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CONTENTS.

Condensed Statement on the Construction of Earthquake-proof Wooden Buildings.

The construction of earthquake-proof buildings differs in method according to the kind of materials used. But the earthquake-proof buildings which will be built at Sakata, Yamagata Prefecture, where great havoc was wrought by earthquake last October, will probably, in view of the conditions of the locality, be constructed of wood. As these structures will comprise public buildings, shops, and city and country houses, the arrangement of rooms will be necessarily diverse, some being simple and others complicated; the outward appearance of the buildings will also differ, for some of them may be one storied and others two-storied. The principle of construction, however, can not but be one and the same, and therefore we give herewith a condensed statement as regards the method of constructing earthquake-proof wooden buildings:—

Construction of the Foundation Works.

Materials for the foundation work may be one of three kinds, viz. 1.—concrete; 2.—" wariguri " (broken stone); 3.—" rōsoku " (stone blocks). The concrete may be made either of cement, or of lime or of both cement and lime mixed; any one of which is better than either the second or the third kinds. Cement concrete is best of all.

In constructing the foundation work, if the soil should be found too soft or damp, piles must first be driven to afford a firm foundation; but the driving of piles should be dispensed with in places where the soil is dry.

For the foundation or footings to be placed upon the ground work, flat, broad stones should be selected, and they should be buried in the earth for half of their height.

Construction of the Framework.

In constructing an earthquake-proof building, it is deemed advantageous to use either foundation-sills ("dodai") or foot-bracers ("ashigatame"), and the method of using them may be seen, 1.—In an ordinary foundation sill (as in Fig. 1.); 2.—In a foot bracer applied to one side of a pillar (as in Fig. 2.); 3.—In a foot bracer stuck between the two pieces of timber forming a double pillar (as in Fig. 3.); and 4.—In two foot bracers so fixed as to embrace both sides of a pillar (as in Fig. 4.) Should more stability be required in the case of Fig. 1., the connection of the pillar and the foundation-sill should be firmly joined by means of iron clamps or straps. In adopting the methods shown in Fig. 2. and 4., foot bracers having a thickness of over one-third of the respective pillars should be used. In case of adopting the method shown in Fig. 3., one solid pillar should be used at each corner of a building, notwithstanding the pillar may be weakened by foot bracer. Still another method is to use pieces of timber as a pillar as shown in Figs. both 5 and 6. In that case, blocks of wood just fitting the space between the pieces of timber should be inserted at every two or three feet, and should be fastened with a metal bond, wire, or bolt in order to prevent the pillar from bending or twisting.

It is desirable that foundation-sills and foot bracers should be crossed with a brace at the four corners and fastened with bolts as in Figs. 7. and 8.

One or two through-bracers ("tōshinuki") should be used as in the case of foot bracers and fastened with bolts as in Figs. 9 to 14; but in the cases of Figs. 11 and 14, the blocks of wood should be inserted at both surfaces of the through-bracers.

In the intermediate space between the pillars where no window or door is to be made, or where outward appearance is of no particular consequence, struts should be placed and fastened to the through-bracers and pillars with bolts.

"Shikii" and "kamoi" (the lower and upper grooved beams respectively in which doors or shutters slide), should be fixed to the pillars simply with screws, if possible, in order to avoid cutting into the pillars and weakening them.

"Nageshi" (a horizontal pieces of timber used in the framework of a building) should be attached to the surface of the pillars and fastened with bolts. Moreover, the weak point of the construction at the four outer corners should be strengthened by jointing the "nageshi" with L shaped metal straps. In places, however, where outward appearance is of no consequence, the "nageshi" should be overlapped at the ends and bolted to the pillars.

In regard to the method of combining with the pillars such lateral timbers as "dōsashi," floor beams and so forth, if possible, only one or two of the timbers should be used in a similar way as in the case of foot bracers and "dosashi."

The connection of "keta" (top ties) and pillars should always be made in the same way as that recommended for pillars and their foot bracers.

The combination of the roof and walling of a building should be effected as in Figs. 15, 16, and 17, e.g. either by holding the upper part of a pillar between double tie beams placed on wall plates, or by using double rafters and letting tie beam fall upon the tenon of a pillar. In both cases, bolts or iron straps should be used in fastening.

Construction of the Roof framing.

Materials of too great a dimension should be avoided in the construction of the roof and the construction should be as in Figs. 15 to 20, using such scantlings as will be just large enough to bear the weight of the roof itself together with the pressure of the wind and the weight of snow; and all connections should be effected with thick wire, iron straps or bolts.

Between the principal rafters, struts, or braces or both should be used and the whole frame of the roof should be bound as in Fig. 20, with iron clamps, or bolts.

For resisting earthquake shocks, a light roof is preferable and therefore all roofs should be made as light as is consistent with their function of keeping out the wind and cold. In case of using tiles, it is better to fasten them with nails or wire.

Joints.

Joints in whatever part of a building should be made as simply as possible, for though a complicated joint may seem strong, yet in reality it is weak. It is desirable that a joint should be made as in Figs. 21, 22 and 23, wood or iron fish-plates being fastened with bolts. In making a tenon, also, the simpler its form the better.

In case a jointed pillar is to be substituted for an ordinary one in building a two or three storied house, the joint of such post should be made at a point about two or three feet either above or below the upper floor line of the house.

Foundation-sills (" dodai "), foot bracers (" ashigatame "), " tōshinuki," "dōsashi," "keta," floor beams and tie beams should all be fixed so as to project somewhat at their outer extremities.

Attachment of a Porch or a Shed with the Main Building.

As regards attaching a porch or a shed to the main building, the old method of tenoning or nailing should be entirely abandoned, as it is dangerously faulty and the two structures should be joined in the same manner as "keta" and pillars are connected.

Materials.

As the quality as well as the size of iron materials has an important bearing upon the construction of a building, serious attention must, of course, be called to the selection of materials and also to the number and distribution of bolts. Besides, the size of washers should be considered; large ones being preferable to small.

Wooden materials are subject to shrinkage, owing to which the bolts lose their tightness and therefore well-seasoned timbers should be selected.

Conclusion.

The most essential points in regard to the construction of a earthquake-proof wooden building lie in the method of building the foundation; in the preservation of the entire power and function peculiar to each timber intact as far as possible, and in case some weakening be unavoidable, re-enforcing it by the application of iron that is much stronger than wood; in the use, wherever possible, of triangle frames, in accordance with the principle that a triangle affords an unchanging form of structure; and finally in the additional strengthening of the whole frame work by the further use if iron materials, thus combining all the parts into a stable and united construction.

Fig. 1.

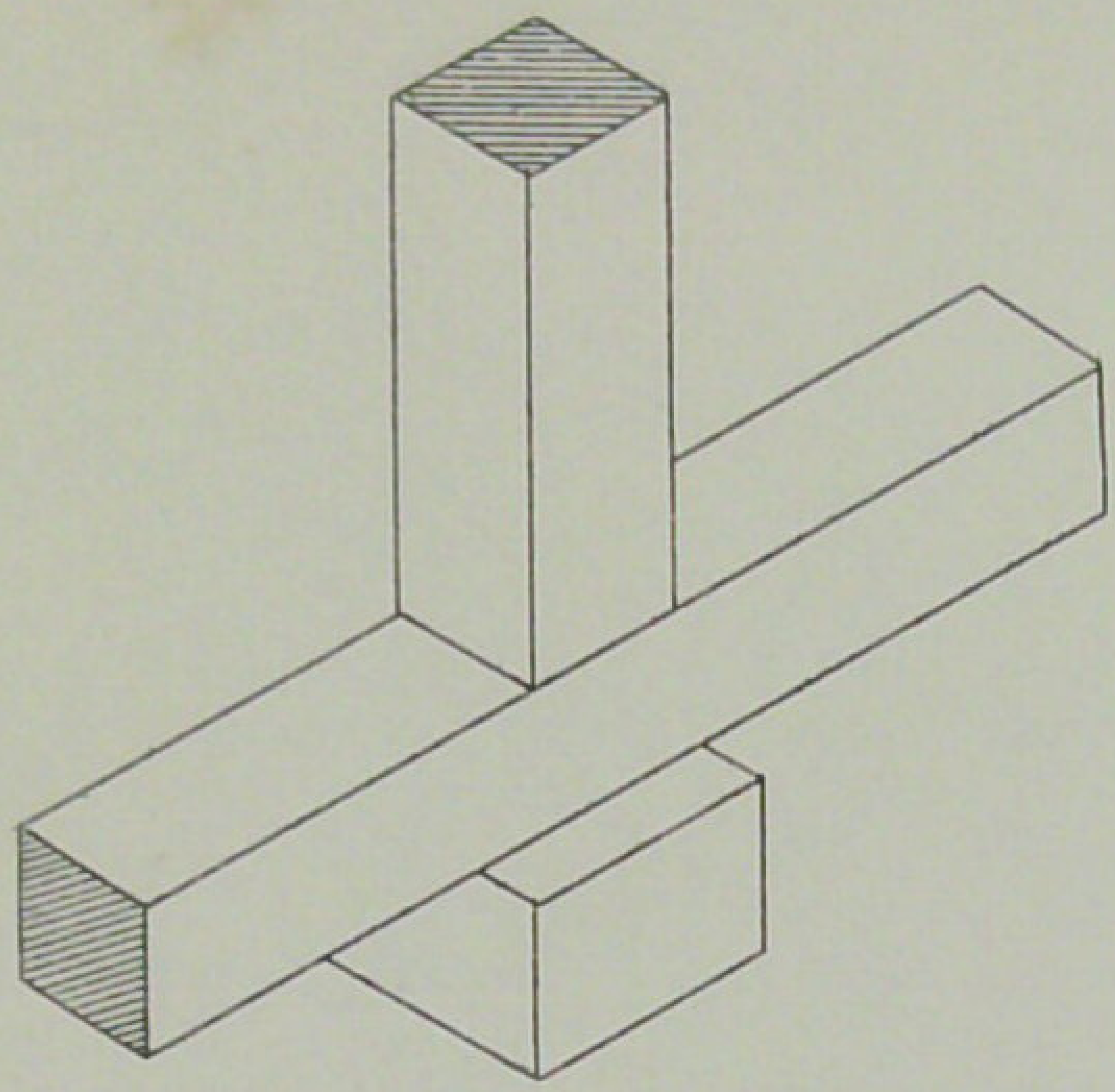


Fig. 2.

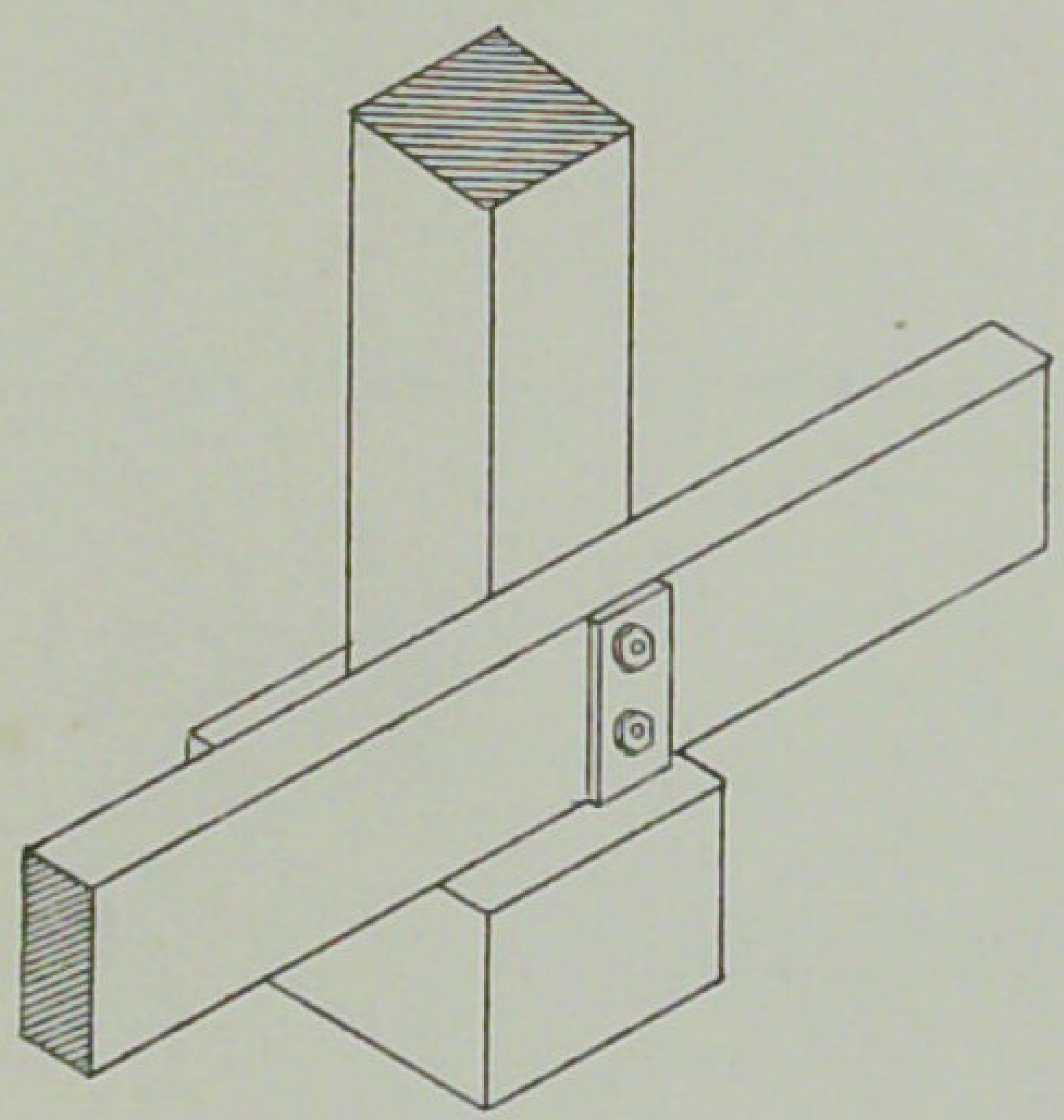


Fig. 3.

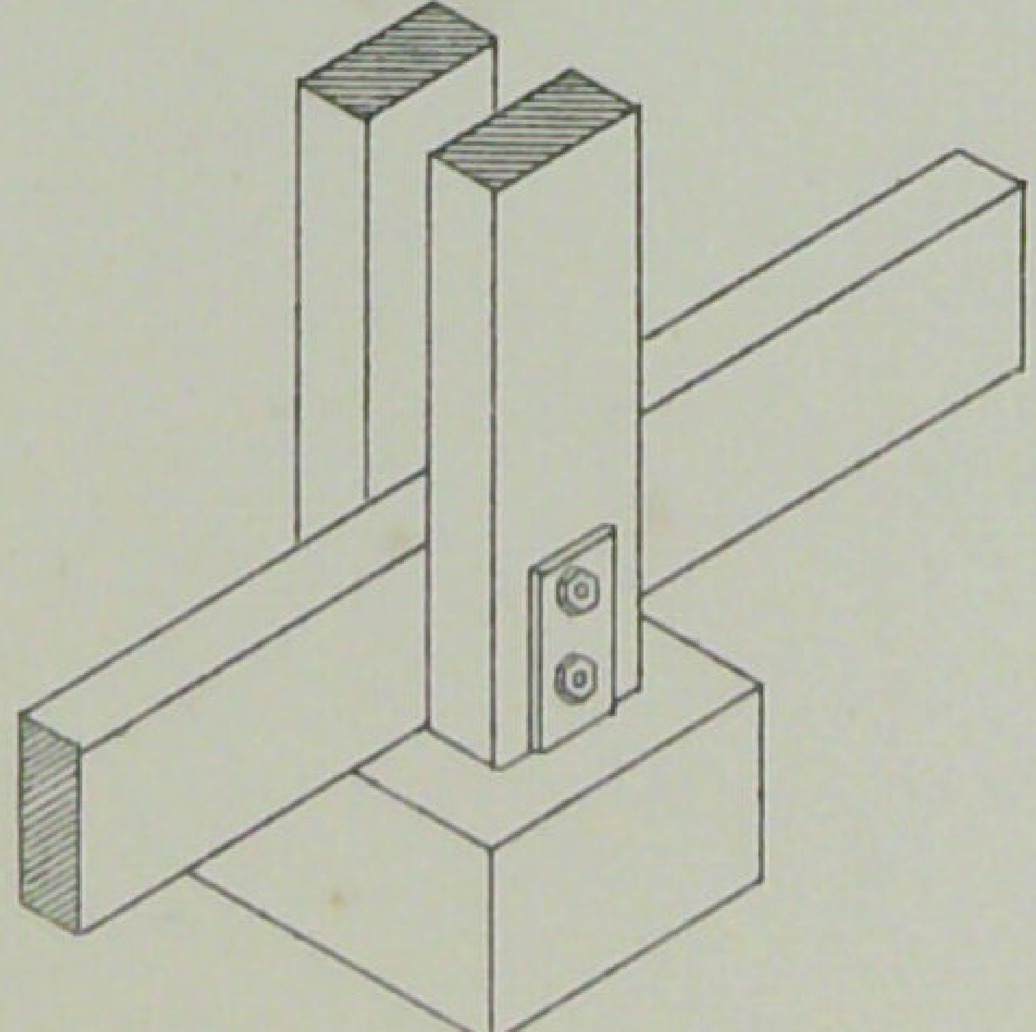


Fig. 4.

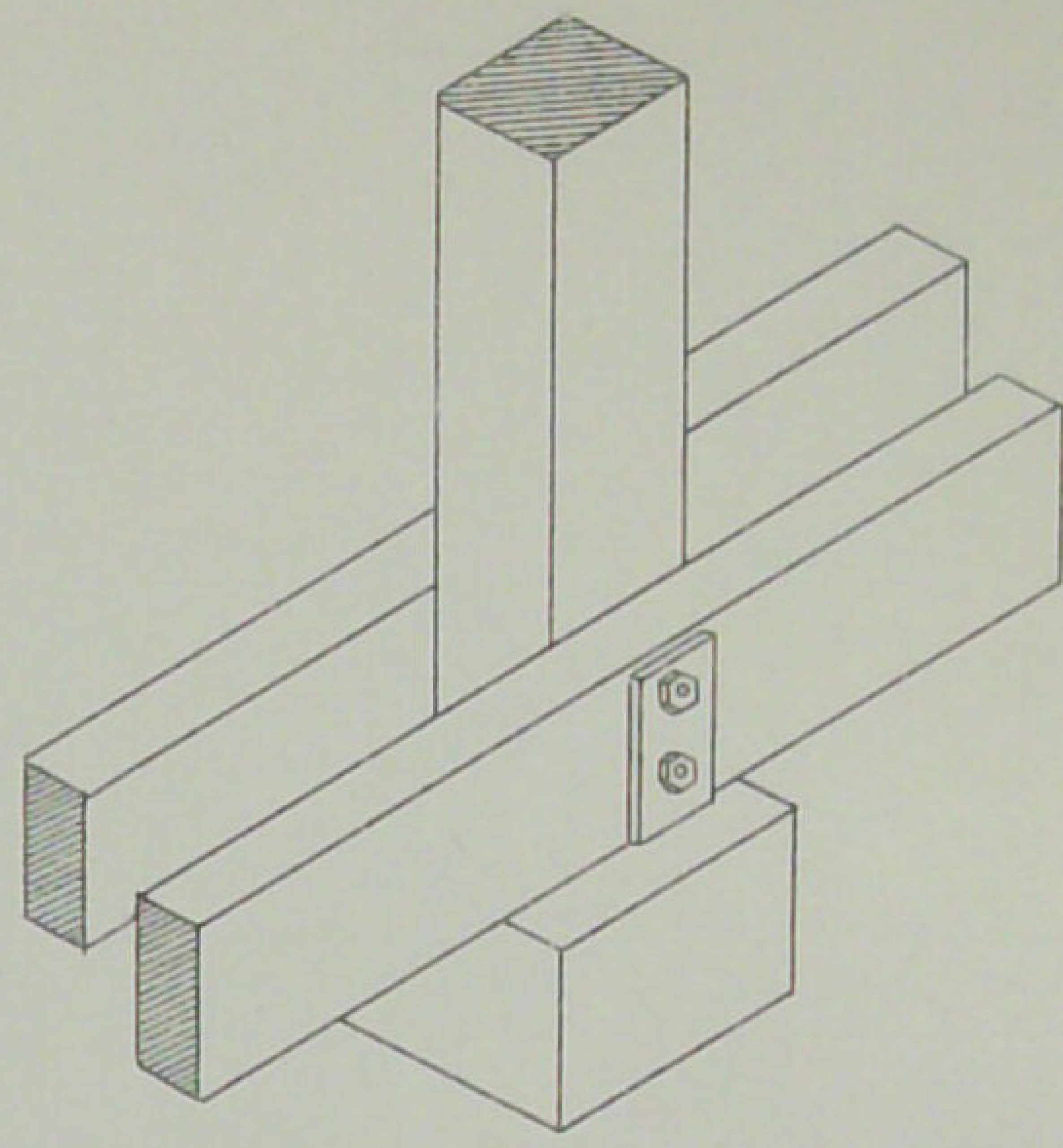


Fig. 5.

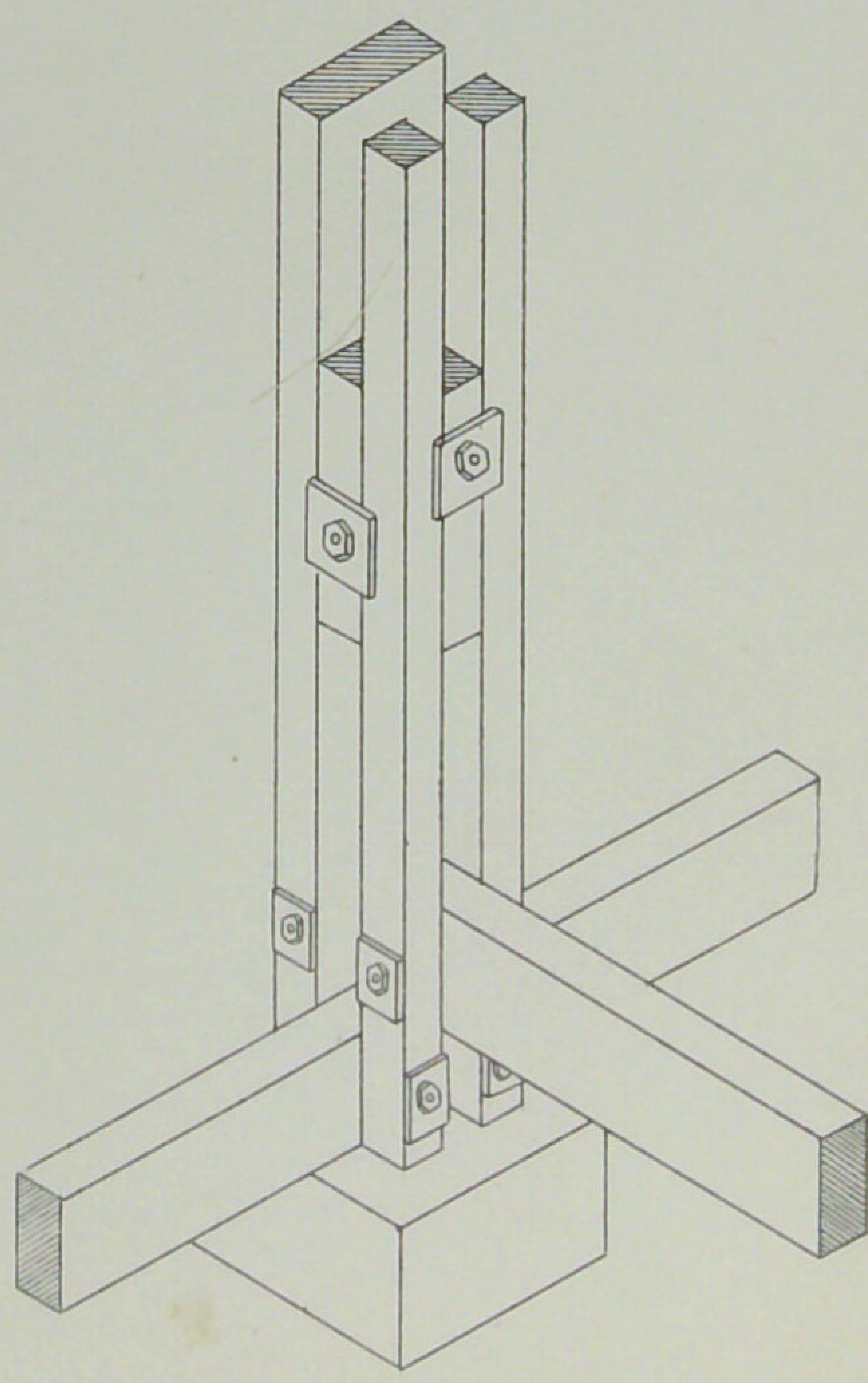


Fig. 6.

