopted.

4. Some Basis for Acoustical Design of Auditorium.

To the acoustical design of auditorium reverberation theory is essential, but some consideration on the form factor can never be neglected. In that viewpoint, at an auditorium for speech, reverberation is rather an impediment to catching a speech. (e. i. the decrease of percentage articuration). We proposed that the perfect reception of speech ought to depend rather on some of direct sound and first reflected sound (at least inclusive of the second reflected) than upon reverberation. The following two methods are proposed for model studies to see if the sound reflector is radically situated.

(a) Smoke Box Method.[4]

The path of sound reflection from a plane wall can easily be detected graphically, yet in case of curved surfaces it is rather difficult to find out the path. We proposed the two dimentional model study as follows.

An auditorium model section made of metalic mirror is placed in a box full of smoke, and a point-light source is deposited at a speakers position. By so doing, the direction of sound reflected can easily be detected, and also the surface which concentrates the sound on a certain place is found by introducing special pie-diagram form lens.

(b) Photo Electric Cell Method.[5]

To get the amount of sound quantities in any point of an auditorium, the point-light source is also placed in the above mentioned aparatus, then sliding the photo electric cell along the seat line, we can get the relative intensity of sound in each point reading the relative intensity of electric current by the galvanometer.

5. Noise Measurement.

The noise problem in Tokyo is vital for the health of townsfolk. To measure the change of noise both in time and space, we used $3-A$ and $2-A$ audiometer of Western Electric Co. These results rendered valuable contribution to the cause of the sound proof construction.

Fig. 1. shows the 24 hour change of noise in Shinjiku-Streets measured by $2-A$ audiometer.[6]

In 1930, in the Tokyo subway, we measured the noise in all its stations and in a running car and got the following empirical formula for noise v. s. velocity of the car.[7]

$$Y_1 - Y_2 = 15.4 \log_{10} \frac{X_2}{X_1}$$

where X=vel. of car.

Y=noise level in db.

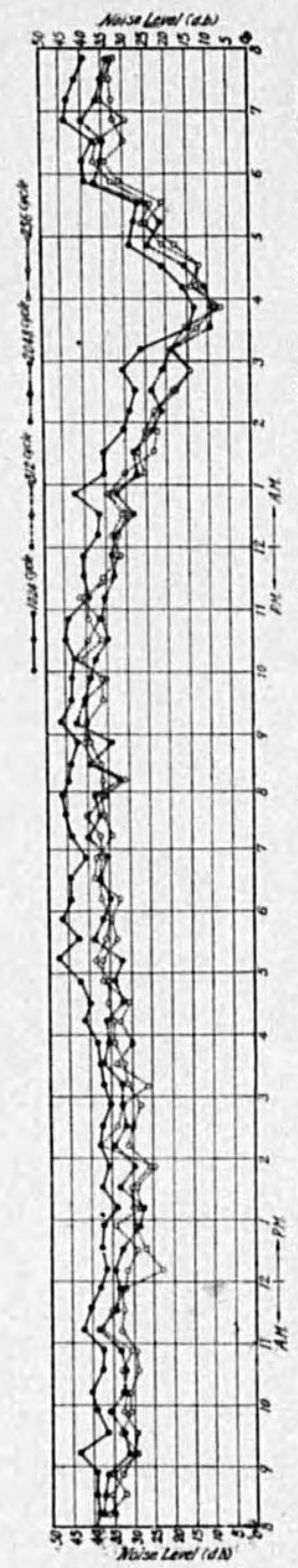


Fig. 1. Change of Street Noise during 24 hour (Shinjiku street Tokyo) From 24th to 25th Aug. 1930.

6. New Method for Measurement of Sound Absorption Coefficient of Building Materials: Reverberation Masking Method.[8]

Reverberation method and a few others for the measurement of sound absorption coefficient of building materials are proposed by P. E. Sabine and many other authorities.

Here we have presented a new method named *Masking Method.*

Let the sound source of constant power worked in a sound chamber, and an observer sitting in the same chamber put the receiver to his ears with 1/2″ distanced ofset of $3-A$ audiometer. The noise (caused by $3-A$ audiometer) ranging about from 5 to 10 db. is applied to the receiver. The very deafening effect of ear is available for measuring the reverberation time in a masked condition. By so doing, we can easily see the grade of decay process of sound, consequently we get the absorbing coefficient by these result, e. i. there exists a lineal relation between sensation level and reverberation time, and by aplying the $3-A$ audiometer to the ear, then the following theory can be established.

Let the reverberation time are t_1 and t_2 when minimum audible intensity shifted is ε_1 and ε_2 respectively, we get the following relation.

$$\varepsilon_1 = E_0 e^{-rt_1} \qquad \varepsilon_2 = E_0 e^{-rt_2}$$

$$\therefore \quad \mu = \frac{\log_e \frac{\varepsilon_1}{\varepsilon_2}}{-(t_2 - t_1)} = \frac{2.3 \log_{10} \frac{\varepsilon_1}{\varepsilon_2}}{-(t_2 - t_1)}$$

But

$$\frac{\log_{10} \frac{\varepsilon_1}{\varepsilon_2}}{-(t_2 - t_1)} = -\tan\varphi = -\mu' \qquad \therefore \quad \mu = 2.3 \times \mu'$$

Accordingly

$$\log_{10}(1-\alpha) = \frac{-4V}{cS}\mu'$$

where V, c, S are the known quantities and $\mu' = \tan\varphi$ can experimentally be got according to each frequency.

Comparison of μ (by Masking Method) and μ' (by Variety Intensity Method) of the sound chamber is shown in table 1, and table 2 shows the mean absorbing coefficient of the sound chamber for frequency from C_2 to C_7.

Table 1.

	Variety Intensity Method (μ')	Masking Method (μ)
C_2	−2.00	−2.41
C_3	−2.06	−2.64
C_4	−2.23	−2.87
C_5	−3.26	−3.33
C_6	−3.66	−4.37
C_7	−8.65	−6.64

Table 2.

	Variety Intensity Method	Masking Method
C_2	0.0195	0.0235
C_3	0.0201	0.0258
C_4	0.0218	0.0280
C_5	0.0324	0.0326
C_6	0.0354	0.0429
C_7	0.0859	0.0664

Literatures.

(1) S. Iketani. Growth & Decay of Sound in a Room by Special Types of Sound.
J. Inst. Japanese Architects. Vol. 47, No. 569, 1933.

(2) T. Satow. Critical Survey of Reverberation Theories.
J. Inst. Japanese Architect. Vol. 44, No. 539, 1930.

(3) S. Iketani. Some Consideration on the Theories of Transmission of Sound Through Partitions & its Measuring Method.
J. of Arch. of Waseda Univ. No. 8, 1931.