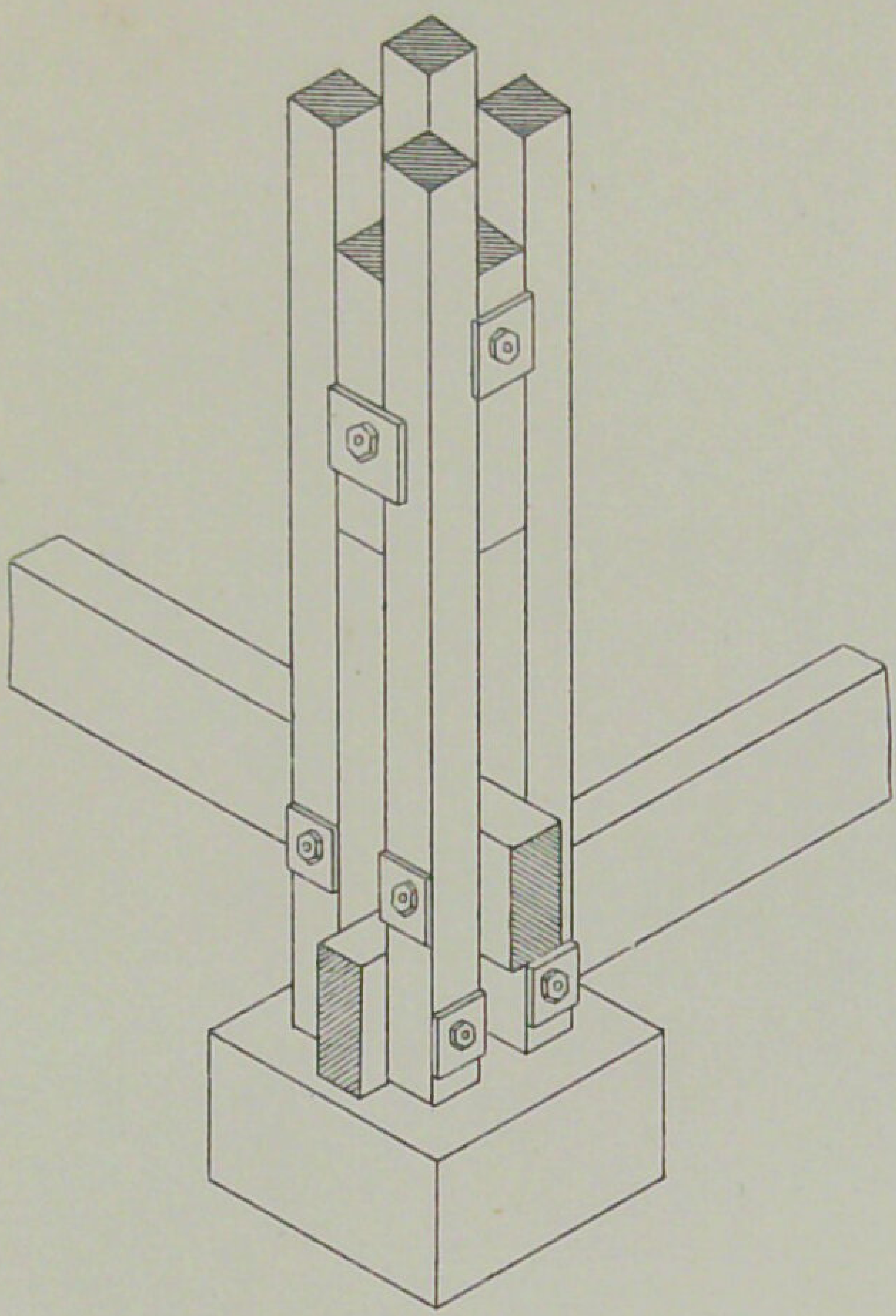
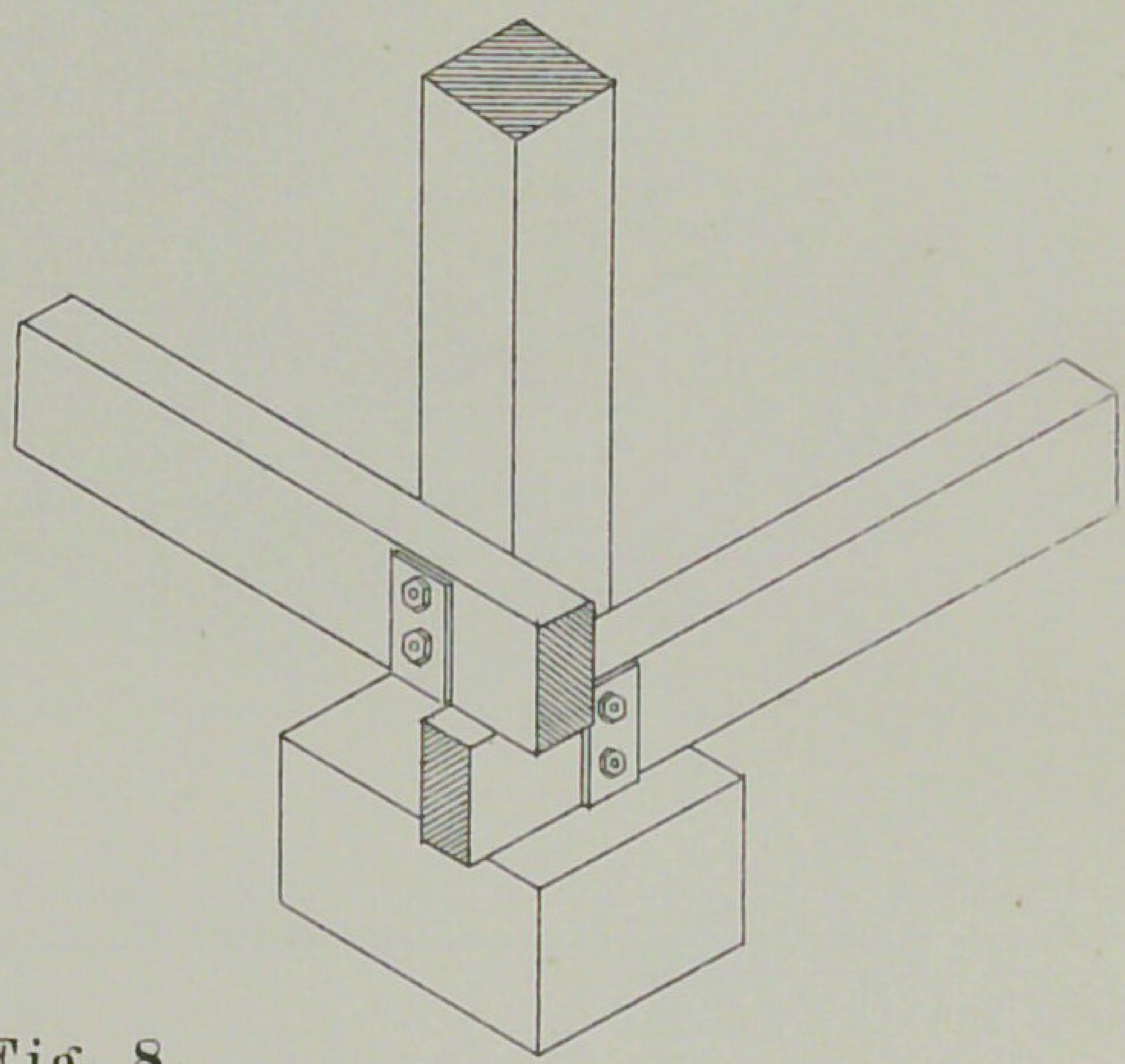


Fig. 7.



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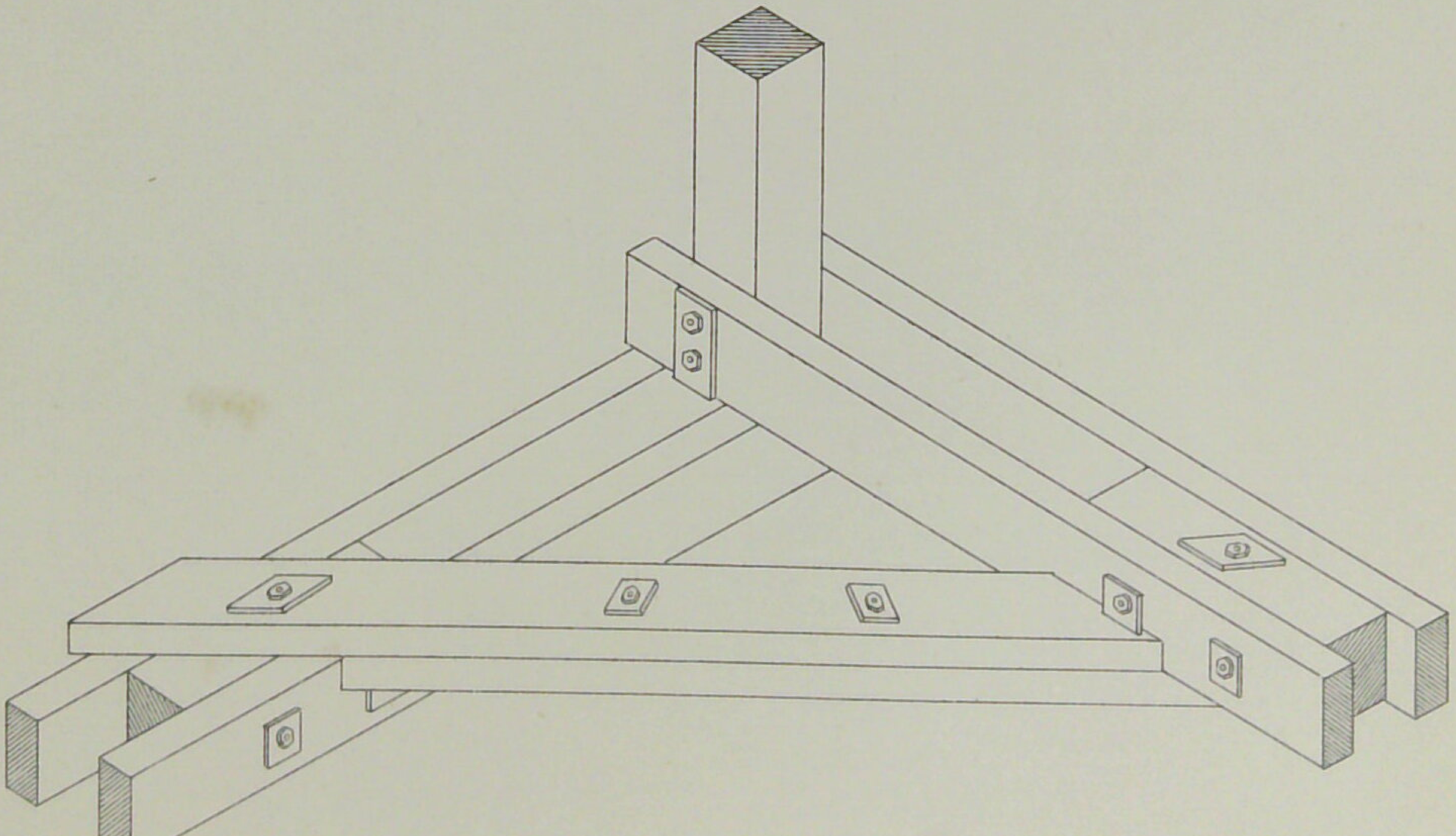

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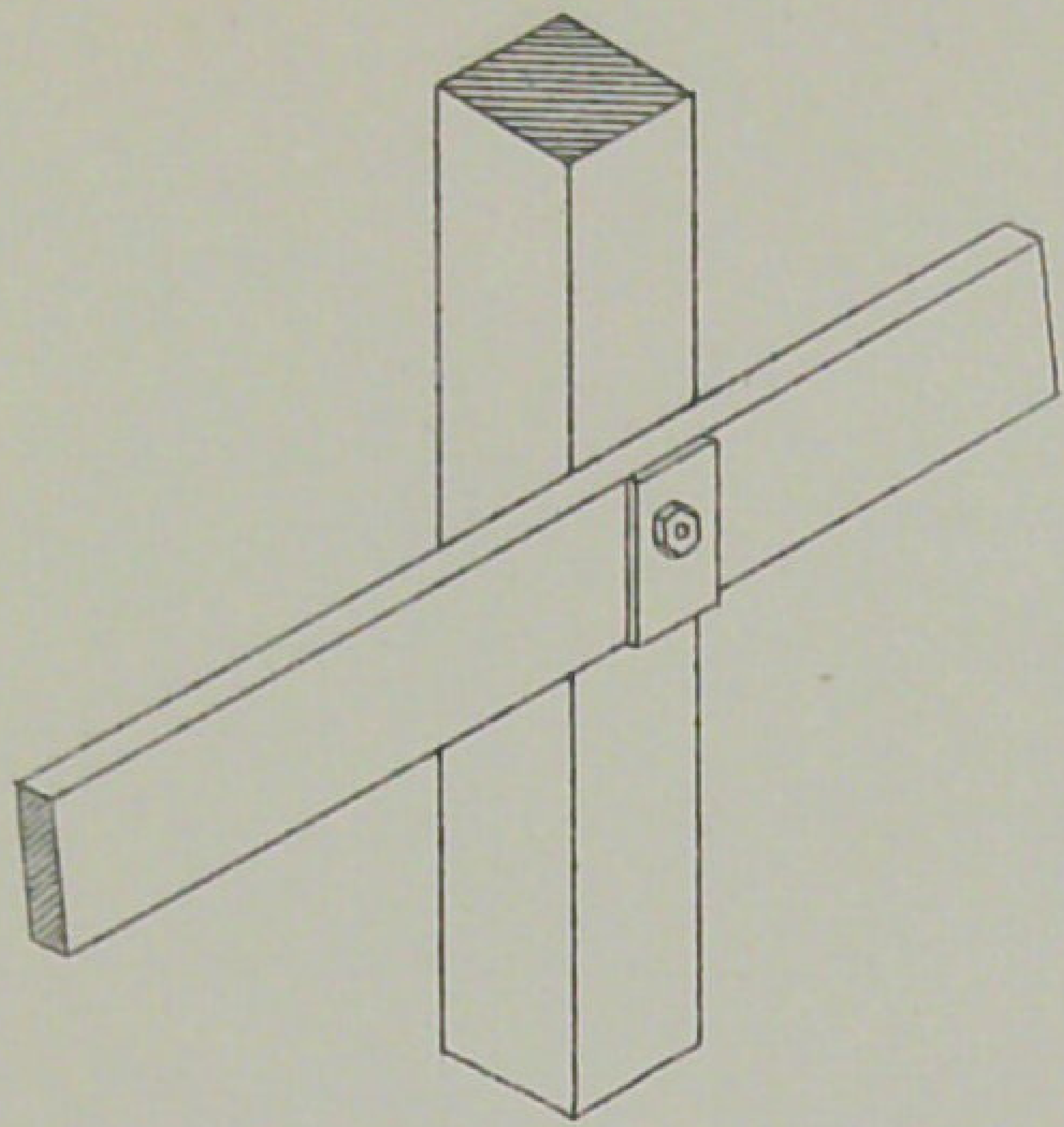


Fig. 11.

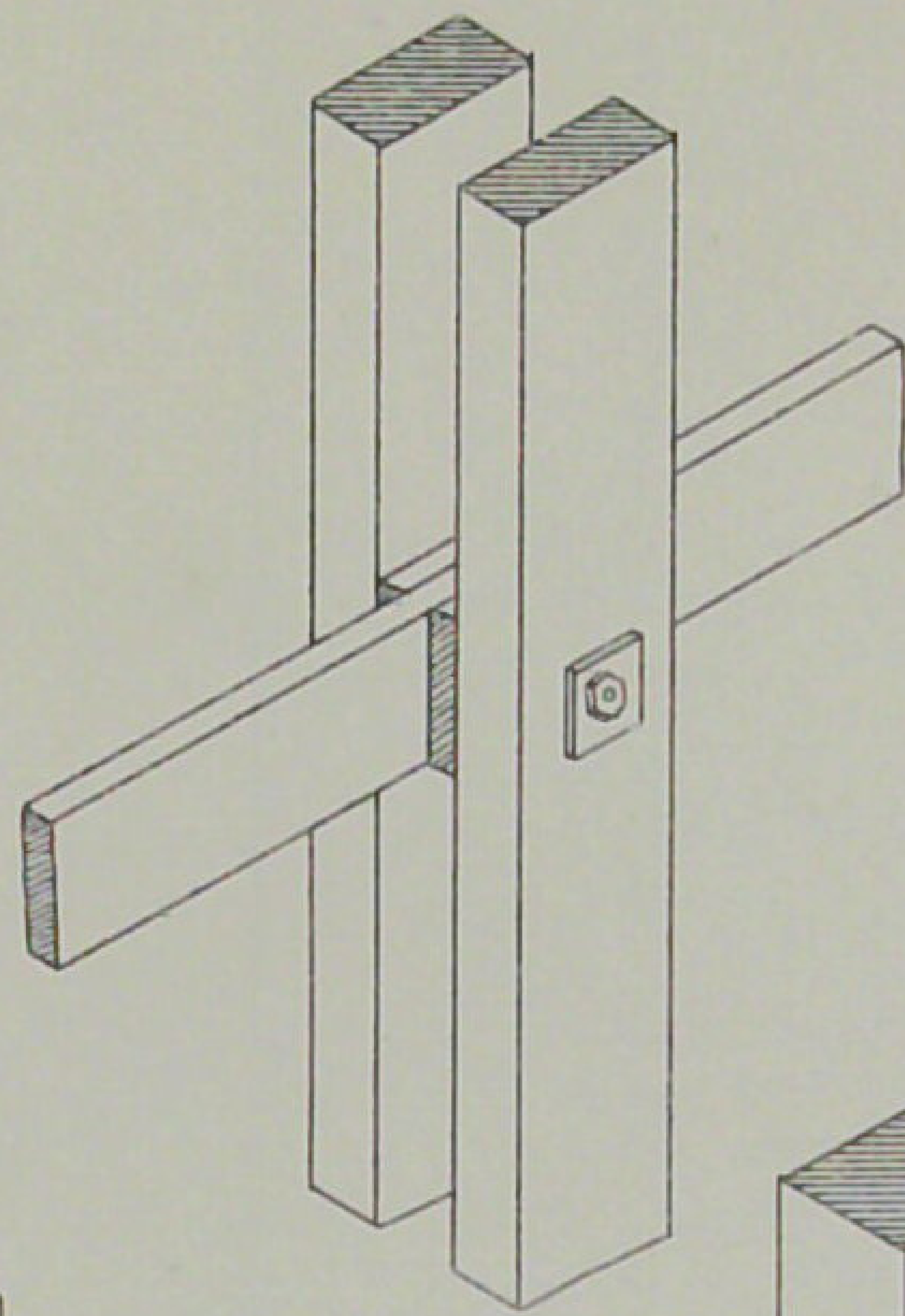


Fig. 12.

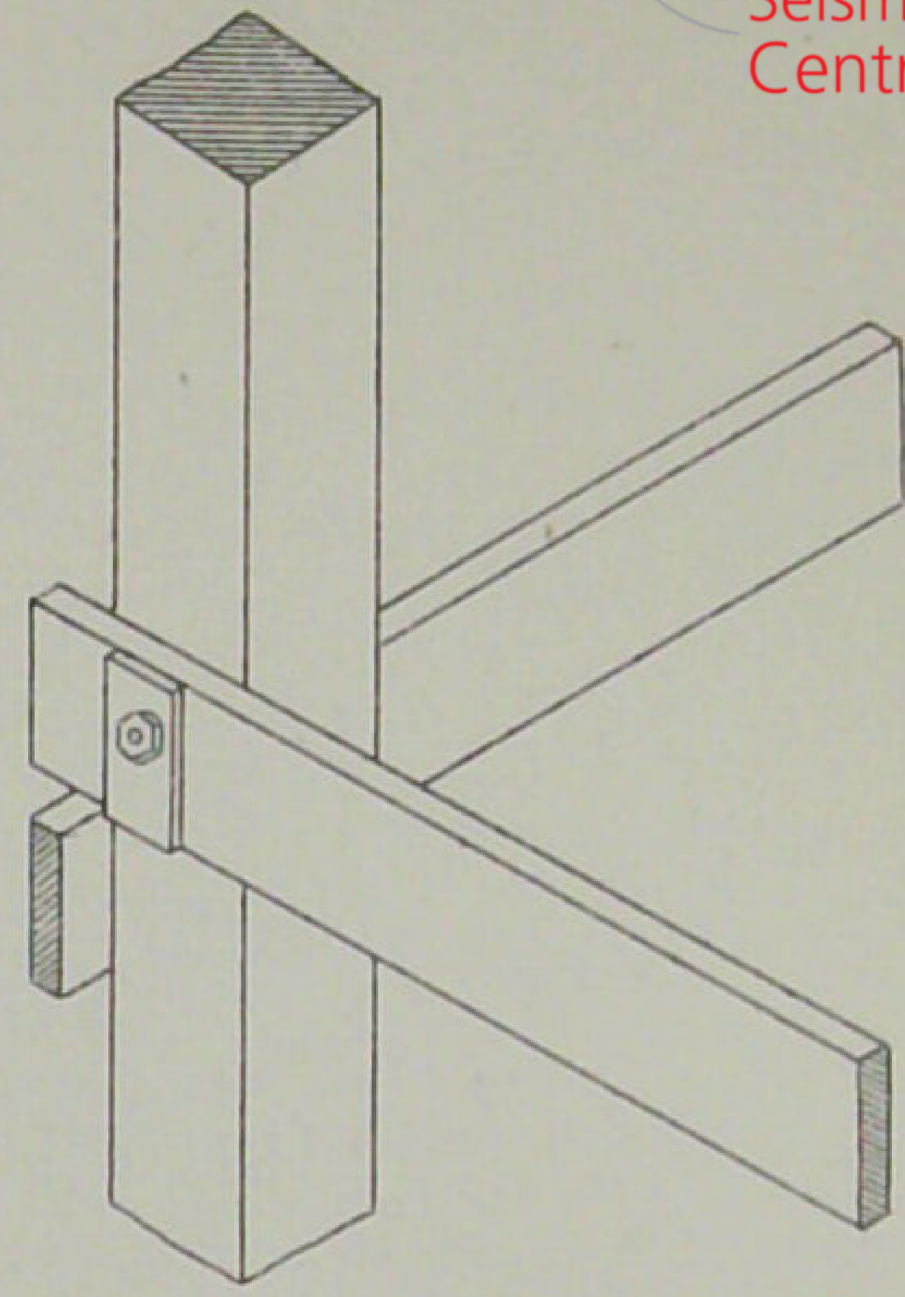


Fig. 10.

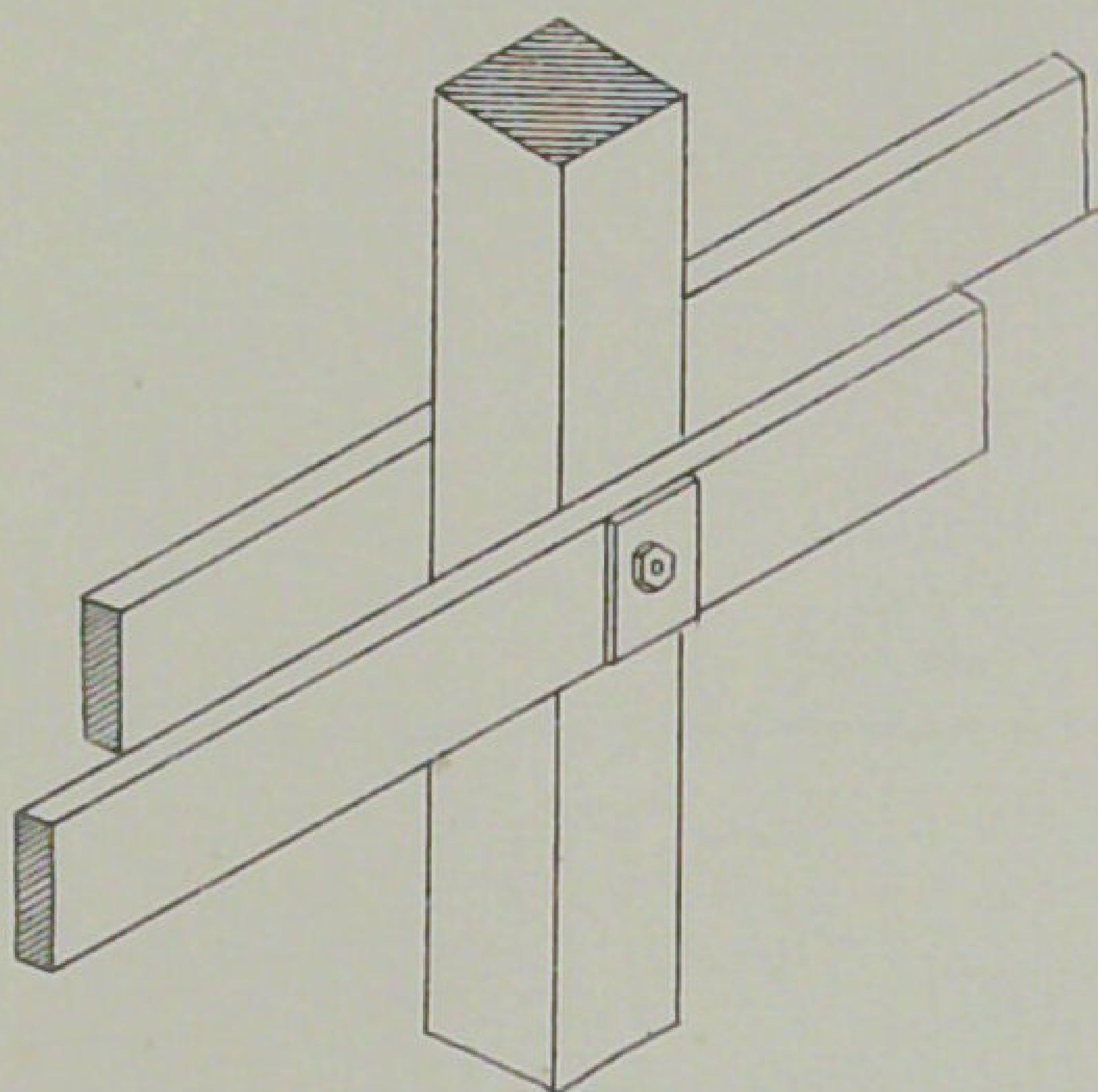


Fig. 13.

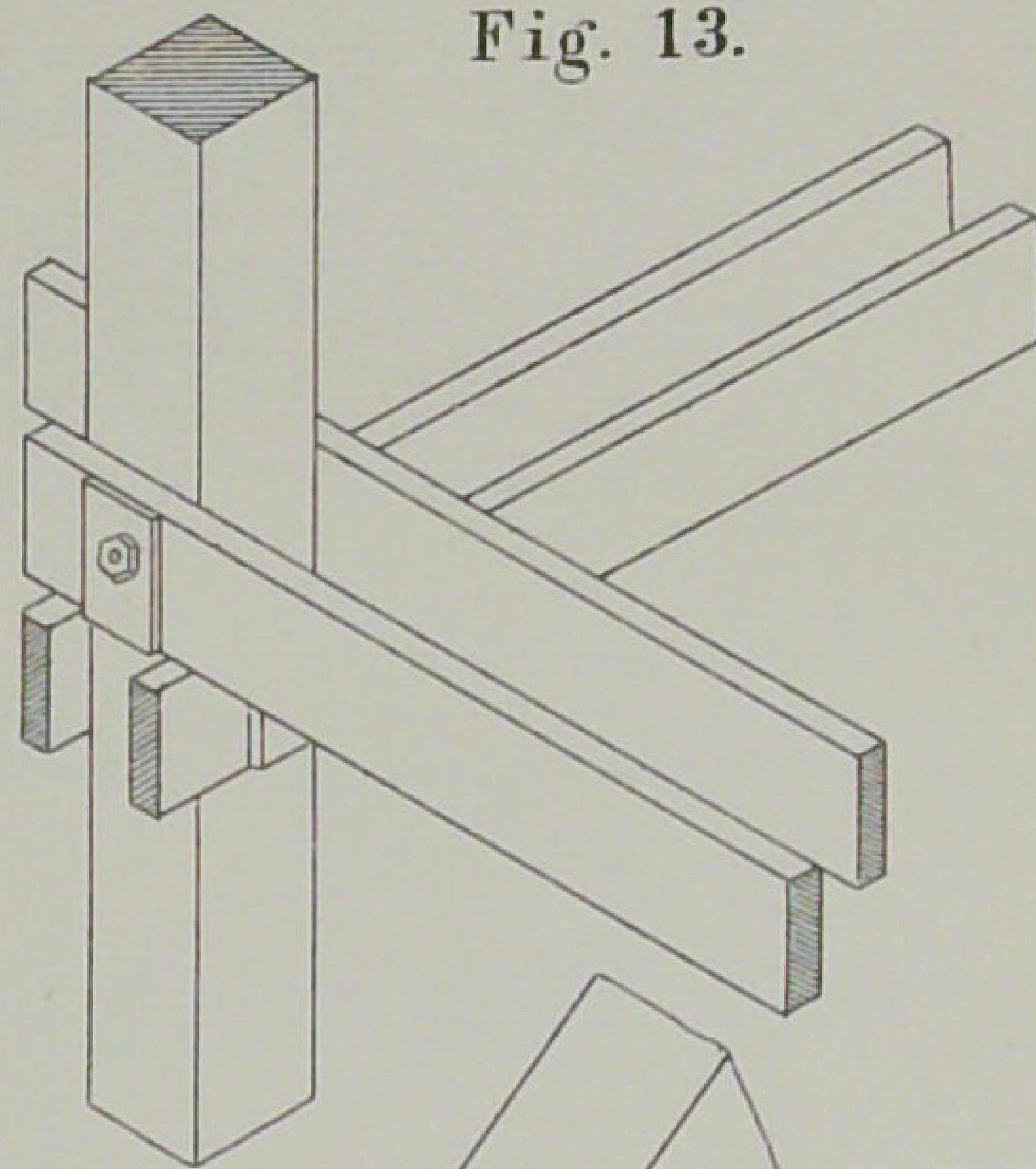


Fig. 14.

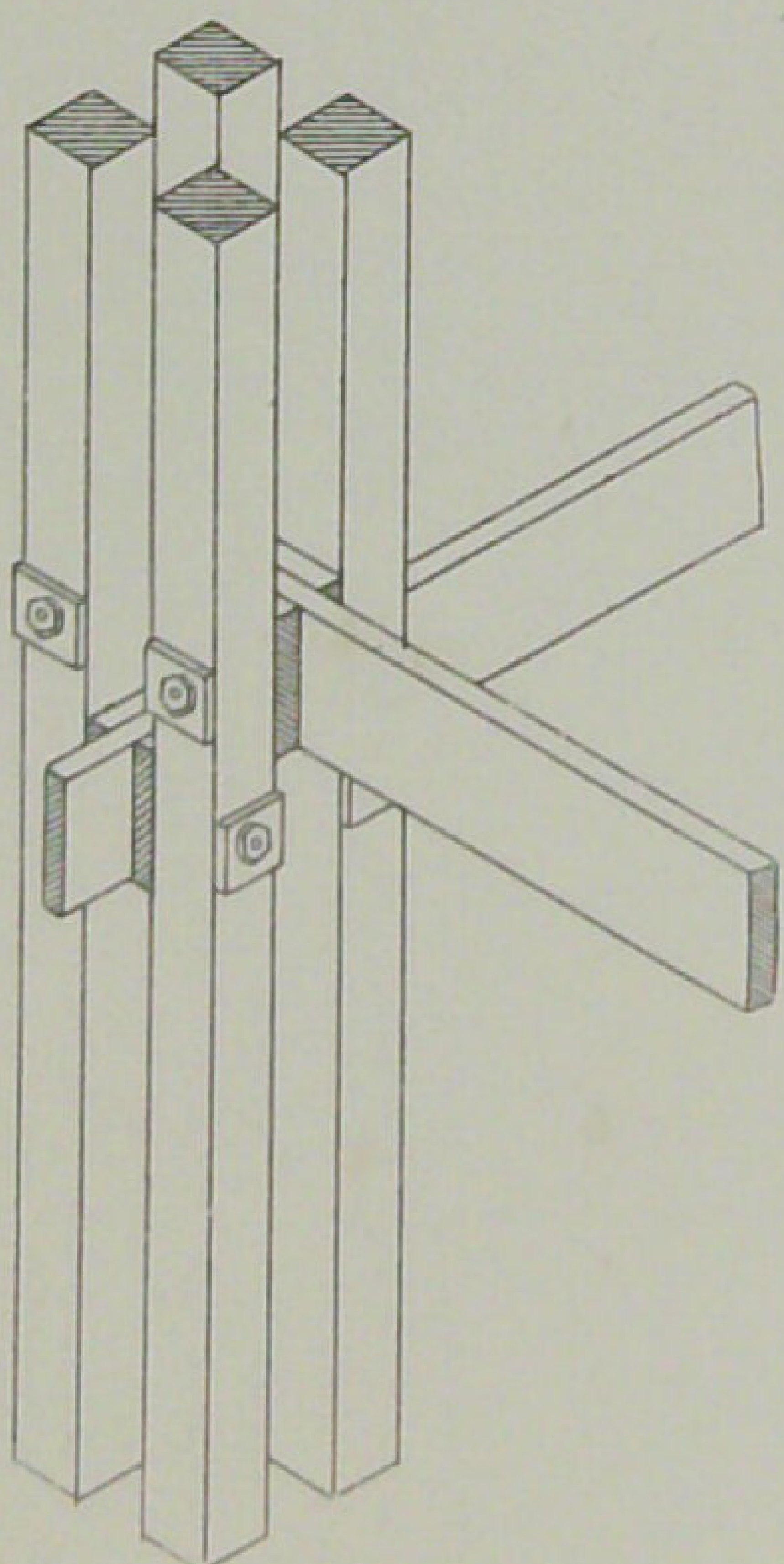


Fig. 15.