(4) T. Satow. Acoustics of Auditoriums Ascertained by Optical Treatment in Models.
Paper 118. World Engineering Congress, 1929.

(5) N. Kadokura & T. Satow.
Model Study of Distribution of Sound Intensities in Auditoriums by using Photo Electric Cell.
J. Inst. Japanese Architects. Vol. 42, No. 516, 1928.

(6) T. Satow. Noise in Tokio.
J. of City Problem. 11. 1930.

(7) T. Satow & T. Nakano.
Noise in Subway.
J. of Arch. of Waseda Univ. No. 8, 1931.

(8) T. Satow. Measurement of Sound Absorption Coefficients by Reverberation Masking Method.
J. Inst. Japanese Architects. Vol. 47, No. 574, 1933.

The Relation Between the Shape of Window and Horizontal Illumination.

BY

Lect. **Kōichirō Kimura.** *Kōgakushi.*

1. Introduction.

It is obvious that horizontal illumination through a vertical window varies according to the area and the shape of the window. The problem is to determine the degree of difference in illumination by the change in the area and the shape of the windows.

The author has investigated the above problem to obtain some theoretical conclusions. The result has been obtained from the studies of the following three problems:

(1) The distribution of horizontal illumination obtained through windows of equal area but having different shapes.

(2) The total amount of horizontal illumination obtained through a fixed window in rooms of constant area but differing in shape.

(3) The total amount of horizontal illumination obtained through windows of various shapes but of constant area, in a room of constant shape.

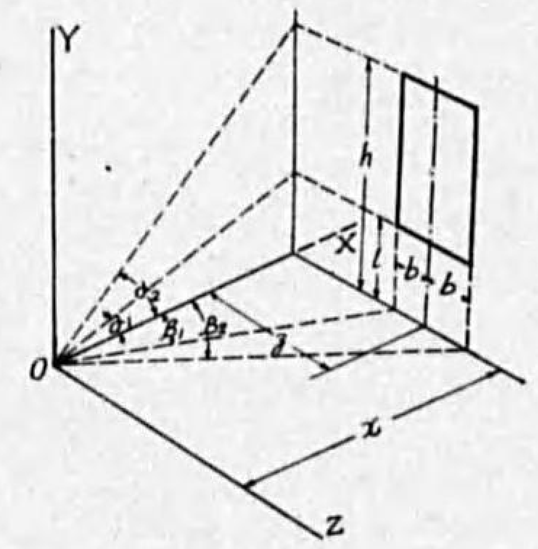


Fig. 1.

2. Method and Results.

The author has assumed the hemi-spherical skydome, having equal brightness, as the source of illumination and not the window surface itself. The horizontal illumination at a certain point in a room is obtained from the window opening, the visible sky-area through it being the source of illumination. The reflected illumination intensity from the ceiling and the walls

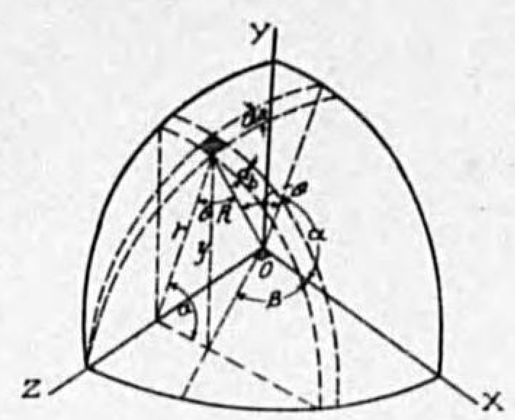


Fig. 2.

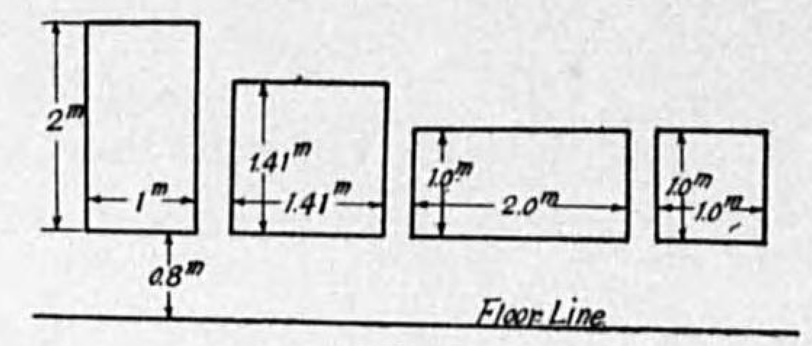


Fig. 3.

upon that point is neglected. Fig. 1 shows these relations.

The following equation comes from the explanation of Fig. 2.

$$I=\int_{\alpha_2}^{\alpha_1}\int_{\theta_2}^{\theta_2} iR^2\cos^2\theta\sin\alpha d\alpha_1 d\theta$$

By solving the above equation by adopting four kinds of window shapes as shown in Figs. 3 the graphs (Fig 4 and 5 or table 1) are obtained.

Table 1. (A) Width of window 1.0ᵐ, height of window 2.0ᵐ, window sill height 0.8ᵐ

x \ z	0ᵐ	1ᵐ	2ᵐ	3ᵐ	4ᵐ	5ᵐ
0ᵐ	0	0	0	0	0	0
0.2	.0366	.0164	.0045	.0015	.00065	.00028
0.5	.0720	.0345	.0103	.0035	.00134	.001025
1.0	.0702	.0427	.0169	.0058	.00184	.001055
2.0	.0443	.0336	.0173	.0103	.00383	.00261
3.0	.0211	.0180	.0174	.0072	.00403	.00278
4.0	.0133	.0122	.0125	.0063	.00466	.00289

Table 1. (B) Width of window 2.0ᵐ, height of window 1.0ᵐ, window sill height 0.8ᵐ

x \ z	0ᵐ	1ᵐ	2ᵐ	3ᵐ	4ᵐ	5ᵐ
0ᵐ	0	0	0	0	0	0
1	.0947	.0633	.0215	.00806	.00308	.00111
2	.0422	.0336	.01723	.01000	.00316	.00270
3	.0214	.0161	.01056	.00584	.00361	.00225
4	.01033	.0089	.00666	.00427	.00250	.00183
5	.00625	.0053	.00417	.00306	.00236	.00114