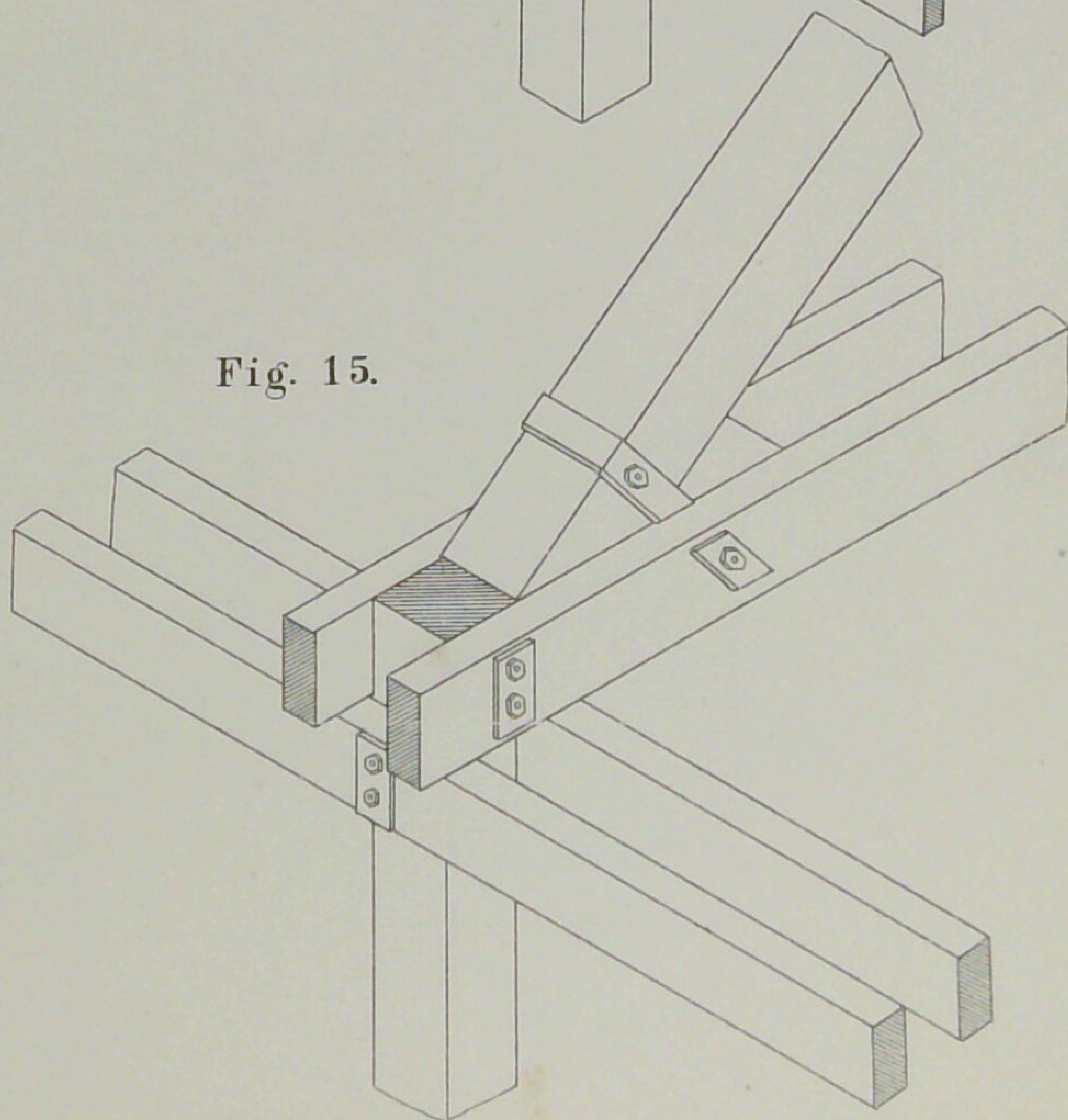


Fig. 16.

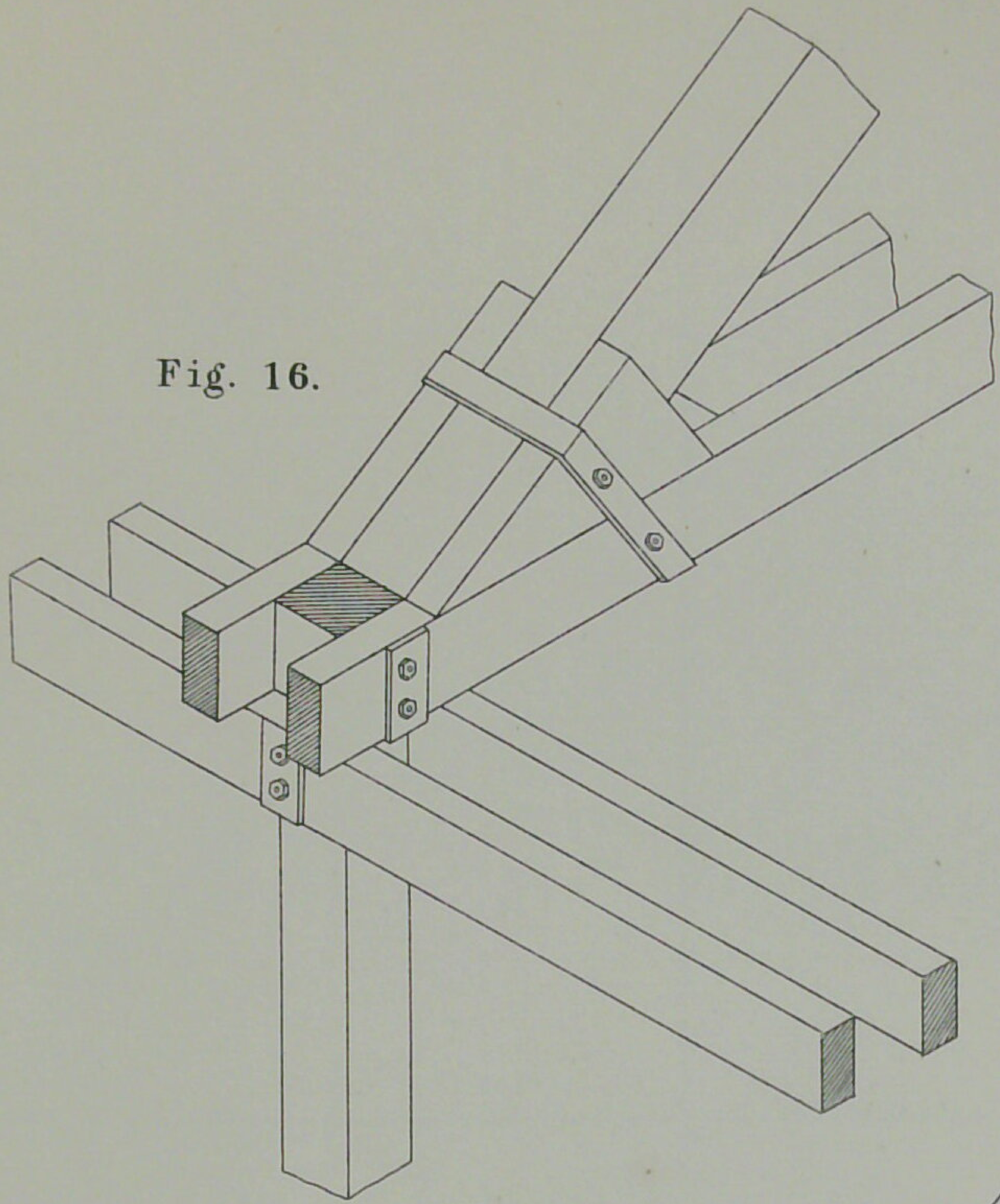


Fig. 17.

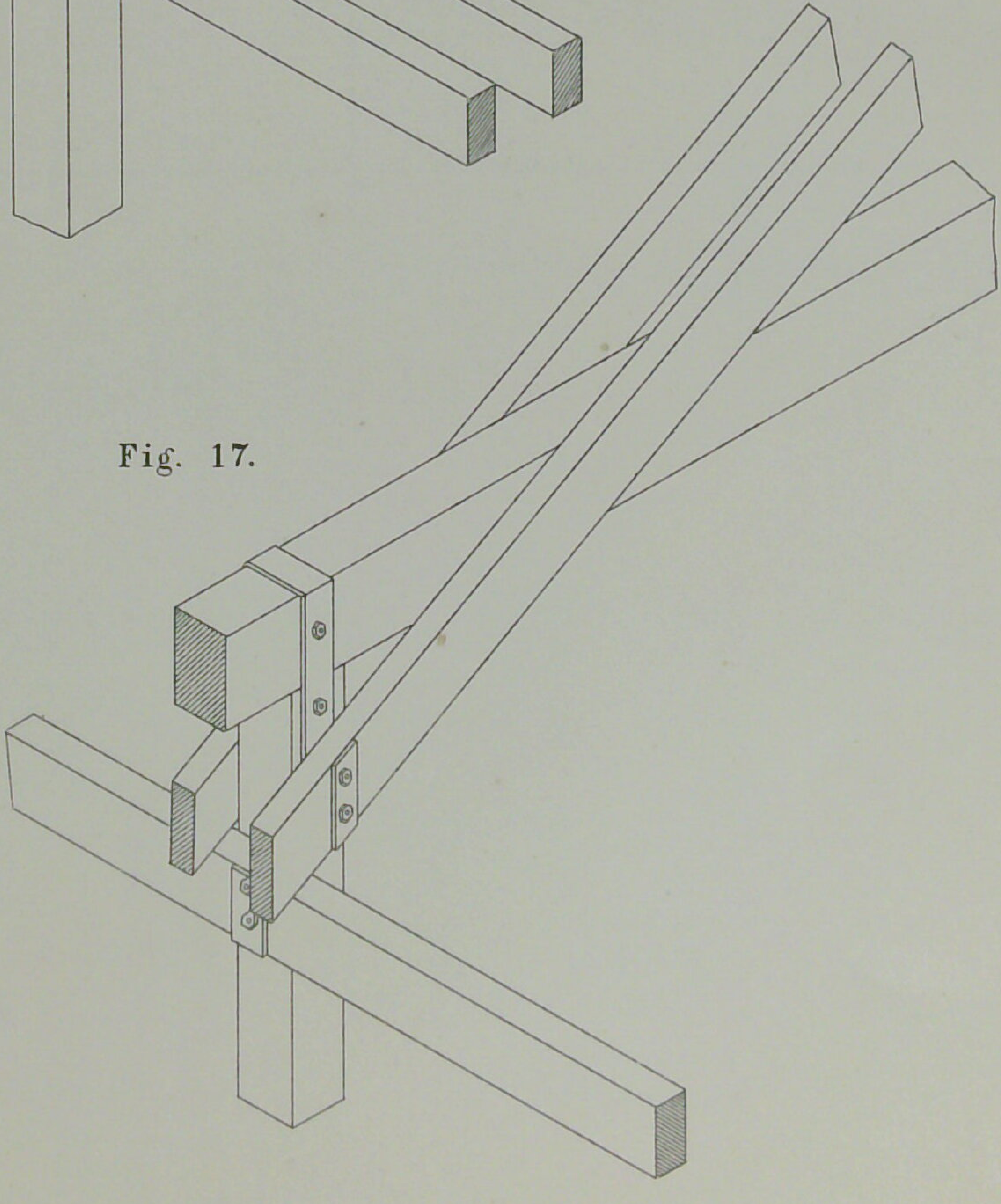


Fig. 18.

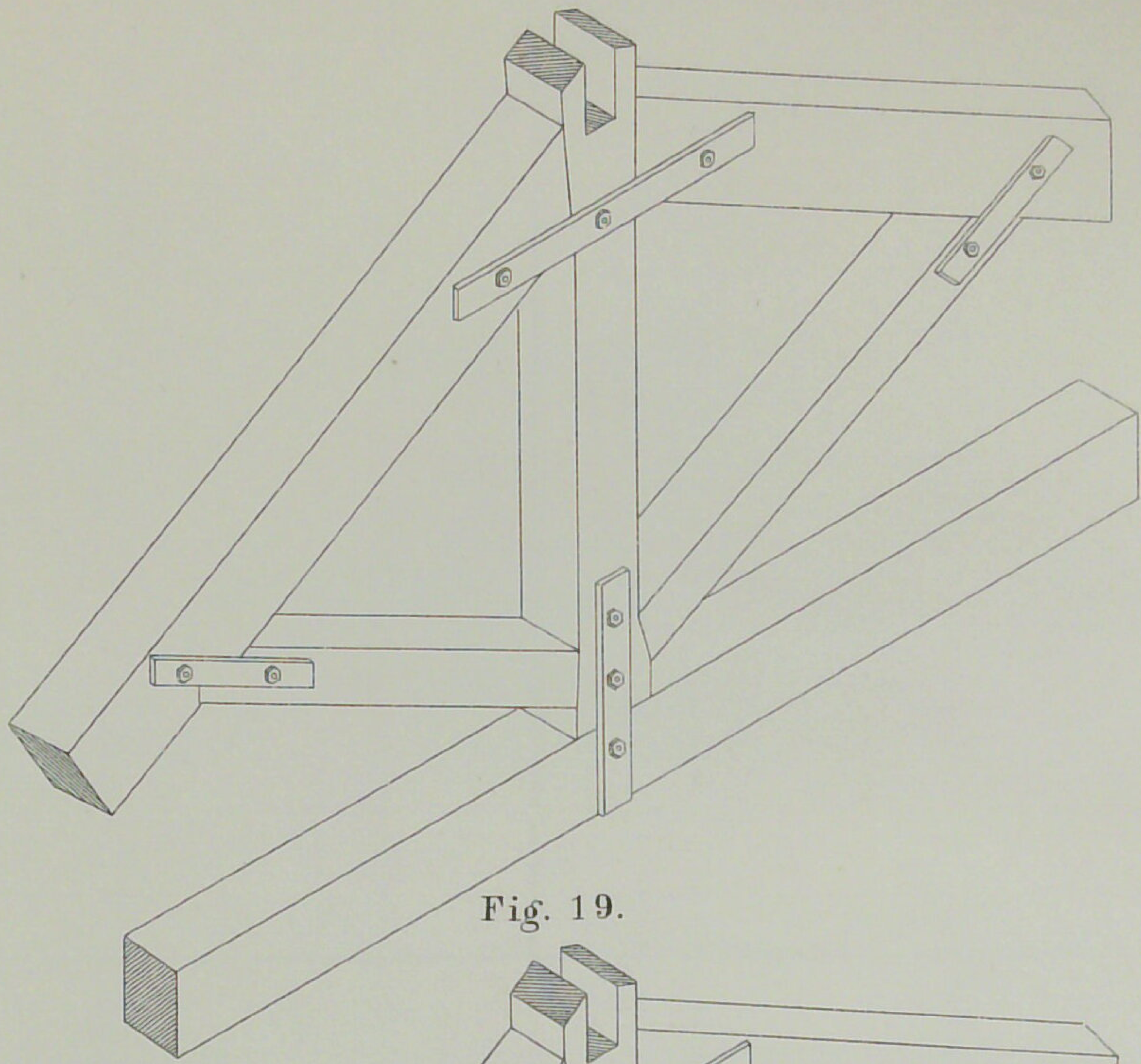


Fig. 19.

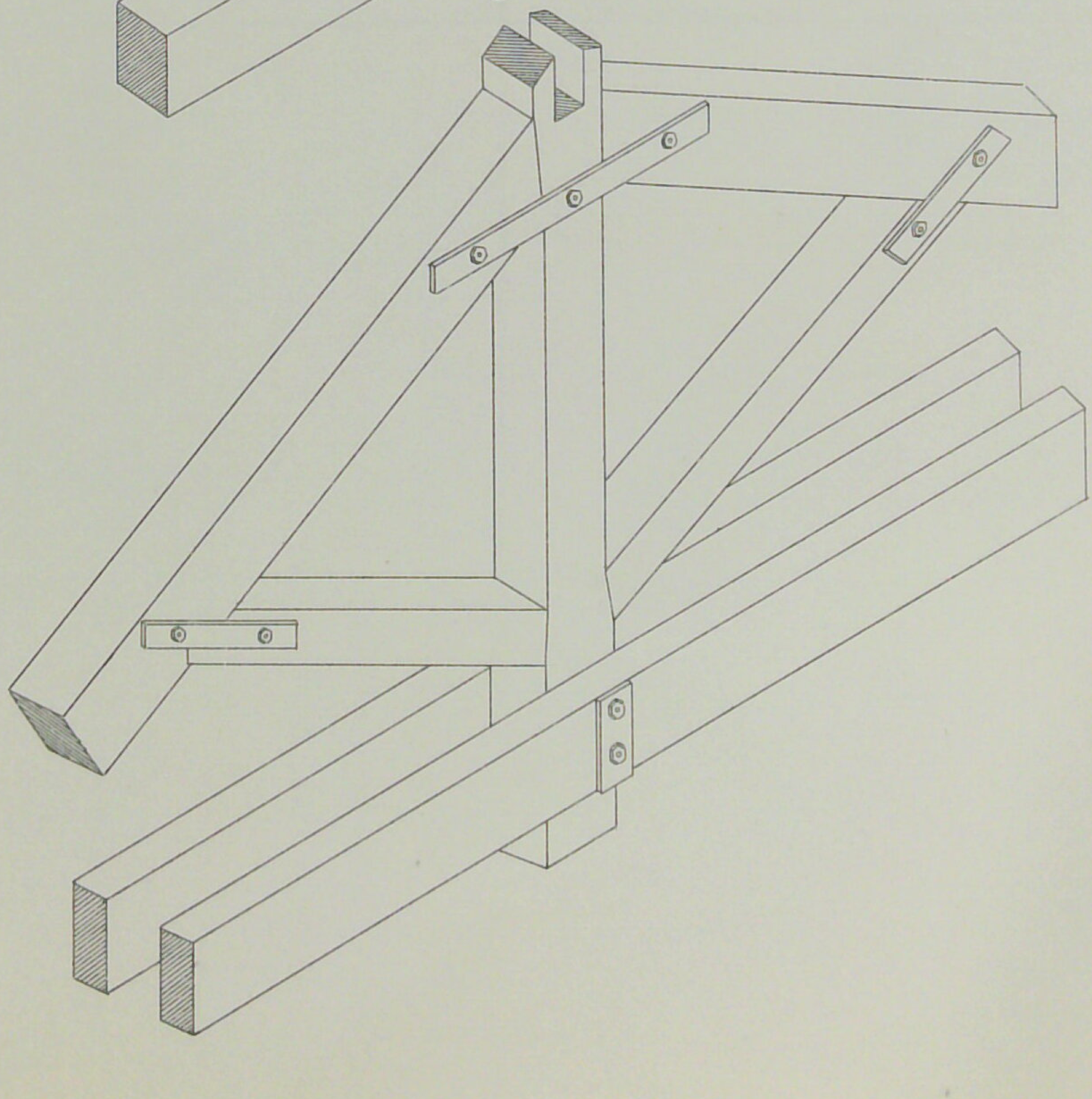


Fig. 20.

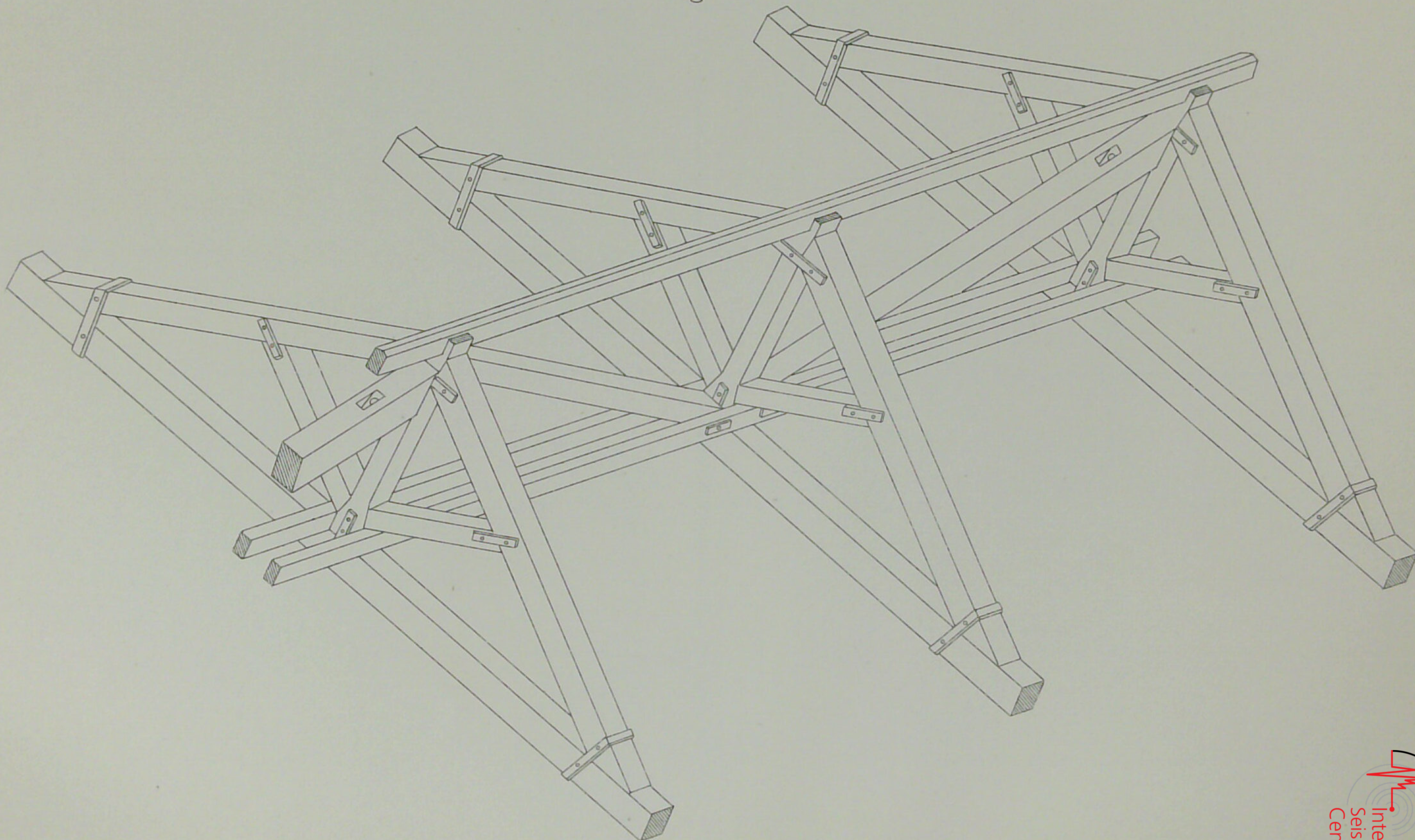


Fig. 21.

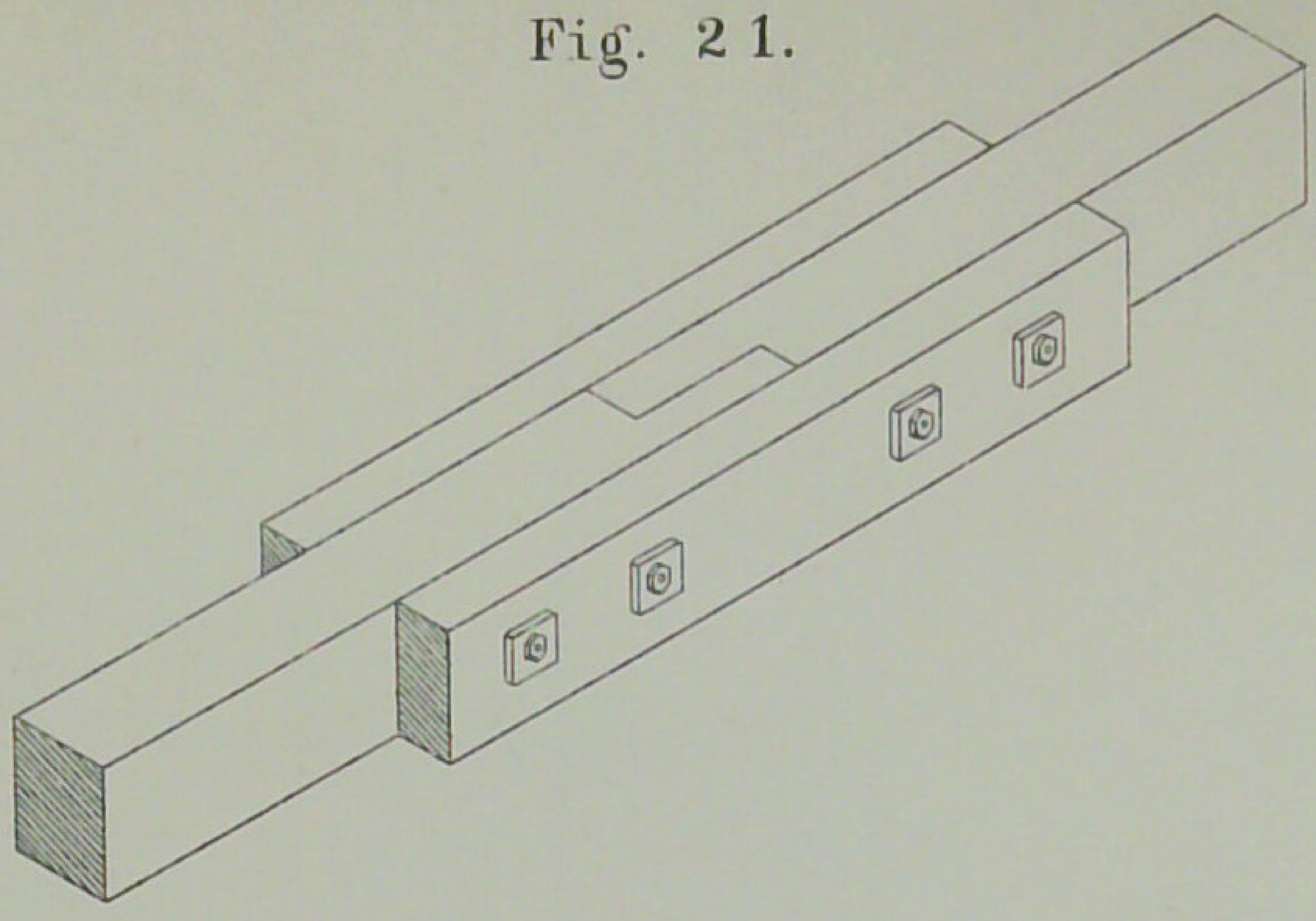


Fig. 22.