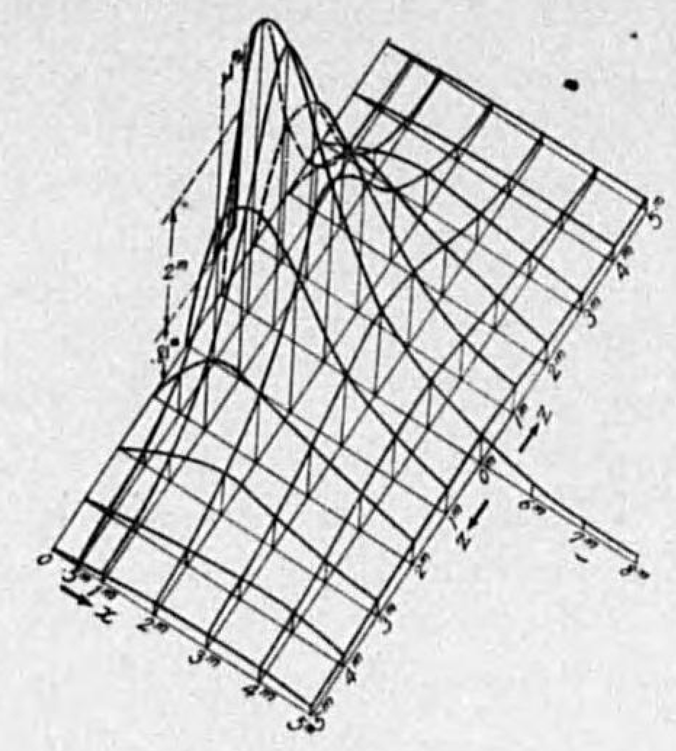


Fig. 4.

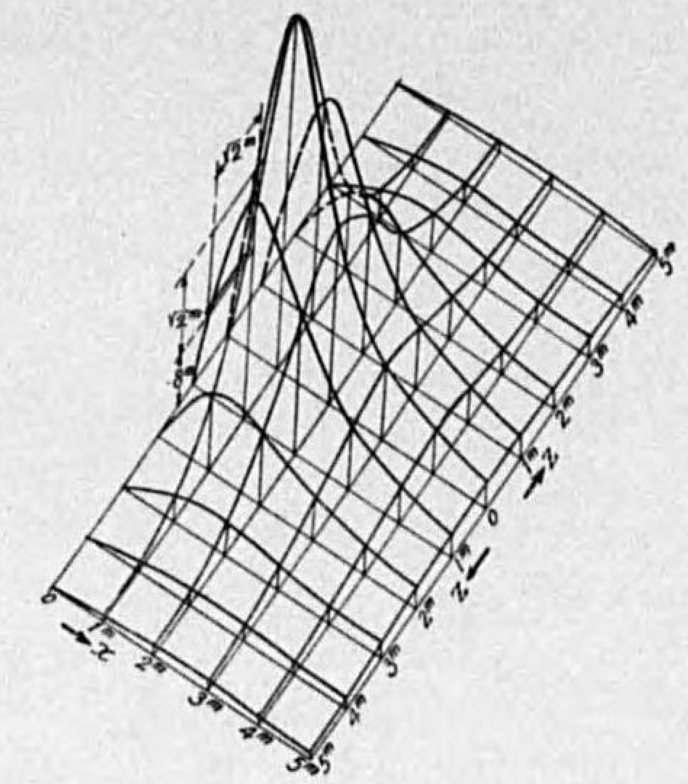


Fig. 5.

In the second problem, if the volumes of the solid bodies with curved surfaces, shown in Figs. 4 and 5 are obtained according to the shapes of the rooms, the volumes will show the total amount of available horizontal illumination to the room. These computations are obtained by the use of the following equation which is derived from the former equation and from Fig. 6.

$$I^0=\int_{A_1}^{0}\int_{B}^{0} I\,dxdz+\int_{A_2}^{0}\int_{B}^{0} Idxdz$$

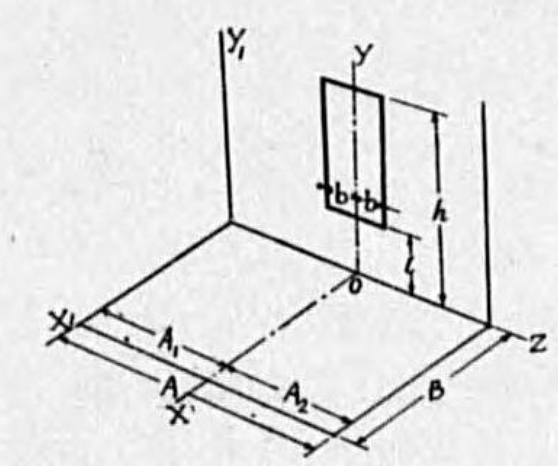


Fig. 6.

In the same problem, for example, if a window having the ratio 1:2 for width and height is used, the total horizontal illumination I° in rooms of various shapes but having a constant area is shown in Table 2 or Fig. 7. It is seen from the results that a room having the length of 5.7 and the width of 3.5 has the maximum horizontal illumination efficiency.

In the third problem, a room of fixed area, namely 4 meter wide and 5 meter long, was chosen, and the shapes of the windows were changed although the area was kept constant at 2 square meters. By using the above equation, the results shown in Table 3 have been obtained.