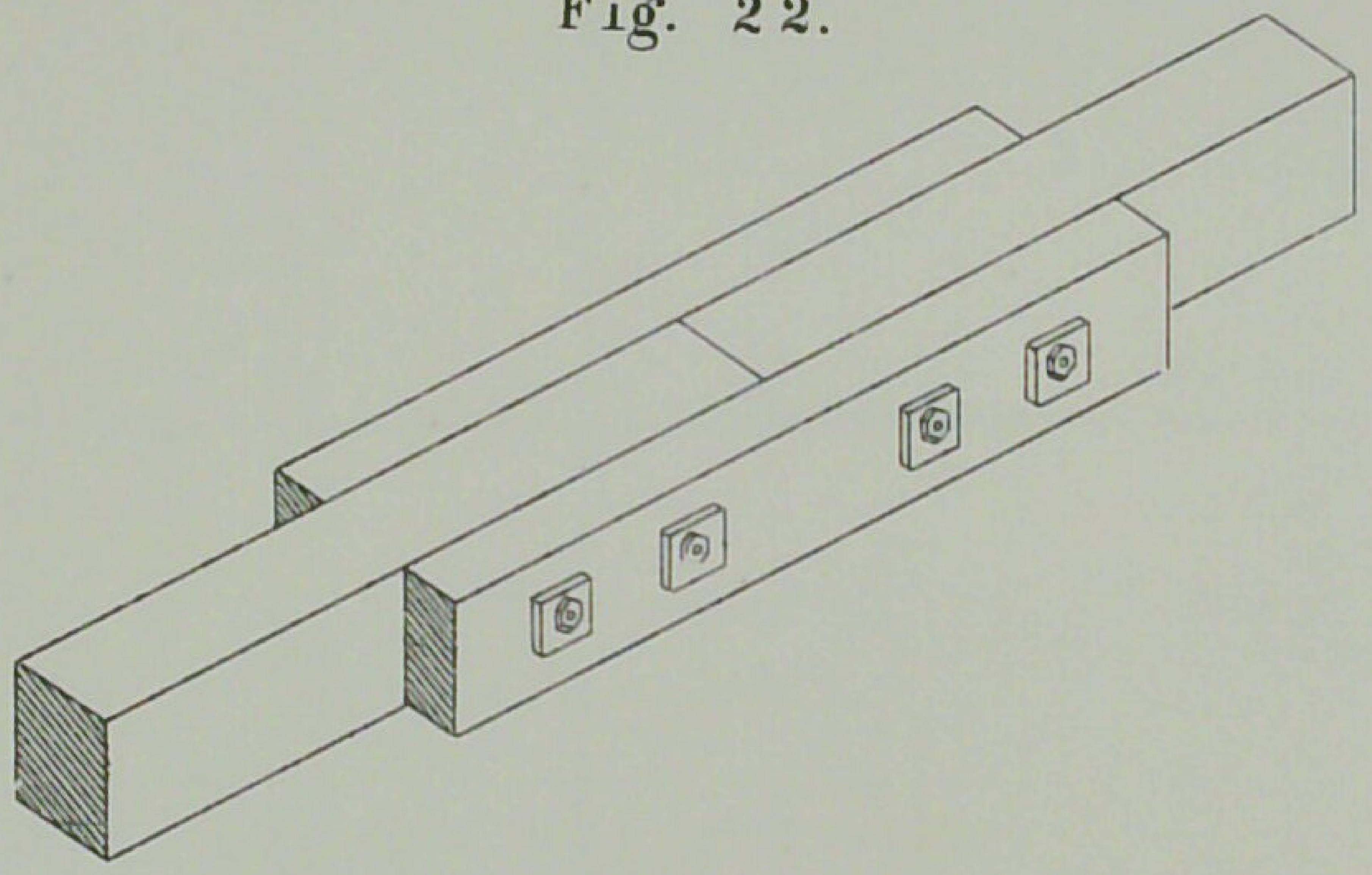
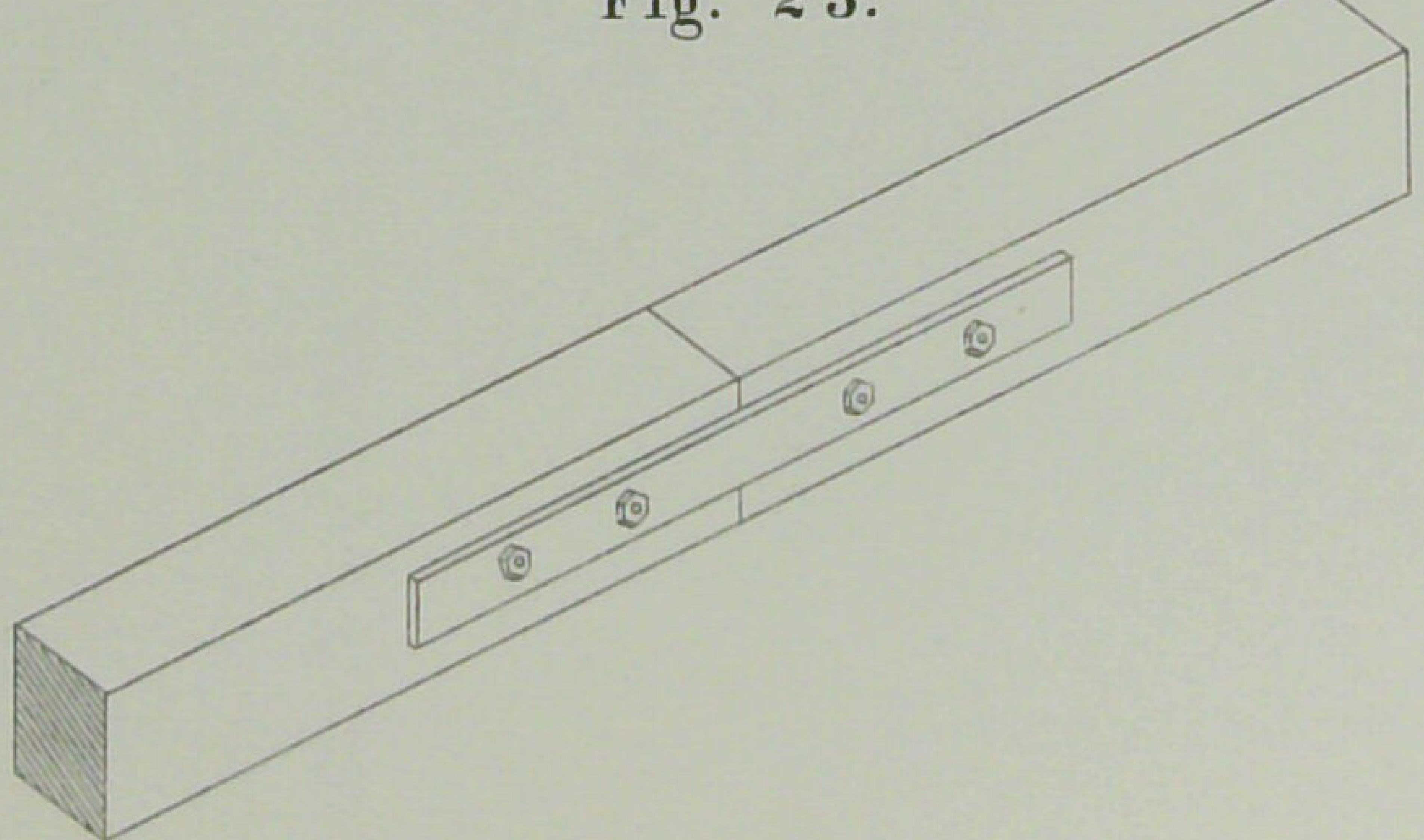


Fig. 23.



International Seismological Centre

Earthquake Measurement in a Brick Building.

BY

F. OMORI, D. Sc.

1.—The great amount of damage done to brick buildings by earthquakes is generally due to the breaking of their walls in consequence of the strong horizontal motion, and therefore it would be of interest to study the shaking during earthquakes of different portions of the walls. As an instance of investigations of this nature, I give in the present paper the result of a seismographical measurement made in the Engineering College of the Tokyo Imperial University, whose object was the comparison of the motion of a wall with that of the ground.*

2.—The wall chosen for the experiment was the external wall of the upper corridor of the western side of the Engineering College, and the motion was recorded by Prof. Ewing's Horizontal Pendulum Seismograph fixed near the top of the wall at the middle of its length. (See figs. 1 and 2.) Another seismograph of the same pattern was set up on the ground between the Engineering College and the Geological Institute, and its record was taken as representing the earthquake motion of the basement of the former, there being no particular topographical irregularity in the grounds, on which these buildings stand. Subsequently the ground-surface obser-

* Similar measurement with duplex pendulum seismographs have been made in 1883-4 by Prof. J. Milne. See his paper: *The movement produced in certain buildings by earthquakes.* Trans. Seis. Soc. Vol. XII.

vations were discontinued, and the records of the instrument in the Seismological Institute were substituted. All the seismographs multiplied the motion five times, the measurement having been restricted only to the horizontal component of the earthquake motion.

3.—The experiment was carried on between 1894 and 1898, during which period ten moderate earthquakes have successfully been recorded. The result is summarized in the following table.

SUMMARY OF RESULT.

Date of Earthquake.		Component.	Intensity.	Period. sec.	Max. Hor. Mot. On the ground. mm.	Max. Hor. Mot. Upstairs. mm.	Ratio. (Upstairs. / Ground)
May 10th, 1894		(EW)	Slight.	0. 4	0. 1	0. 2	2.0
June 25th, 1894		(NS)	Weak.	. 9	3. 7	4. 6	1.2
July 15th, 1894		(EW)	Slight.	......	. 6	. 3	.5
		(NS)		.52	. 6	. 3	.5
Nov. 15th, 1894		(EW)	Slight.	.23	. 2	. 3	1.8
		(NS)			.14	. 1	.7
Nov. 30th, 1894	Slow undulation	(EW)	Strong.	.....	2. 9	3.	1.0
		(NS)		.....	2. 9	3. 2	1.1
	Ripples	(EW)		.23	1. 4	2.	1.4
		(NS)		.22	1. 6	3. 2	2.0
April 9th, 1895		(EW)	Slight.	.22	.18	.16	.9
July 17th, 1895		(EW)	Slight.	. 5	. 5	. 7	1.4
		(NS)		.77	. 8	. 8	1.0
March 6th, 1896		(EW)	Weak.	.43	2. 2	2. 0	.9
		(NS)		.43	3. 2	4. 3	1.3
Oct 20th, 1897		(EW)	Slight.	.14	. 3	. 5	1.7
		(NS)		......	. 3	. 9	3.0
March 27th, 1898		(EW)	Slight.	.22	.14	. 8	5.7
		(NS)		.23	.24	. 8	3.3

To the above table the following explanatory notes may be added.

Eqke. No. 2. Intensity *weak*. The motion had a single well-pronounced maximum.

Eqke. No. 4. Intensity *slight*. The motion had no well-pronounced maximum.

Eqke. No. 5. Intensity *weak*. The duration was 80 seconds.

(Engineering College, upstairs.) The *preliminary tremor* consisted of vibrations of an average period of 0.18 sec. in the EW and 0.21 sec. in the NS direction and was followed abruptly by larger movements which were most active for the next 2.2 seconds, and whose average period was 0.23 second in the EW and 0.19 second in the NS direction. The maximum horizontal motion was 2 mm in the EW and 3.2 mm in the NS direction. Measuring at about 22 seconds from the commencement, the average period was found to be 0.22 second in each direction.

(Engineering College, ground surface.) The *principal portion* whose maximum motion was 1.4 mm in the EW and 1.6 mm in the NS direction had an average period of 0.22 second in the EW and 0.19 second in the NS direction. Measuring at about 23 seconds from the commencement, the average period was found to be 0.22 second in each component.

Eqke. No. 7. Intensity *weak*. The duration was 100 seconds.

(Seismological Institute.) The *preliminary tremor* was followed abruptly by the maximum motion. The average period of vibration in the principal portion was in the EW component 0.4 second.

Eqke. No. 8. Intensity *weak*. The duration was 90 seconds.

(Engineering College, upstairs.) The *preliminary tremor* was followed abruptly by a well-defined maximum movement, whose period was 0.36 second, and whose range was 2 mm in the EW and 4.1 mm in the NS direction, the rest of the shock consisting of far smaller vibrations. In the NS direction, the average periods of vibration in the *preliminary tremor*, *principal portion* and *end portion* were respectively 0.3, 0.4 and 0.46 second.

(Seismological Institute.) The maximum motion was 2.2 mm in the EW and 3.2 mm in the NS component, the average period in the *principal portion* being 0.41 second.

Eqke. No. 9. Intensity *slight*. The earthquake consisted of very quick vibrations, the average period being in the EW direction 0.14 second. (Seismological Institute.)

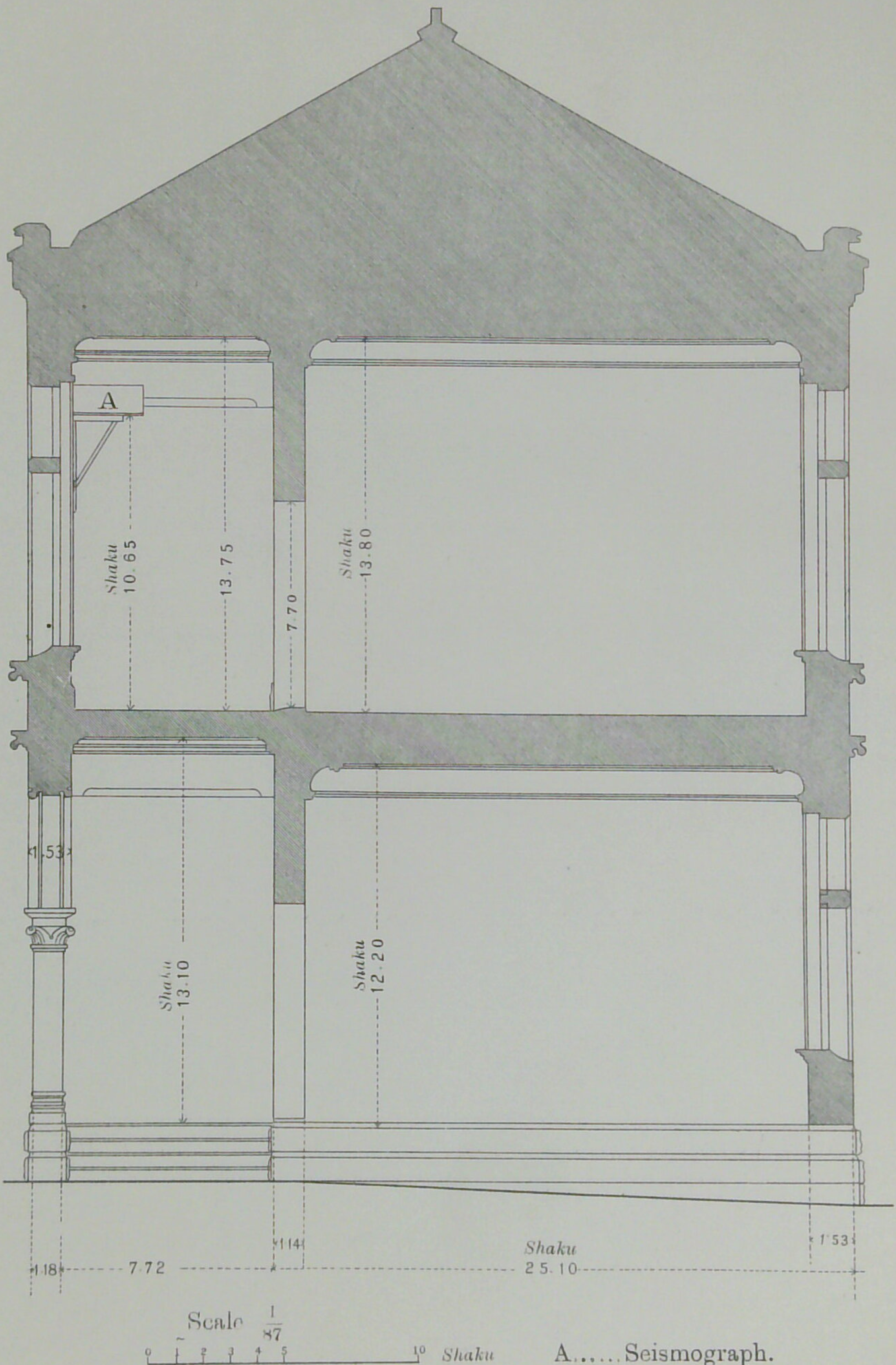
4.—As will be seen from the table, the motion of earthquakes Nos. 2, 3 and 7, which consisted of vibrations of comparatively slow period, that is to say, above 0.5 second, was practically the same in the upper storey of the Engineering College as on the ground surface, the mean ratio of the ranges of motion at these two stations being 1. On the other hand in the remaining seven earthquakes which consisted of quick-period vibrations, the motion of the top of the wall was greater in the average ratio of 2:1 than that of the ground surface. As besides, the period of vibration was the same in both cases, it seems that in shocks of violent nature the wall, on which the roof rested, behaved like an inverted pendulum subjected to a forced vibration, its motion synchronizing with the earthquake motion. Figs. 3a, 3b, 4a and 4b, give, as typical illustrations of this kind, the diagrams of earthquakes Nos. 5 and 8.

Practically, in cases of destructive earthquakes, the damage of two-storied brick buildings is in general limited to the upper storey.

Thus it is not seldom that the walls of the lower storey remain uninjured or very slightly cracked, even though the damage to the upper storey be so severe that its walls are knocked down and its roof fallen in. This is evidently due to the magnification of motion in the upper storey walls. Typical illustrations are given in figs. 5 and 6 which represent the condition after the great earthquake of Oct. 28th 1891, of the Aichi Cotton Mill and the Post and Telegraph Office, both at Nagoya.

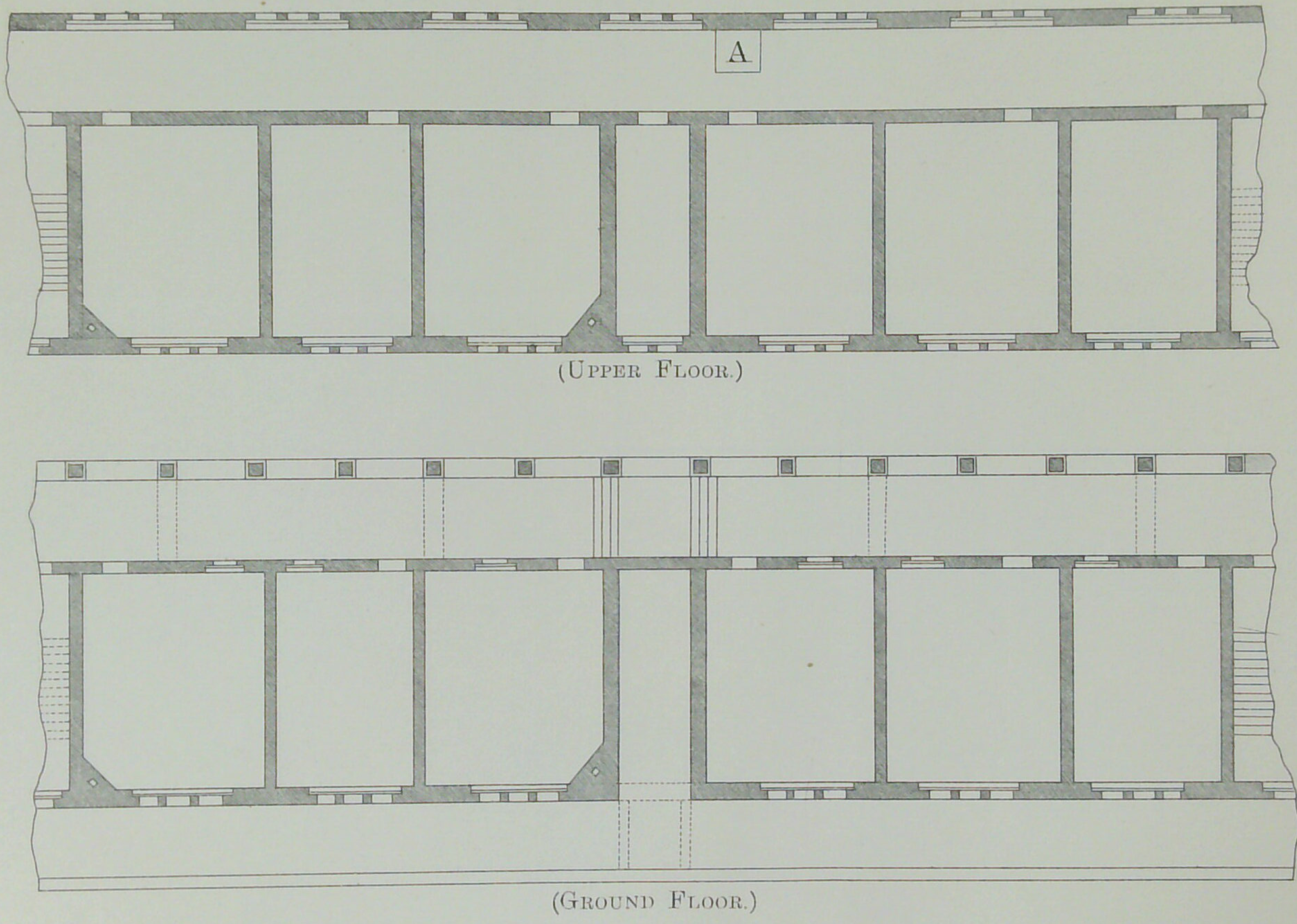
Fig. 1. WEST SIDE OF THE ENGINEERING COLLEGE.

(*Section.*)



A......Seismograph.

Fig. 2. WEST SIDE OF THE ENGINEERING COLLEGE.



Scale $\frac{1}{210}$.

A............Seismograph.

Fig. 6. POST AND TELEGRAPH OFFICE, NAGOYA. Oct. 28, 1891.

Note on the Great Mino-Owari Earthquake of Oct. 28th 1891.

BY

F. OMORI, D. Sc.

1.—The Mino-Owari earthquake of Oct. 28th 1891 was probably as violent as any ever recorded. In the meizoseismal area some of the towns and villages were almost entirely destroyed, and especially at many places in the western part of the province of Mino the number of people killed amounted to 4 °/₀ or 5 °/₀ of the population. The total number of people killed was 7000, while the number of dwelling houses entirely destroyed amounted to 80,000, so that one person was killed, on the average, for every 11 houses destroyed. The comparatively small number of the killed was doubtless due to the fact that common Japanese houses are built of wood.

The earthquake was felt in the whole southern and central part of Japan, the land area disturbed being about 250,000 sq. km. As the mean radius of propagation was about 520 km, the total area of disturbance was probably about 900,000 sq. km, or a little greater than double the area of the whole Empire.

In the present note, I limit myself to giving the result of my investigations on the intensity and direction of the motion at various places in the most strongly shaken districts.

2.—As the motion in the meizoseismal area was nowhere satisfactorily recorded by seismographs, I observed at various places a great number of well-formed stone lanterns and tomb-stones which were overturned, as well as those which were not overturned, and calculated the horizontal acceleration a necessary for overturning by the usual formula*

$$a = \frac{x\,g}{y}, \qquad (1)$$

in which g is the acceleration due to gravity, y the height of the centre of gravity, and x the horizontal distance between the latter and the edge about which the body is overturned.**

The values of the maximum acceleration estimated by this method are given in table I.

It is to be remarked that the calculation has been made on the assumption that the motion was entirely horizontal. This would produce no sensible error in the result except for places very near to the epifocus, where the vertical motion might have been considerable. To take an example, the estimated value of the maximum acceleration at Nagoya is 2600 mm per sec. per sec. Now according to the seismographic diagram of the Nagoya Meteorological Observatory, which gives the vertical and the horizontal motions respectively till the 10th and 15th seconds after the commencement of the earthquake, the ratio of the two components was as 1:3, the angle of emergence being thus about 20°. If we assume that the ratio of the principal vertical and horizontal movements had likewise been as 1:3, the correction to the value of the maximum acceleration, obtained on the above-stated supposition, would be ± 200 mm per sec. per sec.

The nature of the damage caused by this earthquake at different places in the strongly shaken area will be understood by comparing

* The introduction of this formula into seismology is due to Prof. C. D. West.

** See § 31 of the present author's paper: *Seismic experiments on the fracturing and overturning of columns.*

the list of the maximum accelerations (table I) with the *absolute scale* of destructive earthquakes given at the end of my paper on the fracturing and overturning of columns.

TABLE I.—LIST OF THE MAXIMUM ACCELERATIONS AT DIFFERENT PLACES IN THE STRONGLY SHAKEN AREA OF THE GREAT EARTHQUAKE OF OCT. 28TH 1891.

Province.	Place.	Maximum Acceleration.
		mm. per sec. per sec.
Mikawa.	Toyohashi	1700.
,,	Okazaki	900.
Ise.	Tsu	Less than 2000.
,,	Yokkaichi	Less than 1900.
,,	Kuwana	2000.
Owari.	Handa	Between 2000 and 2700.
,,	Tokonabe	Less than 2400.
,,	Nagoya, north-eastern part	2600.
,,	Nagoya, central part..	2600.
,,	Atsuta	Between 2300 and 3500.
,,	Nishi-Biwajima ...	3800.
,,	Jinmokuji	Less than 3900.
,,	Bamba	4100.
,,	Kanie	2700.
,,	Torigae	3000.
,,	Saya	Less than 4000.
,,	Tsushima	3000.
,,	Shobata	Less than 3500.
,,	Inaba...	4000.
,,	Ichinomiya	Between 2500 and 3500.
,,	Koori...	Greater than 2100.
,,	Iwakura	Greater than 4300.
,,	Komaki	Greater than 4300.
,,	Imaichiba	Greater than 2600.
,,	Utsutsu	2500.

Province.	Place.	Maximum Acceleration.
		mm. per sec. per sec.
Owari.	Ikeda	Greater than 2000.
,,	Inuyama	Between 2300 and 4000.
Mino.	Tokiguchi	Greater than 2000.
,,	Takayama	Greater than 1800.
,,	Mitake	1600.
,,	Dota	Less than 2200.
,,	Higashi-Katabira ...	Greater than 2400.
,,	Gifu	3000.
,,	Kasamatsu ...	4000.
,,	Takegahana	Greater than 4000.
,,	Nodera	Greater than 4000.
,,	Takasu	3000.
,,	Imao	Greater than 3200.
,,	Beppu	4000.
,,	Ōgaki	3000.
,,	Kochibara	Less than 1900.
,,	Kōmi	Greater than 2000.
,,	Kitagata	4000.
Ōmi.	Hikone	2700.
,,	Nagahama	2400.
Echizen.	Tsuruga	1200.
,,	Higashiura	1300.
,,	Takefu	1200.
,,	Sabae	1800.
,,	Midochi	2000.
,,	Asōzu	1100.
,,	Fukui	2500.
,,	Fujishima	1300.
,,	Katsuyama	1200.
,,	Ōno	Greater than 1200.
Yamashiro.	Kyoto	1000.
,,	Fukakusa	Greater than 1000.
Yamato.	Nara	2000.
Settsu.	Ōsaka	1000.

3.—*The Range of Motion at Nagoya.* Judging from the seismographical diagram obtained in the Nagoya Meteorological Observatory the complete period T of the principal motion there seems to have been about 1.3 seconds. From this as well as from the value of the maximum acceleration, the probable range of motion or double amplitude $2a$ at Nagoya may be estimated as follows :—

$$2a=\frac{\alpha T^2}{2\pi^2}=\frac{2600\times 1.3^2}{2\pi^2}=233 \text{ mm.}$$

The range of motion thus obtained is by no means too great * when we remember that the range of motion in Tokyo (Hongo) at the time of the far smaller earthquake of June 20th 1894 was 71 mm.

Ideas respecting the magnitude of the motion in the other places may similarly be obtained from the list of the maximum accelerations.

4.—*The Direction of Motion.* In the absence of satisfactory seismographic records, we may determine the direction of motion from observations of the overflowing of liquids, cracks in the ground, rotation of bodies, etc. The best result however is to be obtained from observations of overturned bodies.

It is unnecessary to remark that in direction observations we must choose those cases, in which the direction of falling would not be influenced by the form of the body such as stone lanterns and tomb stones of square or circuler section. It is necessary to multiply the number of the observations and take the mean.

The direction of the overturned bodies in the meizoseismal area was at some places irregular, but at others very uniform.

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* This is a slight modification of the conclusion given in my *note on the overturning of columns.* Vol. II. Seis. Jour. Japan.

As an example of the latter case, I shall describe the direction observations in Nagoya.