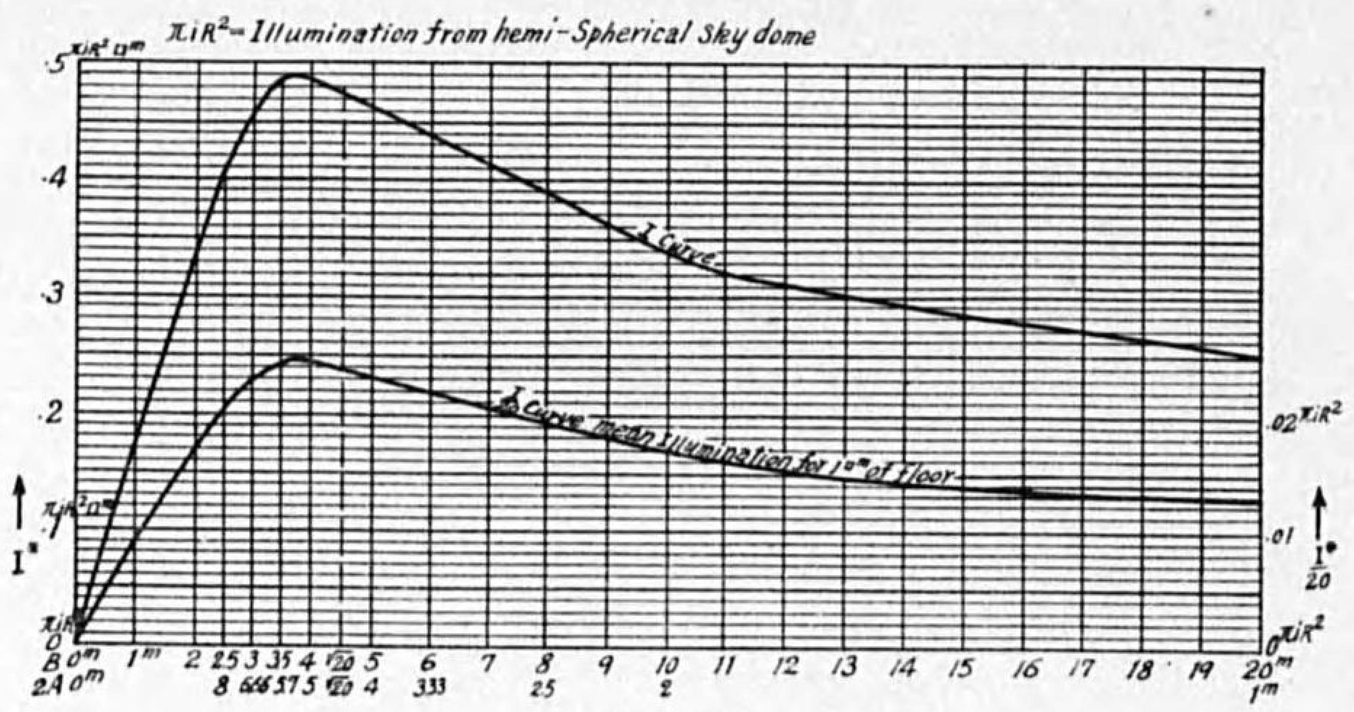


Fig. 7.

Table 2.

width of Room (A)	8.0	6.66	5.7	5.0	$\sqrt{20}$	4.0	2.5
depth of Room (B)	2.5	3.0	3.5	4.0	$\sqrt{20}$	5.0	8.0
I°	.412	.447	.487	.486	.478	.462	.390

unit of $I^{\circ} = \pi i R^2 m^2$

Table 3.

Window-shape / Window sill height 0.8^m	height of Window 2^m / width of Window 1^m	" $\sqrt{2}^m$ / " $\sqrt{2}^m$	" 1^m / " 2^m	" 1^m / " 2^m
I° (unit. $\pi i R^2 m^2$)	.486	.40	.375	.263
Average illumination. $\frac{I^{\circ}}{20}$	.0243	.02	.01875	.01315

Lighting Efficiency of Window in Different Positions.

BY

Lect. **Kōichirō Kimura.** *Kōgakushi.*

1. · Introduction.

When buildings become higher and closer in large cities, then efficiency of daylight illumination inside the buildings is reduced. The efficiency of giving light through windows is determined partly by the height and the width of the obstructions confronting them, and partly by the orientation of the window surfaces themselves.

In the present paper, the author explains how he has obtained by mathematical and graphical methods the lighting efficiency of windows confronted by obstacles. He has chosen, as a standard of comparison, a window facing the south and which is free from obstacles as possessing 100% efficiency.

2. Method.

(a) Illumination Intensity of Skylight on Window Surfaces.

The direct sunlight being excepted, the illumination of the earth's surface by skylight is almost constant all the year round.

The horizontal illumination on the latitude about 35° where most of our large cities are located, is 14,000 lux in summer, 11,000 lux in the spring and autumn seasons, and 8,000 lux in winter.

Based upon these figures and computing the illumination on the vertical plane (here the length of day-time hours is taken into due consideration), following proportional figures are obtained: 3.3 in winter solstices, 6.5 in vernal and autumnal equinoxes, and 9.8 in summer solstices, the unit being expressed in 10,000 lux during one hour. The lower curve in Fig. 1 (B) shows these values, and the area between the curves and the horizontal lines expresses the efficiency of the window surfaces free from obstruction due only to skylight.

When the window is confronted by various objects, the following equation is applicable: viz, assuming the hemispherical skylight having uniform brightness as the source, then the illumination on a vertical window surface is:

$$I_1=\int_0^{\frac{\pi}{2}}\int_0^{\frac{\pi}{2}} iR^2\cos^2\gamma\cos\theta d\theta d\gamma$$

where R is radius of celestial sphere

γ and θ are angle shown in Fig. 2 and

i is the brightness of an infinitesimal area on the celestial surface.

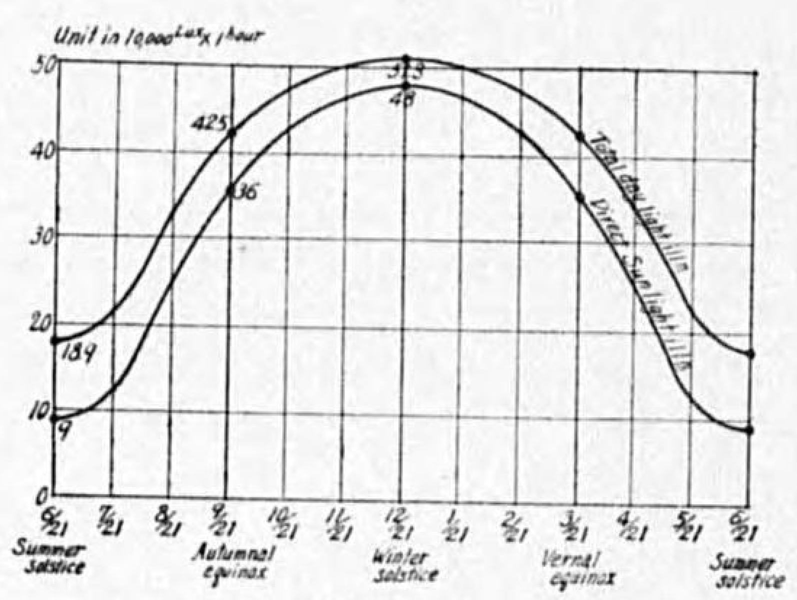


Fig. 1. (A) South window with no obstruction (100%)

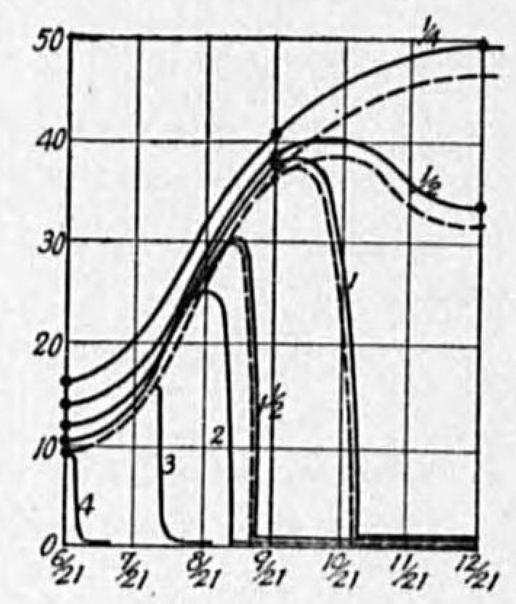


Fig. 1. (C) South facing window confronted by obstruction in front.