The brick buildings of the Nagoya Post and Telegraph Office and two bazaars had their walls parallel NS completely knocked down, while those parallel EW were only cracked. From this, the direction of the earthquake motion at Nagoya may be inferred to have been nearly EW. Besides this, I observed in different parts of the city 200 stone lanterns with cylindrical stems, whose directions of overturning are given in table II. Of these, 15 were overturned towards W, 119 between W and S, 3 towards S, 10 between S and E, 6 towards E, 36 between E and N, 1 towards N, and 10 between N and W. The maximum number of columns was thus overthrown towards WSW, and the next greatest number towards the opposite direction ENE, the mean direction being S 60° W and N 60° E. Table II is graphically illustrated by fig. 1, where each mark (×) indicates a stone lantern overturned in the proper radial direction. When more than one were overturned in one and the same direction, an equal number of the marks has been put on the corresponding radius.

The tendency to be overturned towards W was likewise ascertained to be the case with other bodies in Nagoya. Thus, twenty-seven stone lanterns and tomb stones of square section, with sides parallel to the cardinal directions, which were observed in different parts of the city were overturned as follows:—2 towards E, 21 towards W, 1 towards S, and 3 towards NE. Further, the tops of the chimneys of the Electric Light Company, the Aichi Cotton Mill and the Cement Factory were all projected towards WNW.

Similarly in Gifu, the principal direction of overturning was WSW. Here the four chimneys in the Prison House and the Kasōjō, as well as the anemometer observatory of the Meteorological Station were all overthrown nearly towards W. Again, the brick court-walls of the Prison House and the mud walls of the Nishi-Betsuin, which were parallel NS, were overturned towards W, while those parallel EW remained uninjured.

TABLE II.—DIRECTION OBSERVATION IN NAGOYA.

(DIRECTIONS TOWARDS WHICH 200 STONE LANTERNS WITH CYLINDRICAL STEMS WERE OVERTURNED).

S 50° W	S 30° W	S 27° W	S 55° W
S 40° W	S 15° W	S 60° W	N 80° E
S 70° E	S 60° W	S 60° W	E
N 70° E	S 55° W	S 58° W	S 15° E
W	S 40° W	S 60° W	S 63° W
N 70° E	W	S	S 40° W
S 50° W	S 70° W	S 65° W	S 73° W
S 65° W	N 50° W	N 73° E	N 56° E
N 65° W	S 70° W	S 52° W	N 60° E
E	N 85° E	S 35° W	S 55° W
N 50° W	S	N 45° E	S 55° W
S 40° E	S 40° W	S 50° W	N 55° W
N 40° E	S 40° W	S 33° W	S 55° W
N 10° W	S 20° W	S 42° W	S 30° W
S 55° W	S 40° W	S 42° W	S 73° W
N 70° E	S 57° W	N 70° E	S 65° W
S 60° W	N 65° E	S 30° W	S 65° W
S 80° W	N 50° E	W	S 72° W

S 60° E	W	N 72° E	S 85° W
N 80° E	N 50° E	S 57° W	S 80° W
N 50° W	N 40° E	S 60° W	S 80° W
W	S	S 60° W	S 70° W
W	N 40° E	N 68° E	N 35° W
W	S 75° W	N 55° E	N
S 65° W	N 20° E	S 65° W	N 47° E
N 60° W	S 85° W	N 65° E	S 83° W
S 70° W	N 60° E	S 75° W	S 70° W
S 40° W	S 30° W	S 68° W	N 50° E
S 40° W	S 10° W	S 74° W	S 82° W
S 60° W	S 50° W	W	N 10° E
N 75° E	S 40° W	S 70° W	S 60° W
S 45° W	S 40° W	S 50° W	S 60° W
S 75° W	S 65° W	S 70° W	S 25° W
W	S 85° E	S 80° W	N 60° E
S 72° W	N 25° E	W	S 65° W
S 55° W	S 70° W	S 55° W	S 70° W
S 40° W	N 70° W	N 45° E	S 30° W
S 35° W	S 65° W	W	S 40° W
S 32° W	S 70° W	N 45° E	S 20° E
S 35° W	S 35° W	S 55° W	S 45° W
S 45° W	S 68° W	S 40° W	N 65° W
S 85° W	S 40° E	S 30° W	S 60° W
S 60° W	S 77° E	S 35° W	E
S 57° W	S 55° W	N 40° E	S 40° E
S 55° W	W	E	W
S 50° W	W	N 85° E	W
S 55° W	S 80° E	N 60° E	E
S 40° W	S 65° W	S 60° W	
S 70° W	E	S 80° W	
S 55° W	S 70° W	N 55° E	
S 62° W	S 70° W	S 60° W	

In table III are given the direction of motion at various places in the strongly shaken area, deduced from my observations of numerous overturned stone lanterns and tomb-stones.

TABLE III.—LIST OF THE DIRECTIONS OF MOTION AT VARIOUS PLACES IN THE STRONGLY SHAKEN AREA OF THE GREAT EARTHQUAKE OF OCT. 28TH 1891.

Province.	Place.	Direction of motion.
Mikawa.	Toyohashi...	S 80° E → N 80° W
Ise.	Tsu	N 75° W → S 75° E
,,	Yokkaichi	WNW → ESE
,,	Kuwana	S 75° W → N 75° E
Owari.	Tokonabe	W → E
,,	Handa	NW → SE
,,	Atsuta	ENE → WSW
,,	Nagoya	N 60° E → S 60° W
,,	Bamba	ENE — WSW
,,	Tsushima	E — W
,,	Jinmokuji...	E — W
,,	Shimo-otai	S — N
,,	Komaki	WSW — ENE
,,	Iwakura	SWS → NEN
,,	Koöri...	SW — NE
,,	Inuyama	S 10° W → N 10° E
,,	Ichinomiya	WNW → ESE
,,	Ikeda...	S 20° W — N 20° E
,,	Utsutsu	N 30° E → S 30° W
Mino.	Kasamatsu	ESE → WNW
,,	Gifu	ENE → WSW
,,	Ōgaki	SWS — NEN
,,	Kitagata	E → W
,,	Kurono	S 50° W — N 50° E

Province.	Place.	Direction of motion.
Mino.	Monju	S 60° W — N 60° E
,,	Higashi-katabira ...	S — N
,,	Dota	S 20° W — N 20° E
,,	Tokiguchi...	N → S
,,	Tajimi	SWS — NEN
Echizen.	Ōno	W → E
,,	Katsuyama	S — N
,,	Fujishima...	S — N
,,	Fukui	SWS → NEN
,,	Asōzu	S 30° W → N 30° E
,,	Takefu	S — N
,,	Tsuruga	NWN → SES
Ōmi.	Nagahama	N → S
,,	Hikone	NWN — SES
Yamashiro.	Kyoto	SWS — NEN
,,	Inari	N 15° W — S 15° E
,,	Fukakusa	S — N
,,	Fushimi	SES — NWN
Yamato.	Nara	NEN → SWS

5.—*Relation between the Intensity and the Direction of Motion.* The direction of motion and the variation of the intensity in the three most strongly shaken provinces of Mino, Owari and Echizen will be seen from fig. 2, which embodies the results contained in tables I and III and my other observations in the mountain districts of Mino and Echizen.

The zone of extreme violence of the earthquake motion, shaded in the figure, begins, on the north, at the villages of Nukumi and Kumanoko in Echizen, is continued to the valleys of Oppa and

Okawara in the Motosu county of Mino, then runs in a SES direction along the famous Neo-valley till it reaches the tract about the town of Takadomi where it is divided into two branches. The principal course thence passes between Gifu and Ogaki, is continued to the towns of Komaki and Iwakura, and ends in the tract between Nagoya and Tsushima. The other branch is continued from the vicinity of Takadomi to the village of Higashi-Katabira in the Kani-county of Mino.

The two curves, marked 1 and 2, are the isoseismal lines, along which the maximum accelerations of the earthquake motion were respectively 2000 and 800 mm per sec. per sec. It will be observed that the shaded zone forms the axis of the isoseismal line 2, whose general direction is NWN and SES.

The directions of motion at different towns and villages are indicated by short red lines, the arrow indicating the direction towards which the majority of the bodies at a given place were overturned. As already mentioned, the principal directions of overturning at Nagoya and Gifu were respectively S 60° W and WSW. At the towns of Ichinomiya and Kasamatsu situated between the two above-named cities, the mean direction of motion was ESE and WNW, greater numbers of bodies having been overturned towards W than towards E. At the villages of Monju and Kurono, to the NW of Gifu, the principal direction of the overturning was SW, while that at Kitagata was W. Similarly, at Atsuta which forms the southern continuation of Nagoya, the direction in question was WSW. At the town of Toyohashi, in the province of Mikawa, the direction of motion was S 80° E and N 80° W, more bodies having been overturned towards W than towards E. It is here to be noticed that all these places lie on the eastern side of the meizoseismal zone. Turning next our attention to the places on its western side, we see that the direction of motion at the town of Tsushima, as well as in the counties of Kaisei (Owari) and Kuwana (Ise) was nearly E and W, more bodies having been overturned towards E than towards W. The directions of motion at the towns of Kuwana and Tsu were very uniform and respectively N 75° E—S 75° W and S 75° E—N 75° W, a far greater number of bodies having in each case been overturned towards E than towards W. Further, one chimney at Kuwana, two at Yokkaichi, and one at Tsu, had their tops thrown down all nearly towards E, that is, respectively towards SE, SE, SES and N 70° E. At the towns of Tokonabe and Handa in the Chita Peninsula, the direction was nearly EW, more having been thrown towards E than towards W.

It thus appears that in the Mino-Owari plane and along the Ise Sea and the Mikawa Bay, the direction of the earthquake motion was approximately normal to, and directed towards, the meizoseismal zone ; in other words, the principal displacement was at the places on the east side of the epifocus directed towards W ; and at places on its west side towards E, the earthquake motion converging, as it were, towards the origin. (Iwakura and Shimo-Otai, Owari, where the direction of motion was nearly NS, form exceptions to the above conclusion. It is however to be observed that the intensity of motion at these two places was also abnormally greater than that at the neighbouring places.) Again, the direction of motion at Inuyama, Utsutsu, Ikeda, Takayama, Tajimi, Dota and Higashi-Katabira, was NS or NEN-SWS, and was therefore approximately normal to the course of the branch zone which runs from the vicinity of Takadomi to the Kani-county, and along which all these places are situated.

Note on the Tokyo Earthquake of June 20th 1894.

BY

F. OMORI, D. Sc.

1.—The earthquake of June 20th 1894 is the most violent that has shaken Tokyo since the great catastrophe of the 2nd year of the Ansei period (1855). The land area in which the motion was sufficiently strong to be felt without instrumental aid was about 110,000 sq. km, the mean radius of propagation being about 300 km. In Tokyo many brick buildings were severely damaged, chimneys in particular having been mostly thrown down; some *dozo* (godowns) had their plastered mud walls very much cracked and shaken down, tomb-stones and *ishidoro* (stone lanterns for gardens) were overturned, small cracks were formed in the ground, and in a few cases water was ejected. The number of casualties in the three Prefectures of Tokyo, Kanagawa and Saitama were 26 persons killed and 171 wounded.

2.—The maximum acceleration of the earthquake motion in the low and soft-ground portions of Tokyo, such as Tsukiji, Honjō, Fukagawa and Shiba, was, judging from the bodies overturned, about 1000 mm per sec. per sec., which is equal to $\frac{10}{26}$th of the intensity at Nagoya of the great Mino-Owari earthquake of 1891, and a little less than that at the epicentres of the earthquakes of Kumamoto of 1889, and of Kagoshima of 1893. The radii of propagation of the three last earthquakes were respectively 520, 180 and 160 km.

The intensity of motion in the high and hard-ground portions of Tokyo was about half of that in the districts already named.

3.—The meizoseismal tract, or the epifocus, was a zone which extends in nearly NS direction from the vicinity of the town of Iwatsuki to the eastern part of Tokyo. The earthquake origin thus seems to have been formed under the lowest part of the Musashi plane, which forms the continuation of the axis of Tokyo Bay, and has apparently no connection with the famous fault line, extending from the northern part of the province of Awa to the Miura Peninsula of the province of Sagami.

The earthquake was a little milder at Yokohama than in Tokyo, the intensity becoming still weaker towards the south at Kamakura, where no particular damage was produced. Towards the north, the shock was very strong at the towns of Soka, Hotogaya and Kawaguchi, where the intensity was nearly the same as in the low parts of Tokyo.

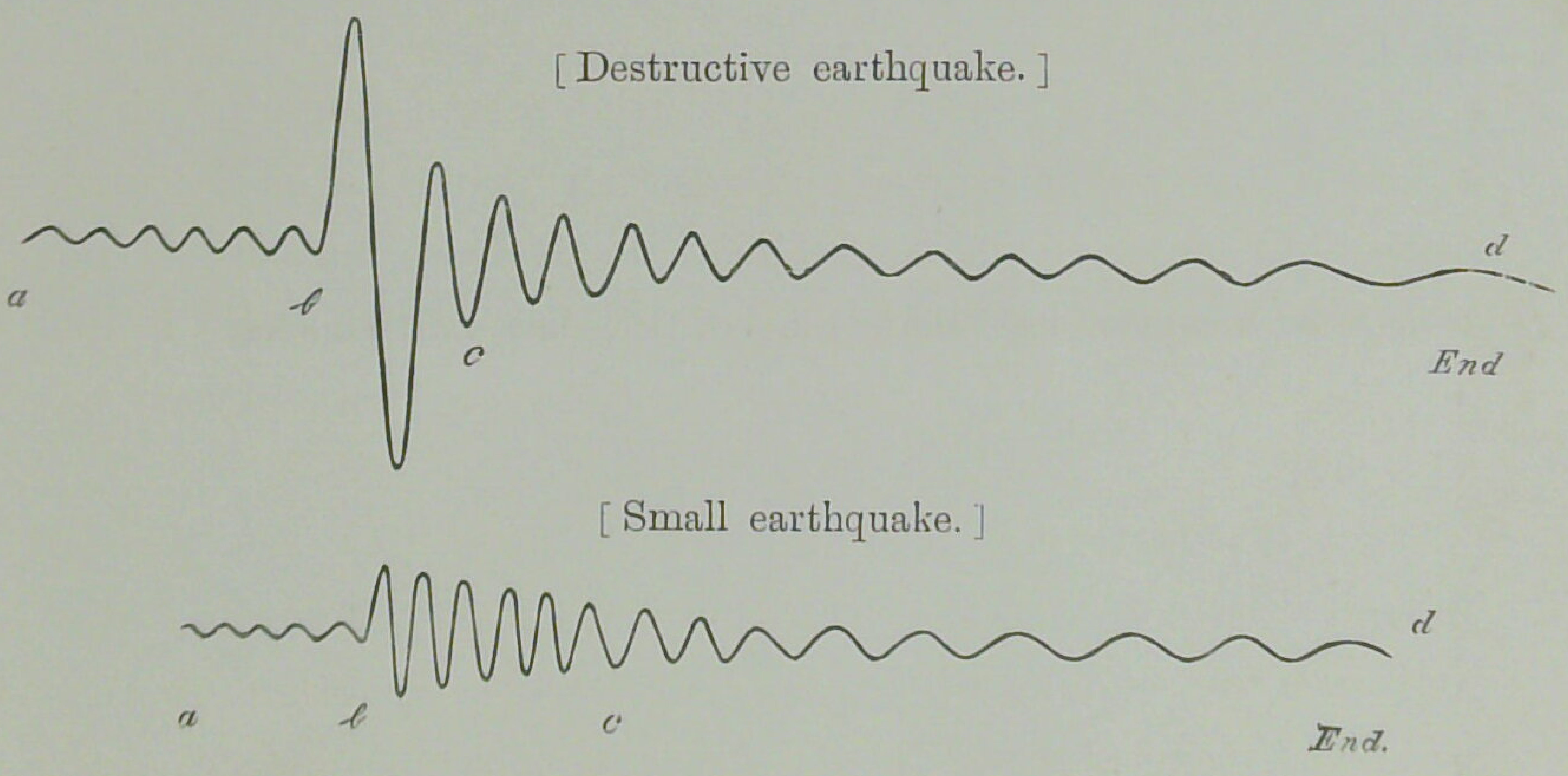
4.—As will be seen from §§ 1 and 2, the magnitude or the extent of this earthquake was much greater than those of the earthquakes of Kumamoto and Kagoshima. Its epifocal intensity of motion was however less than that of the last two shocks. It thus appears that the origin of the Tokyo earthquake was comparatively very deep.

5.—The earthquake was, unlike other destructive ones, followed by only a few after-shocks. This apparent anomaly is however easily explained by assuming the depth of the earthquake origin to be so great that the after-shocks mostly did not reach the epifocus. (See § 4.) On the night succeeding the earthquake, I observed a tromometer, or exceedingly sensitive seismoscope from time to time, but the ground was found to be already perfectly calm.

6.—The earthquake was satisfactorily recorded by a strong-motion seismograph set up in the Seismological Institute (Hongo, Tokyo). According to the diagram,* the maximum horizontal motion was 73 mm, while the maximum vertical motion was only 10 mm. The damage to buildings, chimneys, etc., was therefore practically caused by the horizontal motion only.

The character of the earthquake motion was very simple, the preliminary tremor having been followed at once by the single maximum vibration, which was much larger than the rest of the motion. I believe the motion in the meizoseismal area of destructive earthquakes to be generally of this type, and not necessarily so complicated as at great distances from the origin. In the case of small ordinary earthquakes, there is no single prominent displacement, the motion consisting of a great number of vibrations of nearly equal amplitude. (See the accompanying figure.)

DIAGRAMS ILLUSTRATING THE EARTHQUAKE MOTION.



ab.........The Preliminary tremor.
bc.........The Principal portion.
cd.........The End portion.

* The diagram, originally published by the late Prof. S. Sekiya and myself in Vol. VII of the Jour. Coll. Sc. Imp. Univ. of Tokyo, is reproduced in the following paper.

It is probable that the motion in small earthquakes consists of nearly pure elastic vibrations of the material constituting the earth's crust ; while the principal motion in destructive earthquakes is beyond the elastic limit of the latter and consequently is rapidly diminished.

The simple character of the motion naturally implies the existence of a definite direction in the earthquake. The maximum motion in the Tokyo shock under consideration was directed towards S 70° W.

7.—In connection with the direction of the earthquake motion, I observed in different parts of the city of Tokyo 224 cases of overturning of stone lanterns, of which 152 had cylindrical stems, and the remaining 72 square stems. Besides these, there were 21 cases of projection, or overthrowing of the tops of stone lanterns and tomb-stones. The results of the observation are given in table I.

TABLE I.—DIRECTIONS OF OVERTURNING OF STONE LANTERNS AND TOMB-STONES OBSERVED IN DIFFERENT PARTS OF TOKYO.

(a) 152 stone lanterns with cylindrical stems.			
(Fukagawa.)	W	N 25° E	S 40° W
	S 60° W	N 80° E	N 40° W
N 80° W	S 80° W	N 50° E	S 40° W
N 80° W	N 20° E	S 65° W	N 50° W
S 80° W	S 65° W	S 65° W	SW
W	W	SW	SW
S 60° W	N 85° W	SW	S 80° W
S 45° W	S 80° W	W	N
S 20° E	S 60° W	S 20° E	N 25° E
S 25° W	S 85° W	N 45° E	SE
S 65° W	N 5° E	W	
S 35° W	N 30° E	SW	
SW	S 80° W	S 75° W	(Aoyama)
W	S 25° E	W	
SW	SE	S 60° E	N 10° E
S 60° W	W	S 65° W	N 60° E
S 60° E	S 50° W	NE	
W	S 80° W	N 65° W	
S 60° W	N 75° E	NW	
S 70° E	S 80° W	S 75° W	(Shiba)
S 65° E	N 60° E	S 55° E	
NW	S 85° W	NE	S 70° W
S 60° W	S 75° W	E	SW
W	N 15° W	S 80° W	SW
S 80° W	N 70° E	S 85° W	N 80° E
S 35° W	S 85° W	S 35° E	S 25° W
S 55° W	S 80° W	S 65° E	S 40° E
S 65° E	N 65° E	N 65° E	N 70° E
S 65° W	S 80° W	SE	N
W	E		S 80° W
N 75° W	N 70° W		N 35° E
S 80° W	N 85° W	(Hongo)	W
S 80° W	S 70° W		S 20° W
W	N 75° W	S 50° W	S 40° W
W	NE	N 50° E	N 85° W
W	N 70° E		N 35° E
S 80° W	S 55° E		N 10° W
S 65° W	S 70° W		N 70° W
S 65° W	N 70° W	(Shitaya)	N 80° E
W	NE		W
S 80° W	N 40° E	NE	S 75° W
S 75° W	N 80° E	S 50° W	S 50° W
S 75° W	S 50° W	NE	S

(b) 72 stone lanterns with stems of square section.

(Fukagawa.)

S
S 30° W
S 25° W
N 35° E
S 50° W
S 30° W
N 50° E
W
W
W
NE
S
W
S 65° W
S 75° W
W
W
W
E
WSW
W
E
W
W
W
E
S 30° E
E
SW
S 70° W
W
S 75° E
S 10° E
E
S 50° W
S 75° W
S 70° W
N 80° E
N 20° E
W
N 80° E
E
N 55° E
S 30° E
S 70° W
E
W
W
N 70° W
W

(Hongo.)

W
S 10° W
N 10° E
S 50° W
N 80° W
S 20° W
S
S 70° W
S

(Shitaya.)

N 20° E
N 70° W

(Shiba.)

SW
N 30° E
NE
N 30° E
N 25° E
S 30° W
N 40° E
N 30° E
N 40° E
SW
N 20° E

(c) 21 miscellaneous cases, tops of stone lanterns, tomb-stones, etc.

(Fukagawa.)

NW
NE
S 50° W
N 60° W
N 70° W
S 60° W
W
E
W
W
W
W
S

(Shiba).

SW
S 35° W
S 30° W
N
S 70° E
N 40° E

(Hongo.)

S 10° W
N 70° E

From table I, it will be seen that the majority of the columns were overturned towards WSW. Averaging all the directions of overturning, we obtain the mean value of N 71° E and S 71° W, which is exactly the direction of the maximum vibration as given by the seismograph diagram. Further it is to be noted that the cases of overturning exactly, or nearly, towards S 71° W were much more numerous than those towards the opposite direction. In this earthquake therefore the chief direction of the overturning was not contrary to, but the same as, that of the maximum displacement of the ground.

Uniformity of the overturning direction was likewise found to be the case in Mino-Owari earthquake of Oct. 28th 1891, and the Shonai earthquake of Oct. 20th 1894.

The result contained in table I (a) is graphically illustrated in fig. 2, in which each mark (×) indicates the overturning of one stone lantern in the corresponding radial direction. When several columns were overturned towards one and the same direction, an equal number of marks has been put along the proper radius.

The mean of the directions of projection of the tops of the ten chimneys in Tokyo broken by this earthquake * is nearly the same as that of the overturning, namely ENE and WSW.

8.—Specimens of the brick work of a few houses, chimneys and walls damaged by the earthquake were brought to the Engineering Laboratory and had their tensile strength determined. The results are given in table II.

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* See § 25 of the present author's paper: Seismic experiments on the fracturing, etc.

TABLE II.—Tensile strength of brick-work damaged by the earthquake of June 20th 1894, in Tokyo.

Building.	Quality of brick.	Thickness of the mortar joint.	Tensile strength per sq. in.	Composition of mortar.	Remarks.
Private dwelling house, Atagoshita, Shiba.	Ordinary 1st class.	$\frac{3}{8}$ inch	lbs. 46.94	Lime and sand.	Broke through mortar.
	,,	$\frac{5}{16}$,,	16.52	,,	,,
			mean 31.4		
Chimneys; Educational Department.	Extra-superior class.	$\frac{3}{8}$,,	22.76	Lime and sand.	Broke through mortar.
	,,	,, ,,	9.67	,,	Broke through mortar and brick.
	,,	,, ,,	11.15	,,	Broke partly through mortar and partly by separation of brick and mortar.
	,,	,, ,,	33.26	,,	,,
			mean 19.5		
Wall; the Astronomical Observatory.	,,	$\frac{1}{4}$,,	87.72	Lime and sand.	Broke through brick.
Kinjō Gakkō.	Ord. 1st and 2nd class.	$\frac{9}{16}$,,	43.87	,,	Broke partly through mortar, and partly by separation of mortar and brick.
Rikkyō Gakkō.	Ordinary 1st class.	$\frac{3}{8}$,,	56.89	,,	Broke partly through brick, and partly by separation of brick and mortar.
Chimneys; the Okurashō.	Ordinary 2nd class.	$\frac{5}{16}$,,	22.89	,,	Broke through mortar.
Chimneys; the Naimushō.	Extra superior class.	$\frac{3}{8}$,,	48.72	Cement, lime and sand.	Broke partly through brick, and partly by separation of brick and mortar.
	,,	,, ,,	24.64	,,	Broke by separation of brick and mortar.
	,,	,, ,,	50.37	,,	,,
	,,	,, ,,	51.57	,,	Broke partly through brick, and partly by separation of brick and mortar.
			mean 43.8		
			General mean 44.0		

In the testing those cases of free separation of brick and mortar for which the tensile strength was nearly zero have been excluded. Hence the mean value of 44 lbs. per sq. in. is to be regarded as giving the average tensile strength of the better portions of the common brick-work.

Fig. 1.

The Earthquake of June 20th, 1894.

--- Boundary of provinces.

I, Boundary of the area of slight motion.

II, ... strong "

The Diagram of the Semi-destructive Earthquake of June 20th 1894 (Tokyo).

BY

THE LATE PROFESSOR S. SEKIYA

AND

F. OMORI, D^r. Sc.

The earthquake of June 20th 1894 was the most violent, that has shaken Tokyo since the well-known great catastrophe of the 2nd year of Ansei (1855). The mean radius of the disturbed area was about 80 ri or 200 miles, and the total land area was 7,100 square ri, or 42,000 square miles. The meizoseismal tract was a zone lying to the east of Tokyo, and extending in a N-S direction from the vicinity of the town of Iwatsuki to Tokyo Bay. No house was absolutely destroyed, but in the lower parts of Tokyo many brick buildings received severe damage, and chimneys in particular were mostly thrown down; some *dozō* (godowns) had their plastered mud walls very much cracked and shaken down, tomb stones and *ishidōrō* (stone lanterns in gardens) were overturned, small cracks were formed in the ground, and, in a few cases, ejection of water took place. The number of casualties in the three Prefectures of Tokyo, Kanagawa, and Saitama were 26 persons killed and 171 wounded. In fact it was the severest shock that the younger generation has felt in this metropolis.

The diagram of the earthquake was taken by a Large Motion Seismograph, set up in the Seismological Institute of the University, which was specially designed for recording strong earthquakes. The instrument is in principle the same as the ordinary horizontal pendulum and vertical motion seismographs widely used in this country. The main differences are, (*a*) the working parts are made stouter to withstand severe shakings, (*b*) the writing pointers are made longer so as to record large ranges of motion, and (*c*) the pointers have no multiplication ratio so that the actual magnitude of the motion is given. This is the first time that a clear instrumental record of a destructive earthquake was ever taken in this country; probably no such has ever been obtained in any other country.

The Seismograph records the motion decomposed into three components. The wave lines on the two inner circles indicate one the NE–SW, and the other the SE–NW components, and that on the outermost circle the vertical component. By compounding the component motions we can find the resultant. The recording plate revolved once in 118 seconds, and the short radial lines mark successive seconds of time counted from the start. We can thus determine the magnitude and direction of motion at any instant during the earthquake, as well as the duration, the intensity, and other elements of the shock. Below we give results deduced from the diagram.

Time of Occurrence. 1894, June 20th; 2^h 4^m 10^s p.m.

Horizontal Motion. The earthquake began as usual with tremors. The duration of tremors, as indicated by ordinary seismographs, lasted about 10 seconds. The Large Motion Seismograph does not record minute tremors, and the motion was already a few millimetres in range at the beginning of this diagram. However, we shall take this latter point as the beginning of the earthquake. The motion, already strong in the 1st and 2nd seconds, became suddenly violent and the ground moved 37 mm. during the time interval between the 3rd and 4th seconds. This was followed by a counter-movement of 73 mm., which was the maximum horizontal motion during the earthquake, and was again followed by a motion of 42 mm. The motion during about 1 minute succeeding the above three most prominent shocks was very much weaker, though still great in range. Some large undulations occurred between the 40th and 53rd seconds and again between the 70th and 78th seconds. But the intensity of motion was not so strong as before. The comparatively little damage occasioned by the present earthquake notwithstanding such great horizontal movements is no doubt due to the small number of violent oscillations.

Period of Horizontal Motion. The above maximum horizontal motion was executed in 0·9 second, so that the complete period of oscillation would be 1·8 second.

Direction of Motion. The direction of the motion changed as usual during the earthquake. But the maximum horizontal motion was directed toward S. 70° W., and the chief movements before and after were also directed nearly towards the same point, or else the opposite direction. We have also examined the directions of overturning of 245 stone lanterns (*ishidōrō*) in different parts of Tokyo. Their mean direction of overturning was toward S. 71° W. Thus the direction of overturning of columns is seen to be identical with that of the maximum horizontal motion.

Vertical Motion. The maximum vertical motion of 10 mm. occurred in the 3rd second nearly simultaneously with the first prominent horizontal motion. Vertical motions occurred more or less during the next 30 seconds.

Maximum Acceleration. The maximum acceleration of the motion of an earth particle, calculated from the above values of the maximum horizontal motion and its period, is 444 mm. per sec. per sec. This was the maximum acceleration in the upper districts of Tokyo where the ground is composed of hard loamy soil; in the lower districts, where the ground is soft and marshy, it nearly reached the value of 1,000 mm. per sec. per sec. This latter has been calculated from a similar record given by another Large Motion Seismograph set up in the lower part of the city. Now the maximum acceleration is the quantity which measures the destructive power of earthquake motion, and it may therefore be inferred that whenever it reaches the above values, chimneys will be greatly damaged and buildings affected as on this occasion. For the Mino-Owari Great Earthquake of 1891, one of us has calculated, from observations of numerous overturning and fractured bodies, the maximum acceleration of earthquake motion in the meizoseismal district to have been from 3,000 mm. up to nearly 10,000 mm. per sec. per sec. These results are probably the first numerical estimation that has been made of the destructive power of great earthquakes.

Duration of the Earthquake. The shaking lasted about 4 minutes and 30 seconds.

Note on the After-shocks of the Hokkaido Earthquake of March 22nd 1894.

BY

F. OMORI, D. Sc.

The time and space distribution of the after-shocks of the great Kumamoto (1889), Mino-Owari (1891), and Kagoshima (1893) earthquakes has already been discussed in Vol. VII of the Journal of the College of Science, where it was shown amongst other things that the relation between their frequency (y) and the time (x) is very nearly expressed by the following equation,

$$y = \frac{k}{h + x}, \qquad (1)$$

h and k being constants. Let us here examine whether the after-shocks of the Hokkaido earthquake follow the same law of frequency-decrease, the necessary data being given in tables I to IV, appended to the present note.

The after-shocks of this earthquake, which took place on March 22nd 1894, at 7^h 56^m p.m., were successfully recorded at the Meteorological Observatory of Nemuro, about 120 km. from the epicentre, their numbers during the first five days being given in the 3rd column of the following list.

NUMBER OF THE AFTER-SHOCKS OBSERVED AT NEMURO.

x	Date. March, 1894.	Actual daily frequency corresponding to x.	y, the calculated value of the daily frequency.
0	From the 23rd, noon, to the 24th, noon.	88	89,6
1	„ 24th „ 25th	42	42,2
2	„ 25th „ 26th	31	27,9
3	„ 26th „ 27th	19	20,6
4	„ 27th „ 28th	16	16,3

Calculating by the method of Least Squares from the number of the after-shocks actually observed during these five days, equation (1) assumes the following form :—

$$y = \frac{79.9}{x+0{,}8896}, \qquad (2)$$

in which y is the after-shock frequency, and x the time expressed in days, its origin corresponding to the 24 hours interval between the noons of the 23rd and the 24th of March 1894. The frequency for $x=0$, 1, 2, 3, 4, calculated by this equation agrees, as shown in the 4th column of the above table, very closely with the actual values.

As a verification, let us calculate by equation (2) the number of shocks at Nemuro during the 3rd and 4th years after the first great earthquake. Now the middle day of the 3rd year, April 1896–March 1897, is represented by $x=x_1 = 923$; and that of the 4th year, April 1897–March 1898, by $x=x_2 = 1288$. The values of y which correspond to these values of x are found, by equation (2), to be :

for $x=x_1$, $y=y_1=0{,}0865$;

and for $x=x_2$, $y=y_2=0{,}0620$.

The approximate numbers of shocks at Nemuro during these two years would therefore, according to equation (2), respectively be

(for the 3rd year) $n'_1=365\times y_1=32$,

and (for the 4th year) $n_2'=365\times y_2=23$.

But equation (2), which has been deduced from the observed numbers of shocks during the five days immediately after the initial great earthquake, is to be regarded as giving the Nemuro seismic frequency, only so far as the after-shocks are concerned. To obtain the correct result we must therefore add to the above values of n'_1 and n'_2 the average annual number of ordinary earthquakes at the same place, which according to seismic observations during the nine years 1885–1893, is 39. (See Table III). Hence

$$n_1 = n'_1 + 39 = 71,$$

and $$n_2 = n'_2 + 39 = 62$$

are the estimated numbers of earthquakes we ars seeking.

Further, $$n_1 + n_2 = 133.$$

As a matter of fact there were at Nemuro during the two years under consideration respectively 87 and 52 earthquakes, amounting altogether to $87+52=139$. In discussions of this sort, these numbers are to be regarded as being practically identical respectively with the calculated results n_1, n_2 and (n_1+n_2).

The relation between the time and the seismic frequency at Nemuro is graphically illustrated in figs. 1 and 2.

TABLE I.—Hourly Earthquake Number at Nemuro.

(From the 22nd to the 31st March, 1894).

Hour \ Day	22	23	24	25	26	27	28	29	30	31
0— 1 a.m.	...	6	7	0	2	0	0	0	0	0
1— 2	...	4	1	1	1	0	0	0	0	0
2— 3	...	4	2	1	0	1	1	0	0	1
3— 4	...	12	1	1	2	0	1	0	0	1
4— 5	...	15	0	2	3	1	1	0	0	0
5— 6	...	10	4	2	1	0	3	0	0	0
6— 7	...	7	6	2	1	1	0	0	1	0
7— 8	...	4	4	3	1	2	4	0	1	0
8— 9	...	7	2	1	1	1	0	0	0	1
9—10	...	2	2	2	1	0	0	0	0	0
10—11	...	1	2	2	1	0	0	1	0	0
11—12	...	8	0	2	1	1	0	0	0	0
0— 1 p.m.	...	10	3	1	2	0	0	1	0	0
1— 2	...	4	1	2	0	0	0	0	0	0
2— 3	...	3	0	3	0	0	0	0	0	1
3— 4	...	7	2	1	2	0	0	0	0	0
4— 5	...	3	2	2	1	0	0	0	0	1
5— 6	...	2	3	1	1	1	0	1	0	0
6— 7	...	5	2	1	1	0	0	0	2	1
7— 8	1	8	2	2	1	0	1	0	0	0
8— 9	16	4	1	0	2	2	0	0	0	1
9—10	13	5	6	2	0	2	0	0	0	0
10—11	7	1	0	0	1	0	0	0	0	0
11—12	13	4	1	1	1	1	0	0	0	1
Sum	50	136	55	35	27	13	11	3	4	8

TABLE II.—Daily Earthquake Number at Nemuro.
(From March 22nd to Dec. 31st, 1894).

Day \ Month	III	IV	V	VI	VII	VIII	IX	X	XI	XII
1	...	4	2	3	1	0	2	0	0	0
2	...	4	4	2	3	1	3	2	0	2
3	...	3	0	0	0	1	0	1	0	0
4	...	8	2	0	0	2	0	3	0	3
5	...	5	1	1	1	0	3	1	1	0
6	...	4	2	1	0	0	1	2	0	0
7	...	7	0	1	1	0	1	1	1	1
8	...	2	1	2	2	0	3	1	2	1
9	...	4	1	3	0	0	1	1	0	0
10	...	6	5	3	0	2	1	0	0	0
11	...	11	0	1	0	1	0	0	0	0
12	...	8	3	3	2	1	0	1	0	0
13	...	6	0	0	1	0	0	1	0	0
14	...	2	1	1	1	0	1	1	2	0
15	...	3	3	0	0	0	0	1	1	0
16	...	2	1	2	0	0	1	1	0	3
17	...	1	3	0	0	0	0	2	1	1
18	...	3	0	0	1	2	1	1	1	0
19	...	2	3	2	1	1	1	2	1	1
20	...	2	1	0	3	0	0	0	1	0
21	...	2	1	1	3	0	1	1	1	0
22	50	1	6	1	0	0	0	0	1	0
23	136	5	0	0	0	0	0	2	0	0
24	55	3	2	0	1	0	0	3	0	0
25	35	3	0	1	0	2	1	0	0	0
26	27	0	3	2	0	0	1	0	0	0
27	13	3	0	1	1	1	0	0	2	0
28	11	2	0	4	1	3	0	0	1	0
29	3	4	0	1	0	0	1	1	1	0
30	4	1	0	3	1	3	0	1	0	0
31	8	...	0	...	0	0	...	0	...	0
Sum	342	111	45	39	24	20	23	30	17	12

TABLE III.—MONTHLY EARTHQUAKE NUMBER AT NEMURO.

(1894—1898.)

Month / Year	I	II	III	IV	V	VI	VII	VIII	IX	X	XI	XII	
1894	...	...	363	111	45	39	24	20	23	30	17	12	
1895	11	10	12	16	11	8	10	10	11	10	7	11	
1896	4	3	7	5	7	7	15	6	11	1	6	8	
1897	5	6	7	6	5	9	3	6	2	6	2	4	
1898	4	3	1	3	5	2	1	3	4	10	6	2	
Sum	24	22	27	30	28	26	52	42	47	47	32	35	(A)
	13	12	15	14	17	18	28	22	24	17	15	23	(B)
	23	21	25	22	35	34	29	27	31	33	34	38	(C)

(A), (B) and (C) are the monthly sums deduced respectively from the eqke numbers for the 4 yrs July 1894–June 1898 ; 3 yrs, July 1895–June 1898 ; and 9 yrs, Jaunary 1885–December 1893.

The following monthly and seasonal distributions of earthquakes at Nemuro have been obtained by adding (B) and (C).

Month	I	II	III	IV	V	VI	VII	VIII	IX	X	XI	XII	Sum	
Number of earthquakes	36	33	40	36	52	52	57	49	55	50	49	61	570	(D)

Spring	(III, IV, V)	128
Summer	(VI, VII, VIII)	158
Autumn	(IX, X, XI)	154
Winter	(XII, I, II)	130
	Sum	570

Warmer months	(IV—IX)	301
Colder months	(X—III)	269
	Sum	570

TABLE IV.—Monthly Earthquake Number at Nemuro.

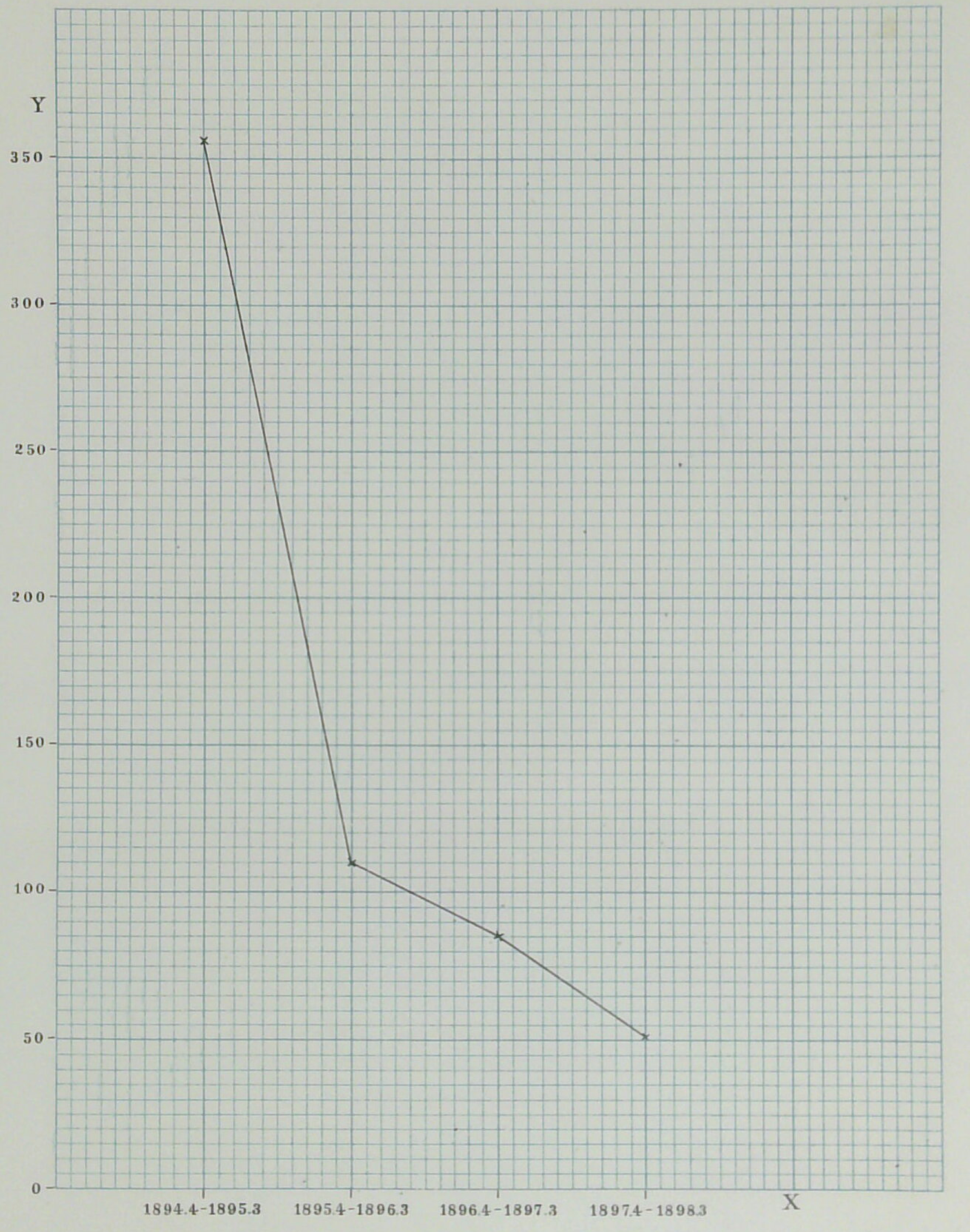
(1885—1893.)

Month / Year	I	II	III	IV	V	V	VII	VIII	IX	X	XI	XII	Sum
1885	0	0	3	1	1	4	5	1	4	7	6	1	33
1886	2	5	4	3	3	3	2	4	4	5	2	6	43
1887	2	2	0	2	7	2	5	5	1	1	3	3	33
1888	0	3	4	0	1	2	1	2	1	2	1	1	18
1889	6	3	1	3	7	2	7	3	6	3	3	4	48
1890	3	3	3	6	7	6	0	3	4	2	10	7	54
1891	2	3	7	3	2	3	3	3	2	4	2	11	45
1892	5	1	1	2	3	0	2	1	4	5	3	3	30
1893	3	1	2	2	4	12	4	5	5	4	4	2	48
Average	23	21	25	22	35	34	29	27	31	33	34	38	352

Mean Yearly Number.........39

Spring	(III, IV, V)	82
Summer	(VI, VII, VIII)	90
Autumn	(IX, X, XI)	98
Winter	(XII, I, II)	82
Sum		352

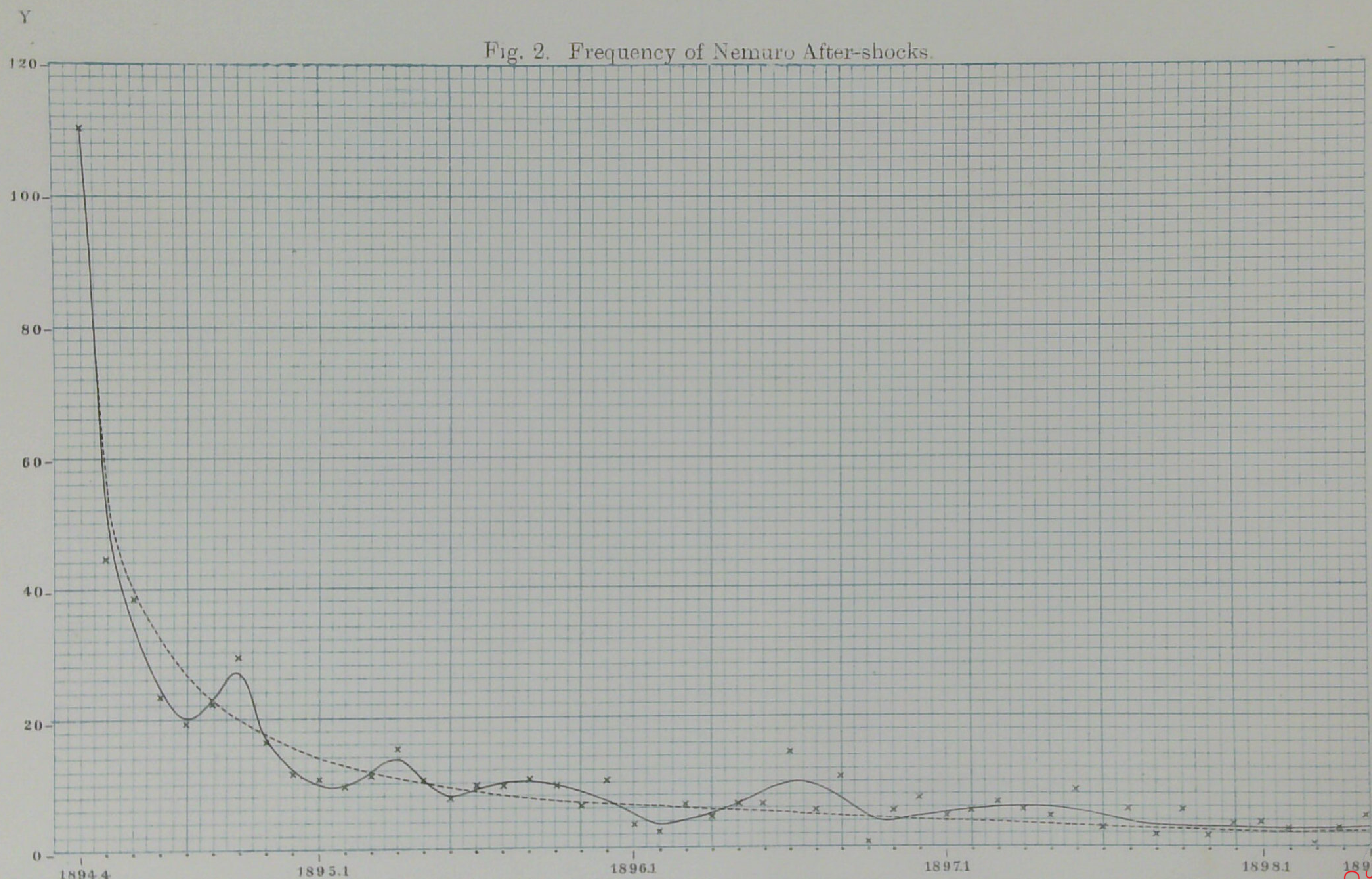
Fig. 1. Frequency of Nemuro After-shocks



X = Time in years.

Y = Number of earthquakes during 12 months.

Fig. 2. Frequency of Nemuro After-shocks.



X=Time, in months.
Y=Monthly number of earthquakes.

Elastic Constants of Rocks and the Velocity of Seismic Waves.

BY

H. NAGAOKA, *Rigakuhakushi* (*D. Sc.*)

Member of the Earthquake Investigation Committee.

The vibration of the earth's crust has from time to time been a favourite subject of discussion among the elasticians, and the propagation of seismic disturbance is a problem, whose solution has long been hoped for, both from the theoretical and the empirical point of view. With improved instruments, seismologists have recently determined the velocity of propagation with tolerable accuracy, but very little is known of the elastic nature of the medium through which the vibration has travelled. The resources from which physicists and seismologists draw their theoretical inferences are so scanty, that among the numerous rocks which constitute the earth's crust, only a few of the most commonly occurring rocks have had their physical properties investigated. The questions of elasticity, having close bearing with the deformation of the earth's crust, have repeatedly been a subject of research by several distinguished elasticians as Lord Kelvin, Boussinesq, Cerruti, and Chree. But we are baffled in our attempt to apply the result of subtle analysis to the actual problem, from the lack of our experimental knowledge as regards the elastic nature of the diverse rocks, which compose the outer coating of our planet. The present experiments were undertaken with a view to fill these gaps, and to supply on the one hand the wants of physicists, whose aim is to apply dynamics to the study of the geological phenomena, and on the other to meet the needs of seismologists, engaged in solving the problems touching the propagation of seismic waves.

Preparation of the Specimen.—The present experiments deal principally with the determination of Young's modulus and the modulus of rigidity, made on specimens of rocks which were easily accessible.

The number of rocks examined amounted to about eighty different specimens collected from various localities. These rocks were first cut in the shape of a rectangular parallelelopiped, and afterwards carefully polished into prisms of nearly 1 cm. square cross section and 15 cm. length. It was at first proposed to experiment with much larger specimens, but it was generally found impossible to find a large homogeneous piece with no trace of cleavage ; in addition to this, the apparatus with which the elastic constants were to be measured would become cumbrously large, and require great solidity, increasing at the same time the difficulties of experiment.

Most of the specimens were apparently isotropic, but on close examination it was found that the isotropy was only superficial. Rocks as slates with distinct sedimentation planes were generally cut parallel and perpendicular to them ; where such planes of symmetry were not easily discernible, the specimen was conveniently cut into prisms.

The thickness of these prisms was measured by a contact micrometer reading by means of a vernier to $\frac{1}{100}$ mm. at three different places in the middle line of two opposite faces ; namely, one at the middle and two at one quarter distance from the ends. The mean density of the prism was measured by dividing the mass by the volume, which was calculated from the known length and thickness. The density of several prisms cut from the same sample did not generally agree, showing that the material was only roughly homogeneous.

Modulus of Elasticity.—Young's modulus was measured by flexure experiment. The specimen to be tested was placed on two steel wedges, which served as fulcrums. The edge of the wedge was slightly rounded in order to prevent cutting on applying heavy weights. The flexure due to the weight hung at the middle of the prism was measured by means of a scale and telescope. By a special arrangement, a plane mirror was attached to the prism at the place where it rested on the wedges. The mirror was nearly vertical and the image of the vertical scale divided in mm., and placed at a distance of 2.73 m., was observed by a telescope provided with a filar micrometer. By this means, the deflection of 1″ was easily measurable.

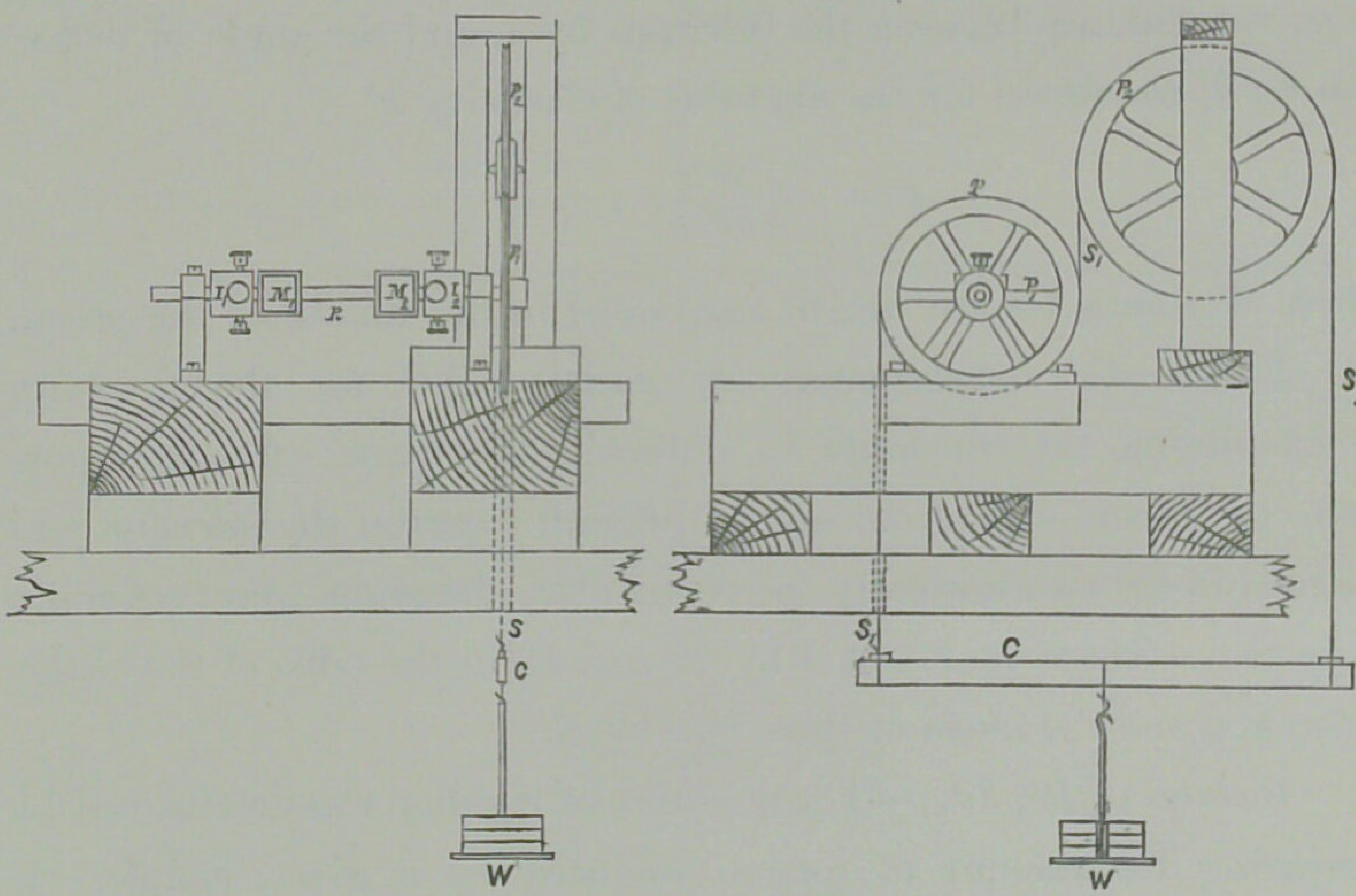
Denoting the length and the thickness of the prism by b and c resp., the distance between the fulcrums by a, and the angle of deflection by δ, we obtain for the modulus of elasticity E

$$E = \frac{3\,\mathrm{W}\,a^2}{4\,b\,c^3\,\delta},$$

where W stands for the weight suspended in the middle of the prism.

The elastic heterogeneity of rocks called for the necessity of examining the constants in different directions; for this purpose, the prism was placed on its different faces on the fulcrum and the moduli for two mutually perpendicular directions were generally measured. These are denoted by E_1 and E_2 in the table of the elastic constants, and the mean of these two by E.

Modulus of Rigidity.—The modulus of rigidity was determined by measuring the amount of torsion produced by a given couple. It would lead too far if I attempt to describe the details of the instrument. The rectangular prism R was placed horizontal and firmly clamped at its both extremities to two solid pieces I_1, I_2 of iron. In order to prevent cracking by too firmly clamping, four small pieces of brass plates with thin sheet lead underneath was interposed between the four faces of the prism and the clamping screws. I_1 was fixed to a solid iron frame. The central steel cylinder protruding from I_2 was filed down to a sharp knife edge on its axis, coinciding with the central line of the prism. An agate plane attached to another solid iron frame supported the knife edge and the twisting pulley P. To the cylinder above referred to, a pulley P_1 of 14 cm. diameter was firmly fixed; a flexible string s_1 attached to a pin p on the circumference of the pulley passed over it, and was tied to a light wooden cross bar c. Another string s_2 was attached to the pulley, and instead of passing over it, was slung around another pulley P_2 such that the line of passage s_2 from P_1 to P_2 was vertical. The string on going over P_2 in



the opposite direction as the former string was again let down vertical and attached to the cross bar. By hanging the weight at the middle of the bar, the tension was the same in both strings and gave rise to a couple = radius of the pulley × weight. By this arrangement, the knife edge did not support the load producing the twisting couple, that of the prism, clamp and pulley being the only weight acting. The amount of torsion was measured by observing the deflection of two mirrors M_1 and M_2, one attached to the prism near the fixed clamp I_1 and the other near I_2. The deflections as measured by a vertical scale and two telescopes were generally large compared with those in flexure experiment, so that no micrometric measurement was needed. The difference of the two scale readings gave the torsion between the two places where the mirrors were fixed by special clamp screws.