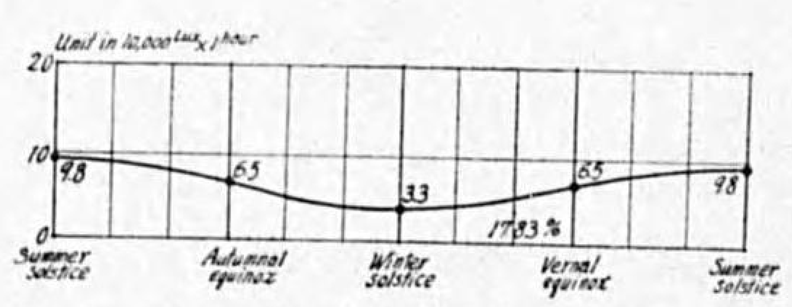


Fig. 1. (B) Illumination by Skylight for a Vertical window with no obstruction.

Accepting the result of this equation as being equal to the area shown in Fig. 1, and changing the angles γ and θ between the integral limit, according to the elevation of the obstactes, the

lighting efficiency of window surfaces confronted by objects may be determined.

(b) Direct Sunlight Illumination.

When the orientation of a window surface is determined, the duration and the illumination of direct sunlight of each season on the vertical surface of the window can easily be determined.

In localities of latitude 35° $N.$, where our principal cities are located, are obtained the following ratios of the values of horizontal illumination multiplied by the duration of sun-shining day-time hours: 80.4 in summer solstices, 54.0 in vernal and autumnal equinoxes, and 23.0 in winter solstices, the unit being expressed in 10,000 lux-hours. For vertical planes, the lower curve in Fig. 1 (A) shows the values of a window facing the south which is free from obsacles.

(c) Total Illumination of Direct Sunlight and Skylight.

The author shows the lighting efficiency of a window in Fig. 1. In the figure, (A) is the summation of the values obtained as explained in the above subtitles (a) and (b). For example, taking the upper curve, which shows the window free from obstacles as 100% efficiency, then efficiency of another window will be obtained by comparing the area of the curve obtained for it with that of the curve of Fig. 1 (A). Fig. 1 (C) and Fig. 3 show graphically a portion of the results obtained.

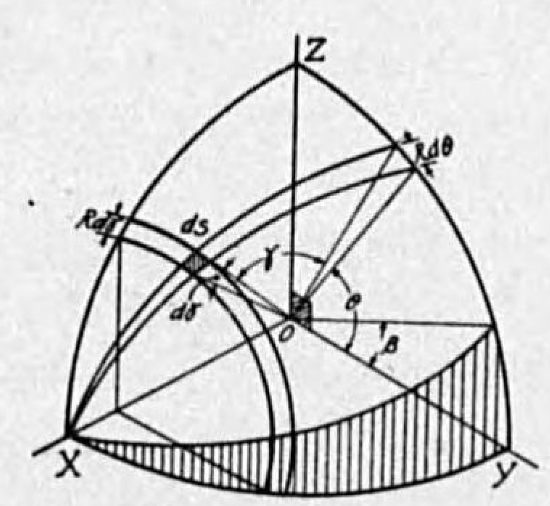


Fig. 2. (A) Obstruction in front of window.

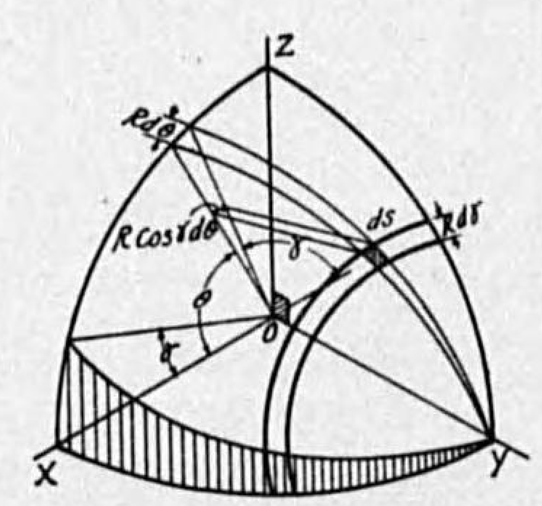
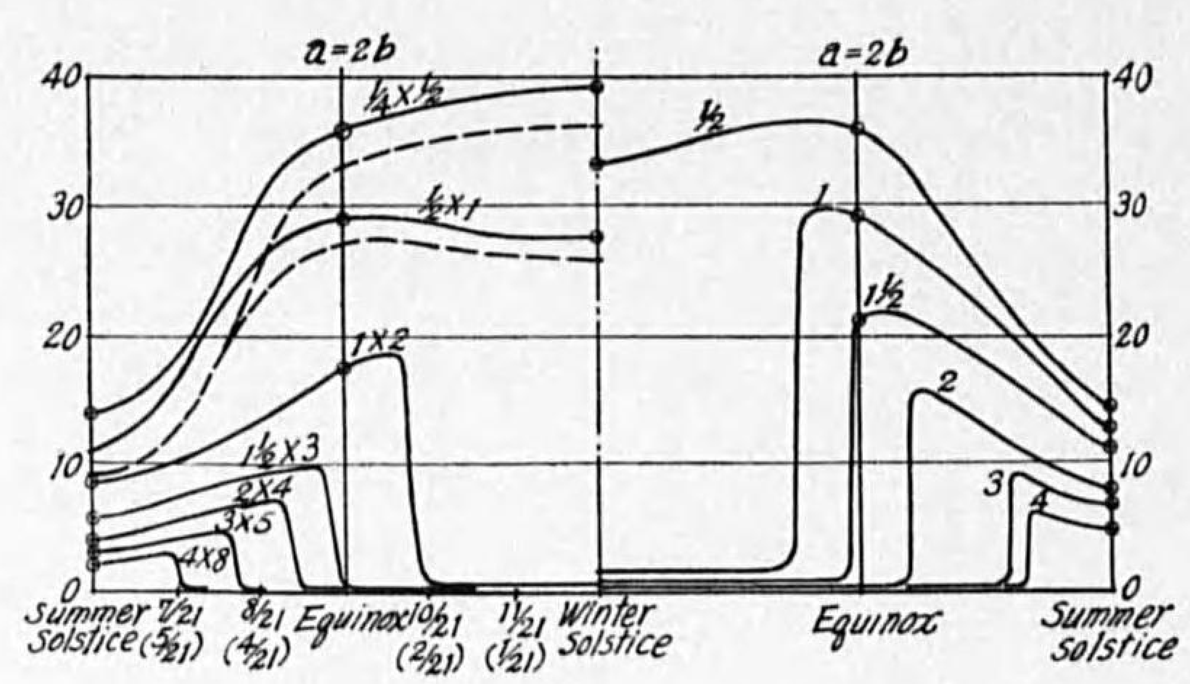


Fig. 2. (B) Obstruction in both side of window.