Denoting the sides of the prism by b and c, the torsion for unit length by τ, the twisting couple by N, and the rigidity by μ, we get by St. Venant's formula for the torsion of a rectangular prism the following expression for N

$$N=\mu\tau b^3 c\left\{\frac{16}{3}-\frac{b}{c}\left(\frac{4}{\pi}\right)^5\sum\frac{1}{(2n-1)^5}\,\frac{e^{\left(\frac{2n-1}{2b}\right)\pi c}-e^{-\left(\frac{2n-1}{2b}\right)\pi c}}{e^{\left(\frac{2n-1}{2b}\right)\pi c}+e^{-\left(\frac{2n-1}{2b}\right)\pi c}}\right\}.$$

It may be a question whether it is justifiable to use St. Venant's formula in the present experiment, as the boundry condition are somewhat different from those considered by St. Venant in deducing the above result. As the length of the prism was large compared with its thickness, and as the twist τ was measured at points not very near the ends of the prism, the result by using the above formula will not be materially different from the actual value. When the rock is of stratified structure and shows great difference in its elastic behaviour the formula will require modification, but in studying the elasticity of rocks in its broad feature, the modulus of rigidity calculated in the above manner will not be far from the general mean. The calculation of the series involved in the above formula is somewhat tedious. Fortunately, St. Venant has calculated a table of

$$\sum \frac{1}{(2n-1)^5} \frac{e^{\left(\frac{2n-1}{2b}\right)\pi c} - e^{-\left(\frac{2n-1}{2b}\right)\pi c}}{e^{\left(\frac{2n-1}{2b}\right)\pi c} + e^{-\left(\frac{2n-1}{2b}\right)\pi c}}$$

for different values of $\frac{c}{b}$. As the section of the prism was nearly square shaped, it was thought advisable to calculate the sum of the series at small intervals, when the ratio $\frac{c}{b}$ is nearly unity. As such tables will sometimes be found useful, I give the result of calculation in the following table.

Table of $\frac{16}{3} - \frac{b}{c}\left(\frac{4}{\pi}\right)^5 \sum \frac{1}{(2n-1)^5} \frac{e^{\left(\frac{2n-1}{2b}\right)\pi c} - e^{-\left(\frac{2n-1}{2b}\right)\pi c}}{e^{\left(\frac{2n-1}{2b}\right)\pi c} + e^{-\left(\frac{2n-1}{2b}\right)\pi c}} = \beta$

$\frac{c}{b}$	β	$\frac{c}{b}$	β
1.00	2.249	1.15	2.563
1.01	2.272	1.16	2.583
1.02	2.294	1.17	2.602
1.03	2.316	1.18	2.621
1.04	2.338	1.19	2.639
1.05	2.359	1.20	2.658
1.06	2.379	1.21	2.676
1.07	2.402	1.22	2.694
1.08	2.422	1.23	2.713
1.09	2.443	1.24	2.730
		1.25	2.748
1.10	2.464		
1.11	2.484		
1.12	2.504		
1.13	2.524		
1.14	2.543		

Hooke's Law and Elastic After-effect.—Preliminary experiments with granite showed that Hooke's law does not hold even for very small flexure and torsion, and that the after-effect is considerably great when the prism is sufficiently loaded or twisted ; the deviation from the direct proportionality between the strain and stress was incomparably great compared with that observed in common metals. This will be chiefly due to the inferior limit of elasticity, so that it is necessary to experiment only within narrow limits of loading or twisting. These limits are widely different for different specimens of rocks, and the modulus of elasticity as well as that of rigidity was always determined with such stresses as will approximately produce the strain proportional to it.

The deviation from Hooke's law was prominent in certain specimens of sandstones, and it was the more marked in torsion than in flexure experiments. In certain rocks, it is indeed doubtful if anything like a proportionality between stress and strain can be found even for extremely small change of shape. On releasing these rocks from stress, the return to the former state is extremely small showing that the elasticity of rocks is of very inferior order. The elastic yielding of rocks under continuous action of stress is very remarkable as the following readings of the deflection in the experiment on torsion will show.

SPECIMEN : IZUMI SANDSTEIN.

A=100.0 mm., B=10.12 mm., C=10.09 mm.,

Torsional loading : 400 grms

Zero reading before loading : 24.2

Loaded : 2^{h} $18.^{m}0$ Sept. 10, 1898

Time.		Reading.
h	m	
2	18.1	72.0
	18.5	75.3

Time.	Reading.
2^{h} $19^{m}.0$	77.1
19.5	78.1
20.0	78.9
20.5	79.6
21.0	80.1
21.5	80.6
22.0	81.0
23.5	81.4
23.0	81.8
23.5	82.1
24.0	82.4
25.0	83.6
27.0	84.1
28.0	84.5
29.0	84.9
30.0	85.2
31.0	85.5
32.0	85.9
33.0	86.2
34.0	86.5
35.0	86.8

It will be seen that the initial deflection amounts to 47.8 mm.; the torsion of the prism gradually increases in course of a few minutes, so that after a lapse of about 19 minutes, the increase of deflection is nearly 30 per cent of the initial. The increase becomes asymptotic with time.

The above mentioned property of rocks will be of no small interest in dynamical geology as it naturally illustrates the possibility of the folding of rocks and other kindred phenomena pertaining to the manifold change of shape in rocks, wrought by the continuous action of stress.

Velocity of Elastic Waves.—It was my intention to determine the modulus of elasticity, and then calculate the velocity of propagation of the longitudinal as well as that of the transversal waves, on the supposition that the material is isotropic. Few experiments with rocks of different ages showed that these attempts are for the most part fruitless, as the assumption of isotropy was not generally admissible. With archæan and palaeozoic rocks, it was possible to sort them into proper shape for experiment only in a certain direction, as they were generally of schistose structure, and extremely brittle in the direction perpendicular to it ; in such cases the elastic behaviour was of course widely different in these directions. Even with granite which apparently is homogeneous in structure, the difference of elasticity with direction was noticed. On enquiry these rocks were pressed from one side during its formation, and thus left its trace in the relation of strain to the stress. For the complete discussion of the elastic nature of these rocks, the determination of the moduli of elasticity and of rigidity considered as an isotropic substance is insufficient ; we are in fact dealing with quasi-crystalline bodies, so that the number of elastic costants must depend on the number of symmetry planes, which can be drawn in these rocks. The type of the elastic waves travelling in such a medium will be determined, when all of these constants are known. As we have no simple means of examining these symmetry planes, a single modulus of elasticity and rigidity was determined, on the supposition that the material is isotropic.

In the discussion of the propagation of seismic waves, we have to deal with wave-length which measures over a kilometre. Geologists tell us that uniform strata of a kilometer thickness are of rare occurrence, and it may be doubted if these waves do not suffer change of type and shape in traversing the earth's crust. Unquestionably longitudinal plane waves whose velocity of propagation in an isotropic medium is given by the formula $\sqrt{\frac{\lambda+2\mu}{\rho}}$ (following Lamé's notation) would seldom come into existence. A complete discussion of waves in quasi-crystalline rocks requires complicated analysis, which necessitates the knowledge of the elastic behaviour of rocks cut in various directions. To obtain a general view of the propagation, I have thought it advisable to calculate $V_l=\sqrt{\frac{E}{\rho}}$ for the longitudinal waves. Suppose the Young's modulus E is determined by flexure experiments on a prism cut parallel to a plane of symmetry, then V_l will give the velocity of longitudinal wave travelling along the prism. The velocity in the sense above explained is given under V_l and the velocity of the transversal wave $\sqrt{\frac{\mu}{\rho}}$ under V_t. I do not mean to say that the actual velocity of longitudinal waves in various rocks is given by V_l but when such values are not obtainable, V_l will probably give a rough estimate. The elastic constants of rocks are tabulated in the order of geological age; for the same geological age, those with larger velocity of propagation V_l come before those with the slower.

ELASTIC CONSTANTS OF ROCKS.

Rock	Specimen No.	ρ	E_1 (C.G.S.)	E_2 (C.G.S.)	E (C.G.S.)	μ (C.G.S.)	$V_l \frac{\text{kilm.}}{\text{sec.}}$	$V_t \frac{\text{kilm.}}{\text{sec.}}$
ARCHAEAN ROCKS.								
Chlorite Schist (*Chichibu*)	9	2.977	112.1×10^{10}	132.4×10^{10}	122.3×10^{10}	24.03×10^{10}	6.40	2.84
,,	50	2.955	146.0	147.6	146.8	31.57	7.05	3.27
(Eruptive)								
Peridotite Serpentine (*Kuzi*)	16	2.825	72.92	58.99	65.96	22.24	4.83	2.81
Peridotite Serpentine	41a	2.777	62.42	55.86	59.14	20.09	4.61	2.69
	41b	2.786	54.15	53.90	54.03	19.73	4.41	2.66
Ophicalcite	45	2.593	38.90	53.71	46.31	......	4.22	......
Peridotite Serpentine	17	2.570	39.03	46.00	32.52	16.00	4.07	2.49
PALAEOZOIC ROCKS.								
Schalstein (*Rikuchyū*)	79	2.653	120.50	92.25	106.4	18.90	6.32	2.67
Clayslate (*Nikkō*)	74	2.149	79.69	83.29	81.49	28.06	6.16	3.61
Schalstein (*Rikuchyū*)	78a	2.768	70.02	95.00	82.51	25.36	5.45	3.03
	78b	2.772	97.90	103.30	100.60	21.25	6.02	2.77
Sandy Slate (*Rikuchyū*)	73	2.640	81.79	92.40	82.10	17.05	5.75	2.54
Clay slate	2a	2.674	98.00	83.09	90.55	13.79	5.82	2.27
	2b	2.690	90.64	86.71	88.68	20.75	5.74	2.78
	2c	2.708	51.92	62.26	57.09	20.74	4.52	2.77
Limestone (*Musashi*)	55	2.630	84.95	88.45	86.20	29.83	5.74	3.38
Limestone	13	2.653	80.20	86.61	83.40	31.00	5.60	3.42
Limestone (*Musashi*)	29	2.682	68.86	79.55	74.20	21.71	5.26	2.84
Weathered Clayslate	1a	2.314	62.15	61.35	61.75	10.03	5.18	2.08
	1b	2.304	56.83	58.90	57.87	8.85	5.01	1.96

Rock	Specimen No.	ρ	E_1 (C.G.S.)	E_2 (C.G.S.)	E (C.G.S.)	μ (C.G.S.)	$V_l \frac{\text{kilm.}}{\text{sec.}}$	$V_t \frac{\text{kilm.}}{\text{sec.}}$
Marble	11a	2.654	76.0×10^{10}	63.72×10^{10}	69.86×10^{10}	30.11×10^{10}	5.13	3.37
	11b	2.625	63.53	46.2	54.86	28.60	4.54	3.45
Schalstein	80	2.824	74.60	70.52	72.56	18.96	5.07	2.58
Schalstein (*Tosa*)	75	2.762	57.68	37.70	47.69	8.98	4.63	1.80
Weathered Clay slate	60a	2.316	39.44	35.27	37.36	4.99	4.02	1.47
	60b	2.306	35.37	36.69	36.03	5.27	3.96	1.51
Marble	12a	2.650	37.26	37.64	37.45	15.08	3.76	2.39
	12b	2.650	37.33	28.33	32.82	18.80	3.93	2.66
Clayslate (*Tanba*)	3a	2.384	34.48	30.76	32.62	8.00	3.70	1.83
	3b	2.392	30.64	30.35	30.50	8.54	3.57	1.87
Contact Clayslate (*Mikawa*)	64a	2.462	30.35	28.10	29.23	......	3.45	1.71
	64b	2.416	31.00	31.86	31.43	......	3.61	......
Weathered Clayslate	7a	2.503	12.45	12.20	12.33	4.60	2.32	1.36
	7b	2.500	13.00	13.64	13.32	4.31	2.31	1.31
Weathered Clayslate	65a	2.490	12.72	12.26	12.49	6.59	2.24	1.63
	65b	2.500	12.54	12.47	12.51	4.43	2.24	1.33
(Eruptive)								
Granite (*Shōdoshima*)	69	2.572	37.91	46.71	42.31	18.43	4.05	2.68
Granite	42	2.550	31.42	......	......	13.99	3.51	2.34
Granite (*Hitachi*)	68	2.549	18.83	20.43	19.63	6.89	2.78	1.64
Granite (*Hitachi*)	71	2.590	14.84	15.12	14.98	5.05	2.42	1.40
Granite	52	2.503	15.23	9.73	22.48	5.47	2.22	1.48
Granite (*Hitachi*)	56	2.530	11.97	9.89	10.93	4.43	2.08	1.32
MESOZOIC ROCKS.								
Izumi Sandstein	5	2.216	9.2	9.0	9.12	3.1	2.03	1.18
	6a	2.236	7.1	7.2	7.12	2.4	1.78	1.04
	6b	2.223	7.7	7.6	7.67	2.7	1.86	1.10

Rock	Specimen No.	ρ	E_1 (C.G.S.)	E_2 (C.G.S.)	E (C.G.S.)	μ (C.G.S.)	V_e $\frac{\text{kilm.}}{\text{sec.}}$	V_t $\frac{\text{kilm.}}{\text{sec.}}$
Schalstein	77	2.778	75.7×10^{10}	83.0×10^{10}	79.4×10^{10}	23.2×10^{10}	5.35	2.89
Clayslate (*Rikuchyū*)	72	2.711	88.4	99.3	98.8	22.6	5.88	2.89
Clayslate (*Rikuchyū*)	53	2.702	83.6	85.3	84.5	18.5	5.59	3.17
Clayslate (*Tsushima*)	62a	2.681	32.2	50.6	41.4	14.8	3.91	2.35
	62b	2.678	43.7	44.3	44.0	14.2	4.06	2.31
CAINOZOIC ROCKS (Tertiary)								
Rhyolite (*Izu*)	51	2.316	32.1	17.5	24.8	14.0	3.24	2.46
Rhyolte Tuff (*Iyo*)	8a	2.346	21.9	21.5	21.73	9.32	3.05	1.99
	8b	2.316	21.8	20.0	20.90	8.05	3.01	1.86
Tuff Sandstone (*Kōzuke*)	19a	2.305	20.6	21.1	20.8	8.74	3.02	1.95
	19b	2.321	21.2	21.4	21.3	8.45	3.02	1.91
Rhyolite (*Kōzuke*)	59a	2.472	21.3	18.7	20.0	8.57	5.85	1.86
	59b	2.454	19.5	18.3	18.9	9.15	2.78	1.93
Rhyolite Tuff (*Mikawa*)	63a	2.228	18.8	19.9	19.3	6.9	3.00	1.79
	63b	2.198	17.4	11.8	14.6	......	2.59	......
Rhyolite (*Izu*)	27a	1.945	11.3	11.7	11.5	5.78	2.43	1.72
	27b	1.944	14.0	15.1	14.6	5.86	2.74	1.74
Rhyolite Tuff	32	1.889	8.1	10.1	9.1	4.2	2.20	1.49
Sandstone (*Chōshi*)	58	2.345	10.9	11.4	11.2	4.60	2.18	1.40
Rhyolite Tuff (*Amakusa*)	6[illegible]	2.263	8.00	7.59	7.80	3.59	1.86	1.26
Rhyolite Tuff (*Iwashiro*)	61a	2.228	10.8	11.1	10.96	6.25	2.22	1.51
	61b	2.198	9.8	9.6	9.67	5.66	2.10	1.67
Rhyolite Tuff (*Tochigi*)	43	1.371	1.43	2.49	1.96	1.06	1.19	0.89
(Diluvium)								
Tuff	36	1.850	35.7	.....	......	6.235	4.39	1.84
Andesite	54	2.557	43.9	45.8	44.9	18.50	4.19	2.69
Andesite	70	2.462	45.5	26.7	36.1	11.69	3.80	2.18
Tuff	30	2.169	28.3	27.6	27.95	10.99	3.59	2.25

Rock	specimen No.	ρ	E_1 (C.G.S.)	E_2 (C.G.S.)	E (C.G.S.)	μ (C.G.S.)	$V_e \frac{\text{kilm.}}{\text{sec.}}$	$V_t \frac{\text{kilm.}}{\text{sec.}}$
Andesite	15	2.201	29.2	23.6	26.38	12.57	3.45	2.39
Tuff	10	2.283	24.3	24.9	24.62	10.74	3.28	2.17
Tuff	14	2.222	21.6	22.8	22.2	8.48	3.18	1.96
Andesite	28	2.165	19.46	27.75	23.51	12.15	3.24	2.37
Andesite	39	2.397	23.07	20.4	21.73	10.13	3.01	2.06
Tuff	20	1.859	14.4	14.5	14.41	5.07	2.79	1.65
Tuff	4a	1.838	10.9	11.86	11.40	4.56	2.99	1.58
	4b	1.817	12.0	12.60	12.33	3.88	2.60	1.46
Andesite	40	2.302	14.76	12.6	13.68	5.99	2.44	1.61
Tuff	57	2.039	11.26	10.70	10.98	5.51	2.32	1.65
Andesite Tuff (*Echizen*)	67a	2.435	13.15	12.77	12.96	5.78	2.31	1.54
	67b	2.400	13.57	13.21	13.39	5.55	2.37	1.52
Andesite	38	1.943	10.30	10.39	10.35	4.13	2.31	1.46
Andesite	49	2.158	8.96	13.1	21.0	5.26	2.26	1.56
Andesite; Tuff	23	1.829	8.23	8.48	8.36	3.92	2.14	1.46
Andesite	34	2.022	9.17	8.44	8.81	6.00	2.09	1.72
Andesite	47	2.425	8.51	8.38	8.45	4.06	1.86	1.29
Tuff (*Izu*)	31	1.915	7.53	5.82	6.68	......	1.86	......
Tuff	33	1.819	6.23	6.42	6.33	......	1.87	......
Andesite	46	2.574	8.87	8.36	8.62	2.92	1.83	1.07
Andesite (*Izu*)	25a	1.984	6.57	5.12	5.85	1.236	1.72	0.79
	25b	1.632	5.57	5.14	5.36	1.63	1.60	0.88
Andesite	48	2.102	5.51	6.81	6.16	2.47	1.71	1.08
Andesite Tuff	21	1.497	3.74	4.12	3.93	1.39	1.62	0.97
Tuff (*Izu*)	35	1.286	3.45	3.31	3.38	1.50	1.62	1.08
Tuff (*Awa*)	44	1.448	2.72	3.87	3.30	1.17	1.50	0.90
Quartz Sandstone	24	2.138	4.04	4.05	4.05	1.30	1.37	0.78
Quartz Sandstone	37	2.230	4.02	......	4.02	......	1.34	......

Some of the specimens which have been examined are nearly isotropic. Most of these rocks are of recent formation. For these, I have calculated the velocities of propagation of longitudinal waves in unlimited medium $V = \sqrt{\frac{\lambda+2\mu}{\rho}} \left(= \sqrt{\frac{k+\frac{4}{3}n}{\rho}}\right.$ using Lord Kelvin's notation$\left.\right)$, which are placed under the following table.

Rock	Age	Density	$V = \sqrt{\frac{\lambda+2\mu}{\rho}} \left(\frac{\text{kilm.}}{\text{sec.}}\right)$
Peridotite Serpentine	Algonkian	2.786	5.86
Marble	Palaeozoic	2.654	4.09
Weathered clayslate	,,	2.490	2.25
Idzumi sandstein	Mesozoic	2.236	2.93
,,	,,	2.223	2.76
Tuff sandstone	Tertiary	2.321	3.35
,,	,,	2.305	3.16
RhyoliteTuff	,,	2.316	3·18
,,	,,	2.346	3.11
Rhyolite	,,	1.944	3.02
,,	,,	2.454	2.78
Rhyolite Tuff	,,	2.228	2.25
,,	,,	2.198	2.14
,,	,,	2.263	1.88
Tuff	Diluvium	2.557	4.44
,,	,,	2.167	4.02
,,	,,	2.222	3.77
,,	,,	2.283	3.38
Andesite	,,	2.397	3.06
Tuff	,,	1.838	2.75
Andesite Tuff	,,	2.014	2.58
Andesite	,,	2.547	2.57
,,	,,	1.943	2.54
Andesite Tuff	,,	2.400	2.50
,,	,,	2.435	2.35
Tuff	,,	2.039	2.32
Andesite	,,	2.022	2.21

I did not think it necessary to calculate the velocity of surface waves, which according to Lord Rayleigh amounts to $0.9554 \sqrt{\frac{\mu}{\rho}}$, as the difference of rigidity in different specimens is so great that the presence of the factor 0.9554 will not materially affect the result.

General Result.—In examining the elastic constants of rocks classified according to the age of formation, we find a distinguished gradation as we pass from those of recent formation to the oldest. The increase of density as well as the quasi-crystalline behaviour of rocks are the most important characteristic of rocks, which are deeply embedded in the earth's crust. The chlorite schist of Chichibu has a density nearly equal to 3, although its modulus of elasticity is greater than that of brass or copper with a rod cut in the direction of strongest tenacity, it is so brittle in the direction perpendicular to it that it is impossible to obtain a single specimen with which the elastic constant can be accurately determined. The elastic constants are widely different as the specimen is cut in one or other direction especially in archaean and palaeozoic rocks, as schists and slates with distinct sedimentation planes. Rocks of eruptive origin are generally free from such directional behaviour, but when they are pressed or otherwise subject to continuous application of stress, the difference of elasticity in different directions can still be traced. Such appears to be the case with marble and granite.

The elastic constants of archaean and palaeozoic rocks are far superior to those of the cainozoic, but the velocity of propagation of longitudinal or transversal waves is not proportionally large. As the ratio of the elastic constant to density determines the velocity of propagation, we can not at once conclude from the increase of elasticity that the waves travel with greater velocity. It would be too bold to draw anything like a general conclusion from the examination of some eighty specimens, but so far as the present experiments go, the tendency is such that the elastic constants increase more rapidly than the density as the rock becomes denser ,and consequently elastic waves travel with greater velocity in the interior than on the surface of the earth's crust. Eruptive rocks are more isotropic than those of nonigneous origin, and have inferior elasticity, but there is the same distinction with age. Elastic waves in eruptive palaeozoic rocks travel with slower velocity than in those of the archaean of the same origin ; a similar remark applies to cainozoic rocks with a few exceptions.

As we go deep in the earth's crust the rocks generally assume schistose structure, we have reason to believe that the elastic constants of the constituent rocks increases in a certain particular direction, which evidently conicides with that of swiftest propagation of elastic disturbance. Pressed by the weight of the superincumbent crust these rocks will be of greater density, so that the increase of elastic constants is attended with corresponding increase of density. We can not conceive that the elastic constant nor the density will continually increase as we approach the centre of the earth ; they will both attain asymptotic values. The alternatives are either the ratio of elastic constants to density goes on gradually increasing, or it first reaches a maximum and then goes on decreasing. The former supposition makes the velocity of elastic waves increase from the surface towards the centre of the earth, while the latter implies the existence of *the stratum of maximum velocity of propagation*. Such a stratum, if it exists, will lie pretty deep in the earth's crust and will be inaccessible to us, but the question will be settled by the seismologists.

Velocity of Propagation of Seismic Waves.—A glance at the table of elastic constants will show the complex elastic nature of rocks composing the earth's crust. The path pursued by waves of disturbance must necessarily assume very complicated forms, as they are subject to manifold reflection, refraction, and dispersion. We can perhaps borrow analogy from a kindred optical phenomenon of curved rays in a medium of heterogeneous density, studied experimentally by Macé de Lépinay and Perot, and theoretically discussed by A. Schmidt and

Wiener. The phenomena presented by the seismic wave will be of still more complex character as the medium is of quasi-crystalline nature, and the wave may suffer refraction something akin to that of light in iceland spar and arragonite. The elastic constants of rocks through which the disturbance propagates will rarely satisfy the condition of giving rise to purely longitudinal or distortional waves, so that the seismic wave will be of a mixed character. What Mr. Milne designates earthquake echos or reverberations will partly find explanation in the intricate behaviour of diverse rocks against the elastic wave travelling through them. The waves propagating from the centre of disturbance will appear on the seismograph as undulations of irregular periods, especially near the origin. At a distance, waves of short period will gradually die out owing to the greater damping effect, while those of long period will still leave their mark, although not felt by us as a shock.

The investigation of the seismic waves affords the best means of feeling the pulse of the interior of the earth ; the elastic nature and the density distribution of the constituent rocks, or even the condition of the inaccessible depth will in some future day be brought to light by the patient study of the disturbance, which traverses the strata of heterogeneous structure and appears as tremors or earthquakes on the earth's surface. I think the introduction of the horizontal pendulum is a great progress in that branch of study, which relates to the earth's interior, not that it records the apparent surface movement of the soil, but that it does not fail to record earthquakes of distant origin, which though insensible to us, sometimes appear as slow waves of gigantic amplitude. By it will be found disturbances, which came through various strata, and probably those travelling through the stratum of maximum velocity of propagation.

Seismic waves travelling through strata of heterogeneous elasticity and density will generally be not purely longitudinal as in the case of sound, nor purely transversal as in the case of light, but a mixture of these two kinds. The velocity of propagation expressed as functions of elastic constants and density is not a simple problem and moreover we do not possess sufficient experimental data to test the result of calculation. The formula $V_1=\sqrt{\frac{E}{\rho}}$ for longitudinal waves in a thin rod will give a rough estimate of the velocity.

From records taken in Italy and Japan, Professor Omori concludes that the velocity of the first tremor is almost always equal to 13 kilometers per sec. The question naturally arises : how can we account for such enormous rate of propagation ? The velocity of plane longitudinal waves in an infinite medium of steel is about 6.2 kil. per sec. ; if we take a rod of steel in place of an uniform medium and give a blow to one of its ends, the longitudinal wave will travel with a velocity of 5.3 kilometres ; if the same experiment be repeated on a piece of iron pyrites cut parallel to its axis of greatest elasticity, the velocity will be 8.4 kil. per second ; in topaz, it will amount to 9 kil. Thus even with substances easily accessible on the earth's surface, we have instances of elastic waves travelling with a velocity of something like 10 kil. In the present experiments the velocity in several primeval rocks ranges from 6 to 7 kil. per sec.; as we go deeper in the crust, we may not fail to find those rocks, whose elastic constants are several times greater than those near the surface. So far as I am aware iron pyrites has the greatest modulus of elasticity among the substances, which have till now been placed under experimental test ; it is about 1.6 times greater than in steel and amounts to 3.5×10^{12} C.G.S units (Voigt). If we now imagine a stratum in which Young's modulus exceeds that of iron pyrites as much as that of iron pyrites exceeds that of steel, we can realize a velocity ascribed by seismologist, had not the increase of density been so great as to bring down the rate of propagation. The velocity of 13 kilm. per second, which is that calculated from the preliminary tremors, will roughly correspond to $E=6.0\times 10^{12}$ and $\rho=3.5$. To speak of the relation between density and elastic constant might seem a little absurd, but in the rocks so far examined, certain relation between these two physical constants seems to exist. Comparing the elastic constants of cainozoic and archaean rocks, we find that with the increase of density from 2 to 3, the modulus of elasticity has increased more than ten times in certain specimens. Thus it would not be a wild conjecture to put $E=6\times 10^{12}$ when the density is 3.5. As the mean density of the earth is little over 5.5, we shall come across a stratum of the density above cited not very far from the surface. These considerations give support to the view above stated that there is a stratum of maximum velocity of propagation.

Elastic waves travel with slow velocity in surface rocks. If the principal shocks in the seismometer record be taken into account, the velocity turns out to be very small and about 3.3 kilm. This evidently is about the mean velocity of propagation in most of the surface rocks, and shows that waves of large amplitude creep along the surface. It is not wonderful that with distant earthquakes, the duration sometimes extends over several hours, as the disturbance travels through strata of different elastic constants and the wave modified in various ways will appear all blended together on the seismograph. Although 3 kilm. may be a mean velocity, there are certain surface rocks in which the velocity is less than a kilometer. The shock at the epicentre may last only for a short time, but the duration at a distance will be lengthened, as the range of velocity is very wide. The disturbance coming from the strata of greatest rate of propagation will first make its appearance as the beginning of the preliminary tremor, followed by waves travelling with slower velocity till the principal shock arrives as surface waves. It will be followed by waves travelling with still slower velocity leaving faint record on the seismograph, till they at length fade away. Neglecting the time of passage from the stratum above mentioned to the surface, it is natural to expect that the duration of the so-called preliminary tremor preceding the earthquake shock increases *linearly* with the distance of the epicentre from the place of observation. The above relation was established from various earthquakes which happened in Japan, recorded by Prof. Omori.

With great earthquakes which are perceptible on a seismograph at very great distances, the duration will continually increase with distance; the disturbance may sometimes propagate still unabated in one or other direction round the earth. If the last mentioned case actually take place, the tremor will probably last even for days. As such records have sometimes been obtained by seismologists, it may not be out of place here to notice the possibility for such undulatory movement of the ground.