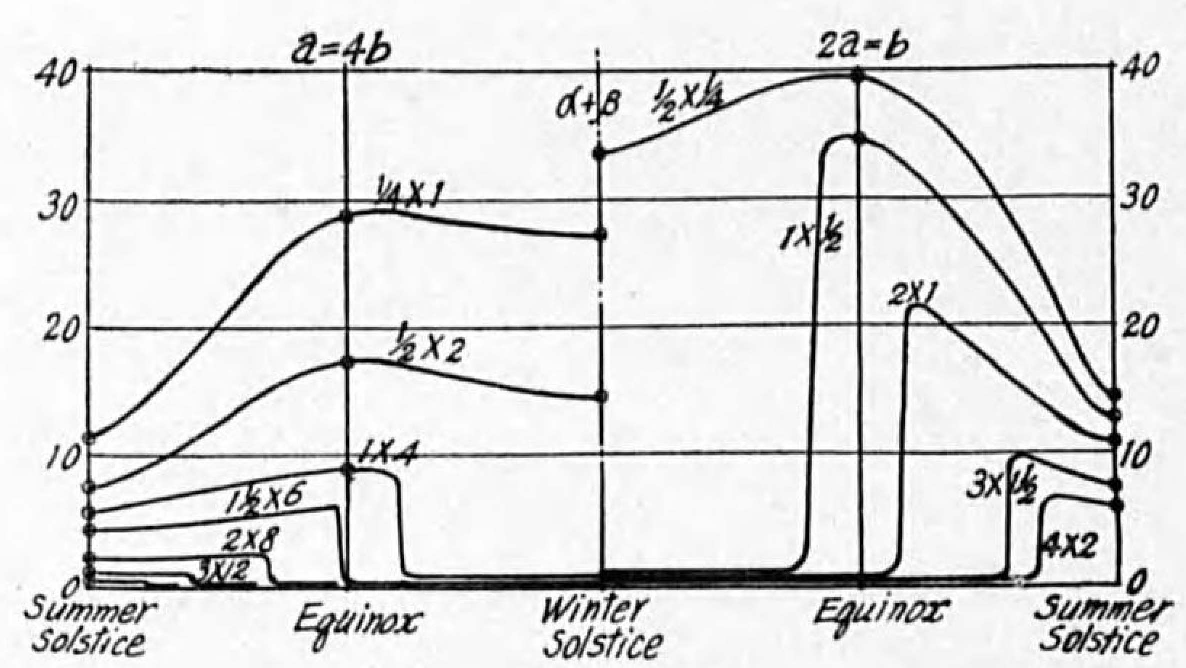


Fig. 3. South facing window of an inner court.

3. Result.

Table 1. South Window.

Altitude of obstruction α or β (h/W)	Obstruction in front of window. Its altitude is α	Obstruction both side of window. Its altitude is β
0	100	100
1/4	96	96
1/2	83	87
1	43.5	67
1½	24.0	52
2	18.0	41
3	7.6	26
4	1.35	21
5	0.13	18.8

h = height of Building. W = width of Street or Court.

Table 2. South Window Facing Inner Court.

α	β	$a=2b$	α	β	$2a=b$
1/4	1/2	81.0	1/2	1/4	90.0
1/2	1	62.0	1	1/2	41.0
1	2	24.2	2	1	16.8
1½	3	11.0	3	1½	4.85
2	4	6.2	4	2	2.70
3	6	3.8	6	3	0.111
4	8	1.6	8	4	0.045
5	10	0.045	10	5	0.023

a = width of Court in front of Window.
b = length of Court from Window to side wall.

Table 3. East Window.

α or β (h/W)	Obstruction in front of Window	Obstruction both side of Window
0	85.0	85.0
1/4	70.5	77.0
1/2	54.0	70.5
1	27.3	49.5
$1\frac{1}{2}$	14.8	40.5
2	9.8	25.3
3	6.0	17.0
4	3.24	10.0
5	2.43	3.24

Great Wall of China Viewed From Architectural Angle

"China," Vol. 22, No. 1, Jan., 1931

BY

Assist. Prof. **Yasushi Tanabe.** *Kōgakushi.*

The Great Wall of China is regarded in general as having been built by the First Emperor of Ch'in. Referring, however, to several reliable treatises on this great building work, the author has found the above interpretation to be incorrect. The author lays stress on that the Emperor gave only the ground work to the Great Wall later on appearing ; the Wall already existing during the dynasties of Ch'in and Han was nothing but a red-clay wall stretching away.

It was after the T'ang dynasty that the Great Wall was actually constructed with brick and stone. Further, the author has dwelt upon the architectural observations concerning those parts of the Wall standing in the vicinities of Pa-ta-ling and chü-yung-kuan.

Pagodas in Tokio-fu

Bulletin of the Tokio Prefectural Office for Historic-Spot Investigation. Vol. 9, Mar., 1932

BY

Assist. Prof. **Yasushi Tanabe.** *Kōgakushi.*

In the introductory note of the first series the author has explained the meaning of pagodas, their origin, classes, their historical observations, and their marked features peculiar to their ages. And the author has specified the localities of the pagodas in Tokyo and its suburban districts. In the second series, the author has made a comment upon each pagoda existing in Tokio and its suburban districts of Tokiofu: the five-storied pagodas at Honmon-j', and at Kwanei-ji, the three-storied pagoda at Hosen-ji, the five-storied pagodas both at Senso-ji and Zôjyô-ji. In addition, the author has commented upon the Tahôtô of Honmon-ji, the three-storied pagoda in the residence of Baron Fujita, Tahôtô in Mr. Hara's mansion. About the above-mentioned pagodas the author has specified the architectural property.

On "The Daito Gozan Shodo-zu"

J. of. A. W. U. No. 8, Feb., 1931

BY

Assist. Prof. **Yasushi Tanabe.** *Kōgakushi.*

Under the heading "The Daito Gozan Shodo-zu" three different scroll-books prepared by hand writing were published, and yet the contents of the three are much the same, so it is well-nigh impossible to determine which is really the original. It is, however, obvious, the author has announced in this paper, that Tettsû, founder of Daijo-ji, was connected with compiling the original. Referring to reliable literature in this line, the author has mentioned that the book was published between the 1st year of Seigen and 2nd year of Kôchô (during these four years). This book affords all important data for the study of "five temples in the Southern Sung dynasty" already existinguished, at the same time through this book only the following is to be confirmed: the Tenjiku style, which was created in the Kamakura ages, is principally ascribable to the traditional influence of Southern Sung.

A Study of "Yin Tsao Fah Shih" By Li Chieh

J. of A. W. U. No. 9, May, 1932

BY

Assist. Prof. **Yasushi Tanabe.** *Kōgakushi.*

"Methods of construction" published in China in the Northern Sung is a book dealing architecture written by Li Chieh. In the present paper the author has introduced the biography of this prominent Chinese writer, has explained the contents of the publication, and has given the following important suggestions based upon this book:—

1. "Methods of Construction" is this only book that tells of the architecture of the Northern Sung period.
2. There have been left few relics through which the building styles in the said period are traceable; the styles are known only through a few remaining structures in Liao and Chin, and this "Methods of Construction" by Li Chieh.
3. In early days of the Southern Sung period a sort of building styles, which may be called "Nampo-kei" (a southern family-line), perhaps existed along the coast of the Yangtze-Kiang. What is called as the Tenjiku style in Japan is the Nampo-kei style introduced into Japan in the period of the Second Emperor of Southern Sung.
4. The Northern Sung style resembles what is called in Japan as an Tenjiku style. After Sung advanced to the south, this style was gradually introduced into the southern part of China; here the Tenjiku style was partly adopted to the introduced style, and then, so-called the Southern Sung style was completed, so the author judges.

5. The arabesque style introduced into Japan in the latter part of Southern Sung period is in reality the Southern Sung style. Judging from "the Daito Gozan Shodo-zu, it is obvious that in those days there was a clever combination of the Tenjiku style with the Southern Sung style.

On Stone Shibi

Bulletin of the Japan Society for Preserving Scenic Beauties
and Historic and Natural Monuments
Vol. 6, No. 8, Aug., 1931

BY

Assist. Prof. **Yasushi Tanabe.** *Kōgakushi.*

"Stone Shibi" must have been found in some district of this land according to "the History and Origin of Cathedrals of Daian-ji" and "the Ryuki Shizai-chô," but no relic of the kind was ever discovered. The author, however, has found one in Sôjya, Gumma-ken: it stands 3 shaku 3 sun high, which is quite large in size for a piece of the kind, and is regarded as belonging to the Nara dynasty.

On Construction of Sagami Kokubunji Temple

J. of I. J. A. Vol. 45, No. 547, July, 1932

BY

Assist. Prof. **Yasushi Tanabe.** *Kōgakushi.*

At the ruins of Sagami Kokubunji Temple, in Kokubu, Ebina village,

Kanagawa-ken (former Sagami-koku) still there remain mouldering ground-stones of "Kondo," preaching hall, and dagobas. The author examined these himself, and has described important things with regard to the construction and annals of the temple once existed. Hereupon, the author has disproved Dr. Numata's opinion that the temple belongs to the Asuka period. Besides, the author has advanced a theory that cathedrals of the Horiuji style belongs not merely to the Asuka period, but too these cathedrals were locally constructed in the Tempiyô period.

A Study of Prehistoric Dwellings in Japan

Proceedings of the World Engineering Congress, Tokyo, 1929

Assist. Prof. **Yasushi Tanabe.** *Kōgakushi.*

The dwelling-houses of Japan owe their development partly to topographical environment, and partly to the way of living in every period. These motive power produced special types of buildings, and every age had something new to add to them as civilization progressed. Thus, our present houses are of a type unique in the world. But there are many opinions among the learned concerning their improvement and other essential problems, in the present age when the world considers the question of dwellings or housing policy as one of the most important social problems.

Investigation as to the origin of our dwellings, or what dwellings our primitive forefathers had in prehistoric times affords not only a means for the study of our houses, but also is important as throwing light on the dawn of our architectural history. The author therefore has proposed a study of prehistoric dwellings judged from various archaeological researches he has made for the past several years.

List of Miscellaneous Papers

Building Construction after the Great Earthquake.

BY

Prof. **Tachū Naitō.** *Kōgakuhakushi.*

(World Engineering Congress. Tokyo, Paper No. 718, 1929)

Earthquake-proof Construction.

BY

Prof. **Tachū Naitō.** *Kōgakuhakushi.*

(Reprinted from the Proceedings of the Third Pan-Pacific Science Congress, Tokyo, 1926)

Strength and Reliability of Arc Welded Joints.

BY

Prof. **Tachū Naitō.** *Kōgakuhakushi.*
Inst. **Akira Tsuruta.** *Kōgakushi.*

(J. of I.J.A. Vol. 44, No. 536, Aug., 1930)

Test on Grip Length of Bar in Reinforced Concrete Beam.

BY

Prof. **Tachū Naitō.** *Kōgakuhakushi.*
Inst. **Akira Tsuruta.** *Kōgakushi.*

(J. of A.W.U. No. 8, March, 1931)

Test on a Strength of Steel Beam Joints embeded in Reinforced Concrete.

BY

Prof. **Tachū Naitō.** *Kōgakuhakushi.*
Inst. **Akira Tsuruta.** *Kōgakushi.*

(J. of A.W.U. No. 8, March, 1931)

Comparative Test on Riveted and Welded Joints of Steel Structure.

BY

Prof. **Tachū Naitō.** *Kōgakuhakushi.*
Inst. **Akira Tsuruta.** *Kōgakushi.*

(J. of Japanese Weld. Soc. Vol. 2, No. 3, Feb., 1932)

The Damage to Buildings done by the Quake on November 1930 in the Northern Districts of Izu.

BY

Prof. **Tachū Naitō.** *Kōgakuhakushi.*
Assist. Prof. **Saburo Soshiroda.** *Kōgakushi.*
Mr. **Tsutomu Sakai.** *Kōgakushi.*
Inst. **Akira Tsuruta.** *Kōgakushi.*

(J. of A.W.U. No. 9, May, 1932)

The Damage to Buildings done by the Quake on September 1931 in the Northern Districts of Kantow.

BY

Assist. Prof. **Saburō Soshiroda.** *Kōgakushi.*
Mr. **Tsutomu Sakai.** *Kōgakushi.*
Inst. **Akira Tsuruta.** *Kōgakushi.*

(J. of A.W.U. No. 9, May, 1932)

On Calculation of Arc Weld Joints.

BY

Inst. **Akira Tsuruta.** *Kōgakushi.*

(J. of Japanese Weld. Soc. Vol. 2, No. 6, Aug. 1932)

Effect of Abrasion And Impact on Buildings.

BY

Prof. **Kyōji Yoshida.** *Kōgakuhakushi*

(J. of I.J.A. Vol. 47, No. 568 March, 1933)

Relations of Building Materials with Humidity, Temperature and Air Conditioning.

BY

Prof. **Kyōji Yoshida.** *Kōgakuhakushi*

(J. of Soc. Dom. & San. Eng., Vol. 17, No. 4, April, 1933)

Rubber Goods as Building Materials.

BY

Prof. **Kyōji Yoshida.** *Kōgakuhakushi*

(J. of Soc. Rubber Ind. Japan, Aug., 1933)

Durability of Buildings.

Section I : Effect of abrasion and impact.
Section II : Freezing effect.
Section III : Effect of expansion and contraction of materials.

BY

Prof. **Kyōji Yoshida.** *Kōgakuhakushi*

(published July, 1933. Tokyo.)

The Secret of the "Roaring Dragon" in Nikko.

BY

Assist. Prof. **Takeo Satow.** *Kōgakushi.*

(J. of A.W.U. No. 5, Sept., 1928)

Model Studies of Distribution of Sound Intensities in Auditoriums by using Photo-Electric Cell.

BY

Assist. Prof. **Takeo Satow.** *Kōgakushi.*
Assist. Prof. **Noriyuki Kadokura.** *Kōgakushi.*

(J. of I.J.A. Vol. 42, No. 516 Dec., 1928)

Acoustics of Auditorium ascertained by Optical Treatment in Models.

BY

Assist. Prof. **Takeo Satow.** *Kōgakushi.*

(World Engineering Congress, Paper No. 118, 1929)

Acoustics of the Hibiya Municipal Auditorium, Tokyo City.

BY

Assist. Prof. **Takeo Satow.** *Kōgakushi.*

(J. of J.A. Vol. 7, No. 1, July, 1930)

Critical Survey of Reverberation Theories.

BY

Assist. Prof. **Takeo Satow.** *Kōgakushi.*

(J. of I.J.A. Vol. 44, No. 539, Nov., 1930)

The Fundamental Analysis of Hearing in Auditoriums—The Basis of Acoustical Design ; Part I.

BY

Assist. Prof. **Takeo Satow.** *Kōgakushi.*

(J. of I.J.A. Vol. 44, No. 539, Nov., 1930)

Distribution of Sound Energy Density and Reverberation in Auditoriums—The Basis of Acoustical Design ; Part II.

BY

Assist. Prof. **Takeo Satow.** *Kōgakushi*

(J. of I.J.A. Vol. 47, No. 572, June, 1933)

Acoustics and Hearing in Auditoriums ; Part III.

BY

Assist. Prof. **Takeo Satow.** *Kōgakushi.*

(J. of I.J.A. Vol. 47, No. 573, July, 1933)

Measurement of Sound Absorption Coefficients by Reverberation Masking Method.

BY

Assist. Prof. **Takeo Satow.** *Kōgakushi.*

(J. of I.J.A. Vol. 47, No. 574, Aug., 1933)

Criterion for the Arrangement of Seats in Motion Picture Theatres with Special Reference to the Picture Distortion.

BY

Assist. Prof. **Takeo Satow.** *Kōgakushi.*

(J. of I.J.A. Vol. 42, No. 511, July, 1928)

The Design of Seating for Good Visibility.

BY

Assist. Prof. **Takeo Satow.** *Kōgakushi.*

(J. of I.J.A. Vol. 47, No. 569, 1933)

Some Considerations on the Transmission Theories of Sound through Partition and its Measuring Methods.

BY

Inst. **Sadao Iketani.** *Kōgakushi.*

(J. of A.W.U. No. 8, March, 1931)

On the Transmission of Sound through Partition.

BY

Inst. **Sadao Iketani.** *Kōgakushi.*

(J. of I.J.A. Vol. 45, No. 547, Aug., 1931)

Über dem Verfallen des Nachhallschalles zwischen den zwei durch Öffnung begrenzten Räumen.

BY

Inst. **Sadao Iketani.** *Kōgakushi.*

(J. of A.W.U. No. 9, May, 1932)

On the Decay of Sound in Large Room.

BY

Inst. **Sadao Iketani.** *Kōgakushi.*

(J. of I.J.A. Vol. 46, No. 565, Dec., 1932)

Sound Growth and Decay in a Room when Special Types of Sound Sources are given.

BY

Inst. **Sadao Iketani.** *Kōgakushi.*

(J. of I.J.A. Vol. 47, No. 569. 1933)

Some Proposal to the Improvement for the defect of the Slow-burning Construction.

BY

Assist. Prof. **Saburō Soshiroda.** *Kōgakushi.*

(J. of A.W.U. No. 8, March, 1931)

On the Construction of Airtight Steelsashes and its Infiltration.

BY

Assist. Prof. **Saburō Soshiroda.** *Kōgakushi.*

(J. of A.W.U. No. 9, May, 1932)

Two Problems for Airtight Steelsash and Automatic Revolving Door.

BY

Assist. Prof. **Saburō Soshiroda.** *Kōgakushi.*

(J. of I.J.A. Vol. 47, No. 567, Feb., 1933)

On Dry Construction for Heat Insulation.

BY

Assist. Prof. **Saburō Soshiroda.** *Kōgakushi.*

(J. of I.J.A. Vol. 47, No. 569, 1933)

A Study of Deterioration of Wooden Column oots Fof Structures and A Method of Decay Provention.

BY

Assist Prof. **Saburō Soshiroda.** *Kōgakushi.*

(J. of A.W.U. No. 10, June, 1933)

Studies of the Relation between Sun-Light and Vertical Window of Various Aspects.

BY

Lect. **Kōichirō Kimura.** *Kōgakushi.*

(J. of I.J.A. Vol. 43, No. 527, Nov., 1929)

On the Amount of Sunlight Passing through Window in Principal Cities in Japan.

BY

Lect. **Kōichirō Kimura.** *Kōgakushi.*

(World Engineering Congress. 1929 paper No. 298)

Shape of Building and its Shadow by Sun-light.

BY

Lect. **Kōichirō Kimura.** *Kōgakushi.*

(J. of I.J.A. Vol. 45, No. 531, March, 1931)

An Experiment for Light Transmission of Deck-glasses.

BY

Lect. **Kōichirō Kimura.** *Kōgakushi.*

(J. of I.J.A. Vol. 47, No. 569, 1933)

A Measurement of Daylight Illumination.

BY

Lect. **Kōichirō Kimura.** *Kōgakushi.*

(J. of A.W.U. No. 10, June, 1933)

Contents of the Preceding Number.

Memories of the Faculty Science and Engineering, Waseda University.

昭和八年十二月 十 日印刷
昭和八年十二月十五日發行

編輯兼發行者　東京市牛込區　早稻田大學

印刷者　東京市牛込區榎町七番地　五十嵐良晃

印刷所　東京市牛込區榎町七番地　日清印刷株式會社

終