In conclusion, I wish to express my thanks to Professor Koto and Mr. Fukuchi for valuable information concerning the geological and petrological character of rocks examined in the present experiment.

Seismic Experiments on the Fracturing and Overturning of Columns.

BY

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I. Introduction.

1. *Object of the experiments.*—For the application of seismology to construction in earthquake countries, it is necessary, besides the observation of the damage due to actual earthquakes, to investigate experimentally the effects of artificial seismic movements on various models of construction, the ultimate object being the calculation of the seismic stability of given structures.

We must naturally begin with the simplest structures. Thus, for instance, in the case of a building, it would be convenient to investigate separately its parts or elements, such as columns, walls, arches, roofs, vaults, etc. In the present paper, which is to be regarded only as a preliminary report, I give the results of experiments on the fracturing and overturning of brick and other columns, carried on in the Tokyo Imperial University between March 1898 and March 1899.

The effects of earthquakes on buildings and works of construction are generally of a complex nature, but may conveniently be divided into two elementary phenomena—fracturing and overturning. The present experiments have accordingly been conducted in these two divisions.

2. *The shaking table.*—Artificial earthquake motion was produced by a *shaking table*,* which consisted of a stout wooden floor properly

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* Designed by Professors Mano and Inokuchi.

mounted on strong supports, and which could be made to move with independent horizontal and vertical simple harmonic motions by means of steam engines. As the maximum ranges of motion of the table amounted to 150 and 90 mm. respectively in the horizontal and the vertical directions, and as the period* could be made as small as 0,2 second, movements much greater than those which took place at Nagoya or Gifu on the occasion of the great Mino-Owari earthquake of Oct. 28th, 1891, could easily be produced. (Fig. 1.)

The shaking table is described by Prof. Mano elsewhere in these volumes.

3. *The vertical motion.*—In ordinary cases, the vertical component of earthquake motion is much smaller than the horizontal. Thus, in the severe Tokyo earthquake of June 20th, 1894, the strong-motion seismograph in the Seismological Institute recorded a maximum horizontal motion of 73 mm. (period 1,8 seconds), while the maximum vertical motion was only 11 mm. In this case, therefore, the damage was essentially due to the horizontal motion, and generally the non-synchronizing in horizontal motion of the walls and roof is doubtless the principal cause of the destruction of brick buildings. Vertical motion, even when very great, seems unable by itself to produce any great damage. Referring to actual instances, we find that one-storied brick buildings are most damaged at the junction of the walls with the roof, and that two-storied ones are similarly most damaged at the junction of the upper walls with the roof, while the lower walls remain uninjured or very little affected. Hence, with respect to the destructive power of the earthquake motion, the vertical component is only of a secondary importance in comparison with the horizontal; in other words, the damage of brick structures may, excepting in cases

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* The *period* in this paper always means the *complete period*.

of foundation sinking, be regarded as due to the horizontal motion. From these reasons, as well as for the sake of simplicity, I have limited myself in the present course of experiments to the consideration of the destructive effects of the horizontal motion.

I may here note that earthquake motion, though sometimes very violent, is continuous and does not consist of isolated jerks or shocks. The idea prevalent among certain engineers that in destructive earthquakes buildings are first uplifted by the vertical motion and then destroyed by being suddenly thrown downwards, is quite erroneous.

4. *The intensity of motion in destructive earthquakes.*—The maximum acceleration of the earth-particles in the Tokyo earthquake of 1894 was, calculating from the values of the range of motion and the period given in § 3, found to be 444 mm. per sec. per sec. In the great Mino-Owari earthquake of 1891, the maximum acceleration and range of motion at Nagoya were respectively 2600 mm. per sec. per sec. and about 233 mm., the corresponding quantities at Gifu and many other places in the meizoseismal area being still greater.*

The motion of the shaking table varied in the present experiments between 30 mm. and 112 mm., its range being thus comparable to that of strong and destructive earthquakes.

5. *Literature.*—The only work of reference is that written by Prof. J. Milne and myself, published in Vol. I of the Seismological Journal of Japan (1892), giving an account of experiment on the fracturing and overturning of columns by horizontally applied motion carried on in 1891 in the Work-shop of the Engineering College of Tokyo Imperial University. The method of experiment then adopted was nearly similar to that in the present instance.

* See the present author's Note on the Mino-Owari Earthquake of 1891.

II. Fracturing of Brick Columns.

6. *Method of experiment.*—The brick columns was firmly fixed to the shaking table and then fractured by giving proper movements to the latter. By the arrangement described in § 7, the motion of the shaking table was mechanically registered in the form of a diagram, while the fracture of the column was carefully watched and its exact moment electrically recorded. The intensity of the motion, which caused the fracture was then calculated from the diagram and compared with the theoretical value of the seismic stability of the column.

7. *Diagram of motion of the shaking table.* (Fig. 2.)—For recording the motion of the shaking table, a pointer *b* is fixed to the latter at *a* and carries a steel pen hinged at its end. This pen writes in ink the motion of the table in its natural size on the white paper *c*, which is wrapped round two parallel cylindrical tubes *e*, whose axes are screw-cut, and which are turned by the clock-work *d*, the paper being displaced transversely 1 cm. per revolution.*

For gauging exactly the motion of the paper *c*, time ticks are marked by means of a small pendulum *f*, 148 mm. long (Fig. 3), which allows an electric current to pass through the coil *h* (Fig. 2) each time its index crosses the mercury pool *g*. The complete period of vibration of the pendulum was 0,77 sec.

The moment of fracture of the brick column was signalled by pressing a key, which caused an electric current to pass through the coil *i* (Fig. 2).

The amplitude and period of the shaking table which caused the fracture of the brick columns, can thus be exactly measured from the diagram. In a few cases the movements of the tops of these columns have likewise been recorded. For illustrations, see figs. 8-15.

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* This record-receiver forms part of Professor Tanakadate's Seismograph.

8. *The brick columns.* (See Figs. 4 and 5.)—Twenty-four brick columns, numbered from 1 to 24, were used in the experiments, their details being given in table I. As, however, some of the columns were used several times over, the total number of the fracturing experiments amounted to fourty-four. The columns were constructed between March 20th and May 22nd, 1898, and the fracturing experiments were carried on between Oct. 12th, 1898, and Feb. 15th, 1899.

The three columns Nos. 22, 23 and 24, were composed of ordinary bricks, while all the rest were composed of specially made small bricks. Thus the nine columns Nos. 1 to 9, and the remaining twelve Nos. 10 to 21, were composed of bricks, whose linear dimensions were respectively one-fifth, and one-half of that of the ordinary bricks.

The largest columns were Nos. 16–24, whose heights varied between 1580 and 1810 mm., and whose sectional areas between $\overline{230}^2$ and $\overline{233}^2$ sq. mm. Nos. 19, 20 and 21, were hollow, all the others being solid. As in experiments of this nature it is absolutely necessary that the bodies should be uniform in tensile strength, each column was constructed of bricks of one and the same quality.

TABLE I.—Brick Columns Used in the Fracturing Experiments

2 x = thickness of column ; 2 y = height of column.

No.	2 x. mm.	Sectional area. (sq. mm.)	2y. (mm.)	Remarks.	Quality of brick.	Date of construction of the column.
1	96	96 × 120	720	$\frac{1}{5}$-bricks, 50 layers.	Ordinary 1st class.	April 6th, 1898.
2	”	”	”	” ”		”
3	”	”	”	Same as Nos. 1 and 2, furnished with a concrete base box.		May 21st, 1898.
4	97	$\overline{97}^2$	730	$\frac{1}{5}$-bricks, 50 layers.	”	May 16th, 1898.
5	”	”	”	” ”		”
6	”	”	”	Same as Nos. 4 and 5, furnished with a concrete base box.		May 21st, 1898.
7	72	97 × 72	720	$\frac{1}{5}$-bricks, 50 layers.	”	May 19th, 1898.
8	”	”	”	” ”		”
9	”	”	”	Same as Nos. 7 and 8, furnished with a concrete base box.		May 21st, 1898.
10	111	$\overline{111}^2$	902	$\frac{1}{2}$-bricks, 25 layers.	Ordinary 2nd class.	March 26th, 1898.
11	”	”	”	” ”		”
12	110	$\overline{110}^2$	915	Same as Nos. 10 and 11, furnished with a concrete base box (240 × 240 × 145).		April 2nd, 1898.
13	110	110 × 232	1800	$\frac{1}{2}$-bricks, 50 layers.	”	March 31st, 1898.
14	”	”	”	” ”		”
15	”	”	”	Same as Nos. 13 and 14, furnished with a concrete base box (330 × 460 × 180).		”
16	230	$\overline{230}^2$	1744	$\frac{1}{2}$-bricks, 50 layers.	”	March 20th, 1898.
17	”	”	”	” ”		”
18	”	”	”	” ”		”
19*	233	$\overline{233}^2$-$\overline{131}^2$	1810	$\frac{1}{2}$-bricks, 50 layers ; hollow.	”	March 24th, 1898.
20*	”	”	”	” ” ”		”
21*	”	”	”	Same as Nos. 19* and 20*, furnished with a concrete box (460 × 460 × 180).		March 31st, 1898.
22	230	$\overline{230}^2$	1580	Ordinary bricks, 23 layers.	Extra 2nd class.	April 3rd, 1898.
23	”	”	”	” ”	”	”
24	220	$\overline{220}^2$	1587	Same as Nos. 22 and 23, furnished with a concrete base box (460 × 460 × 280).	Extra 1st class.	May 22nd, 1898.

* Those marked with asterisks (*) are hollow columns, the inner side being = 131 mm.

For all the columns one and the same mortar, consisting of one part of cement and two parts of sand, was employed, the tensile strength, as shown in table II, being generally high. The object of the use of such a good mortar in the fracturing experiments was to avoid as far as possible, discordance in the results. If the morter have been of a bad quality, the strength of the column might very easily have been modified by the non-uniformity of the joints, thereby rendering difficult the comparison of experiment and theory.

The small bricks used in columns Nos. 1-21 were inferior in quality, being of the kinds known as *ordinary first* and *ordinary second class* bricks. For our experiments, however, this circumstance was quite favourable, since the tensile strength of the mortar in these columns was nearly equal to that of the bricks themselves, so that the breaking of a brick column, which usually took place at a joint, was rarely caused by the clean separation of mortar and bricks, the fracture, in the majority of cases, occurring through the mortar and the brick. Such columns may therefore be regarded as having nearly a uniform tensile strength throughout.

As remarked in § 12, the tensile strength of the mortar was higher when used with bricks of superior quality than when used with those of inferior quality.

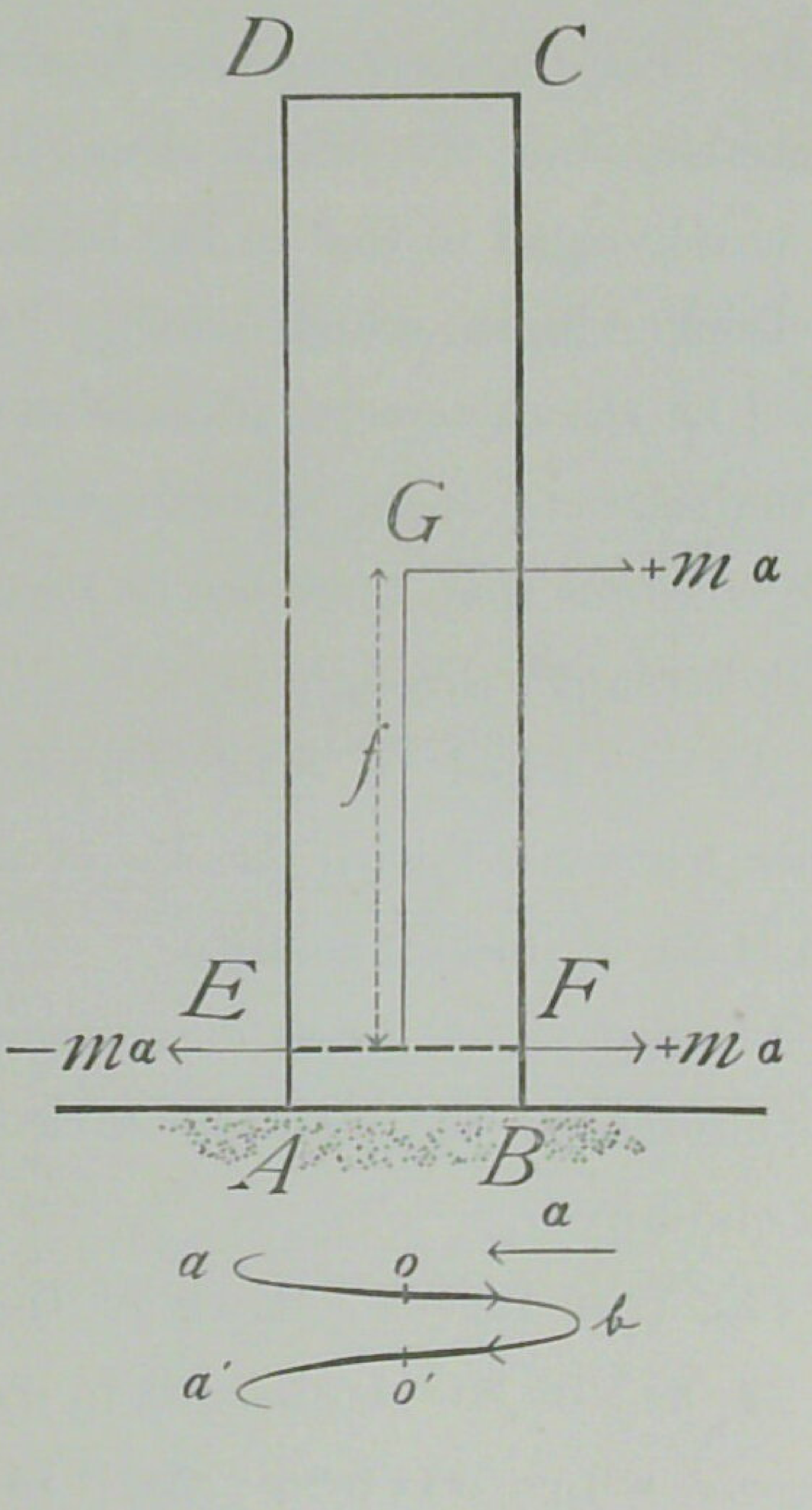
9. *Methods of fixing the columns to the shaking table.*—For fixing the columns to the shaking table, the following two methods have been employed.

(1). (See Fig. 5.) Each of the columns Nos. 3, 6, 9, 12, 15, 21 and 24, had its base embedded in concrete contained in a stout wooden box *a*, which was firmly fixed to the table by means of two strong wooden beams *b*, and of four bolts *c*.

(2). The rest of the columns, which were free and had no concrete base, were fixed to the table by means of iron frames *a*,* shown in Fig. 6.

10. *Formulæ.*—To obtain formulæ by which the seismic stability of brick columns can be calculated, it is necessary to examine the action of earthquake motion on such bodies.

The complete period of the principal vibration of destructive earthquakes is probably one or two seconds, and its range of motion varies from some 50 mm. to upwards of 200 mm.** Consequently a brick column would not be overthrown in a direction opposite to the motion of the ground, unless its height be comparatively very great. In this section, I shall confine myself to the consideration of *short* columns, viz. those whose height is not *infinitely* great in comparison with its thickness and the range of the earthquake motion. In such columns, a centre of percussion does not exist, as will be seen from the following consideration.



Let ABCD be a brick column fixed at AB to the ground. Supposing that the ground is, at a given moment, moving from *a* towards *b*, the acceleration of the earth-motion is zero at the equilibrium position, *o*, and gradually increases to the maximum at the right hand extremity *b*. Thereafter the acceleration decreases and becomes again zero at the equi-

* Designed by Messrs Mano, Inokuchi and Yasunaga.

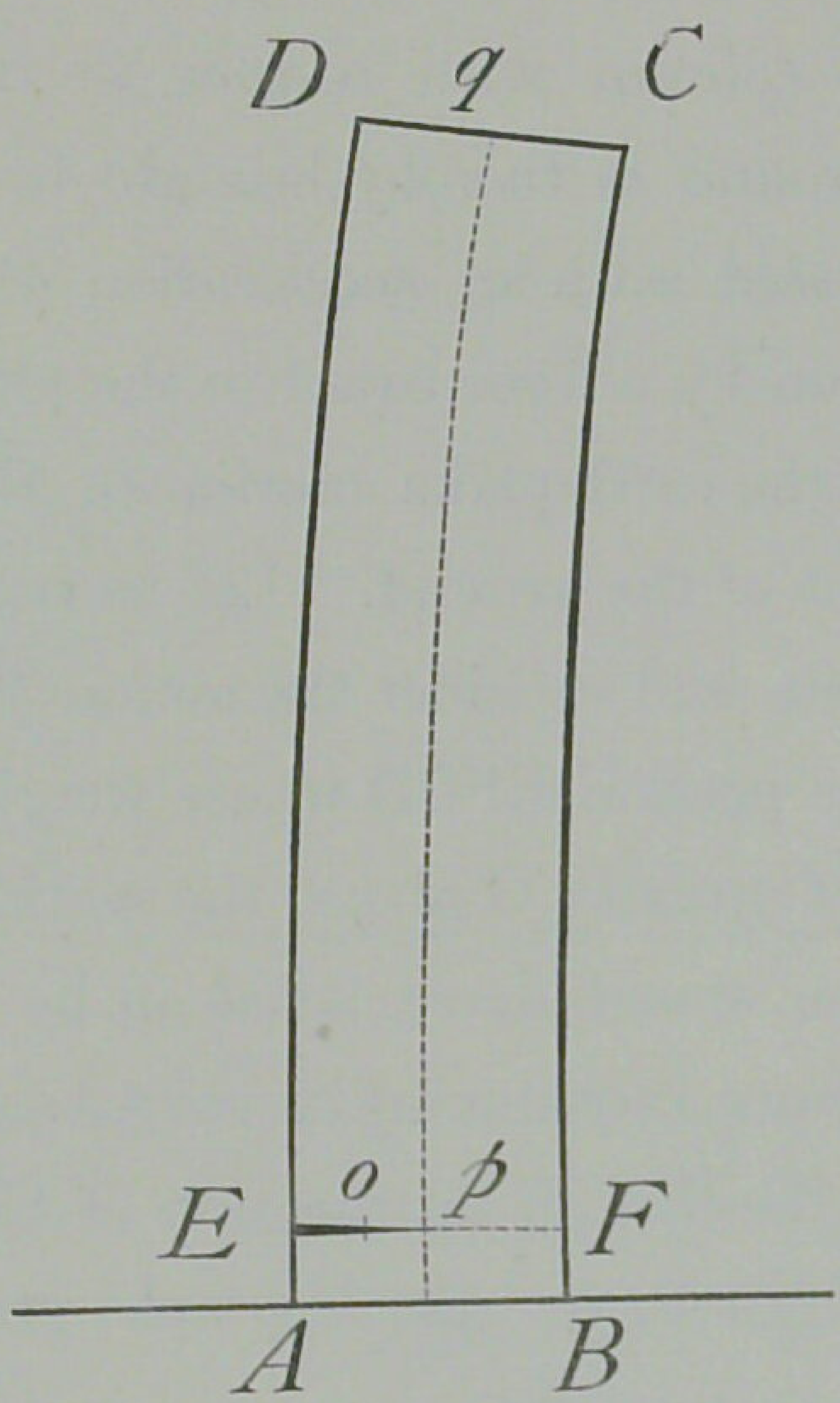
** See the present Author's Notes on the earthquakes of Tokyo in 1894 and of Mino-Owari in 1891.

librium point o'. (In the figure, the two paths aob and $bo'a'$ have for the sake of clearness been separately drawn. These two are of course one and the same, o' and a' coinciding respectively with o and a.) Now if we impress uniformly to the column and to the ground an acceleration $+\alpha$ equal and opposite to that of the latter, which may be denoted by $-\alpha$ and is directed towards the equilibrium position o, the relative motion of the column with respect to the ground would not be changed. The ground is thereby brought to a state of rest, while the column is impressed with an acceleration $+\alpha$, and therefore may be regarded as acted on by a force equal to the product of its mass and the acceleration of the earthquake motion in the same direction as the actual displacement of the ground. Let us now take any given section EF of the column and consider the action by which the latter may be fractured. The portion EFCD whose weight is W and the height of whose centre of gravity G above the section EF is f, is, according to what has been stated above, acted on by a horizontal force $\frac{\alpha W}{g}$ applied at G. Hence, introducing two equal and opposite forces $+\frac{\alpha W}{g}$ and $-\frac{\alpha W}{g}$ along the line of intersection of the section EF and the principal plane through G, we get a couple producing flexure of the column and having a moment

$$M = \frac{f\alpha W}{g}. \qquad (1)$$

The shearing force $+\frac{\alpha W}{g}$ in the plane EF may be neglected in comparison with the bending moment (1), when the height of the column is many times greater than its thickness. If we denote by T the complete period and by a the amplitude, or semi-range, of the earthquake motion, the acceleration α varies between zero and the maximum value $\frac{4\pi^2 a}{T^2}$. I shall hereafter regard the α in equation (1) as denoting the maximum acceleration, and consequently the maximum value of M as denoting the fracturing power of the earthquake motion.

Flexure of the column.—Let us assume the column to have a central axis, (pq), that is either to be uniform in section or to be such that the centres of inertia of all the section lie in a straight line. If now the column ABCD be slightly deflected by a horizontal force normal to the axis pq, the plane which originally passed through pq and was normal to the direction of the force becomes a *neutral surface*, undergoing curvature only and remaining unchanged in length, while the longitudinal filaments on each side of the surface suffer respectively contraction and elongation. The longitudinal tension (or pressure) P, at any given point o, is given by the following equation :—



$$P = \frac{M}{I} x, \tag{2}$$

in which x is the distance op, I the moment of inertia of the section EF with respect to its line of intersection with the neutral surface, and M the bending moment, which in the case of the earthquake motion is given by equation (1). Substituting the value of M from equation (1), we obtain

$$P = \frac{x f a W}{I g},$$

so that P is proportional to x and its maximum value occurs when $x=x_0$, supposing the thickness of the column $=2x_0$. Thus

$$\mathrm{P}\,(\text{maximum}) = \frac{x_0 f\, a\, \mathrm{W}}{\mathrm{I}g}.$$

When P is sufficiently large and equal to F the tensile strength of the column, the latter will be cracked at the side E, and then we obtain

$$\mathrm{F} = \frac{f\, a\, \mathrm{W}\, x_0}{\mathrm{I}g},$$

or

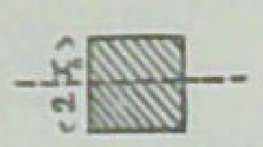
$$a = \frac{\mathrm{I}\, g\, \mathrm{F}}{x_0 f\, \mathrm{W}} = \frac{\mathrm{I}\, g\, \mathrm{F}}{x_0 f\, w\, \mathrm{V}}, \qquad (3)$$

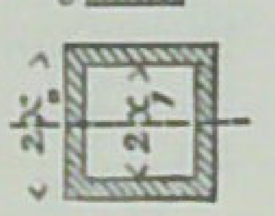
in which V is the volume of the portion fractured and w the weight per unit volume of the column.

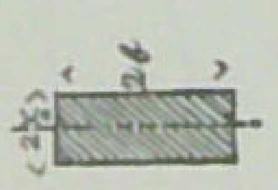
Fracture of the column. The *fracture* of the column here means simply the production of cracks at the side E, and hence it is a phenomenon totally different from its *overturning*. Equation (3) gives the acceleration of the earthquake motion capable of fracturing a given brick column.

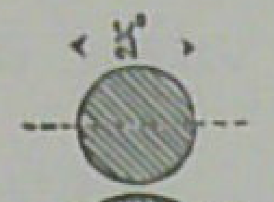
11. *Remarks on equation* (3).—a denotes the maximum acceleration of the earthquake motion in mm. per second per second. $g=9800$ mm. F is the tensile strength of the column in lbs. per sq. in. w is the weight per unit volume of the column, and is in ordinary cases equal to 0,0603 lb. The remaining quantities, I, x_0, f and V, are all to be expressed in inches.

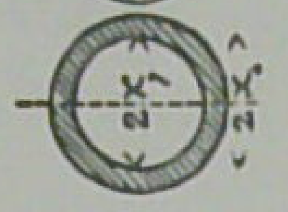
I shall next give the values of I in a few common cases.

(*a*)  Section square ; each side$=2x_0$. $\mathrm{I} = \frac{4}{3}x_0^4$

(*b*)  Section hollow-square ; outer side$=2x_0$, inner side$=2x_1$. $\mathrm{I} = \frac{4}{3}(x_0^4 - x_1^4)$

(*c*)  Section rectangular ; the thickness, or dimension in direction of the earthquake motion$=2x_0$, and the other side$=2b$. $\mathrm{I} = \frac{4\, b\, x_0^3}{3}$.

(*d*)  Section circular ; diameter$=2x_0$. $\quad \mathrm{I}=\frac{\pi x_0^4}{4}$.

(*e*)  Section annular ; outer diameter$=2x_0$,
inner diameter$=2x_1$. $\quad \mathrm{I}=\frac{\pi(x_0^4-x_1^4)}{4}$.

Further if the section be uniform, we obtain from (*a*), (*b*) and (*c*) :

(Section square) $$a=\frac{2g\mathrm{F}x_0}{3w.\,2f^2} \tag{4}$$

(Section hollow-square) $$a=\frac{2g\,\mathrm{F}(x_0^2+x_1^2)}{3x_0.\,w.\,2f^2} \tag{5}$$

(Section rectangular) $$a=\frac{2g\mathrm{F}x_0}{3w.\,2f^2} \tag{6}$$

The columns of square and rectangular sections lead naturally to one and the same result, provided their dimensions in the direction of the earthquake motion be equal to one another. Equations (4) and (5) are the working formulæ in the discussion of the fracturing experiments.

12. *Tensile strength of the brick columns.*—To calculate by equations (3), (4) and (5) the seismic stability of the columns, it is necessary first to find out the value of the tensile strength ; consequently I made a number of experiments by means of a testing machine in the Work-shop of the Engineering College, on the tensile strength of the fragments of the columns used in the fracturing experiments. The results are given in table II.

The pieces of the brick-work tested were cut into ordinary shapes, as shown in the figure, and the stretching load was applied always very gradually. In the table, the time intervals between the first application of the load and the breaking of the test pieces are given only in a few cases as illustrations.

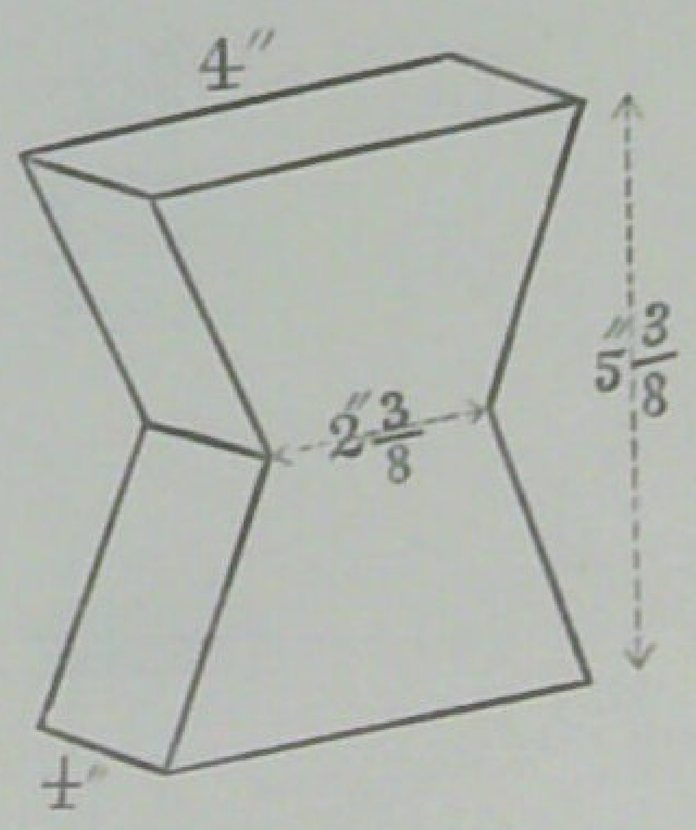


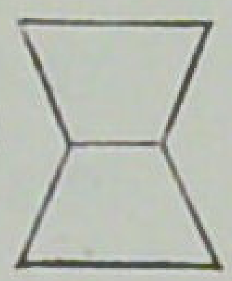
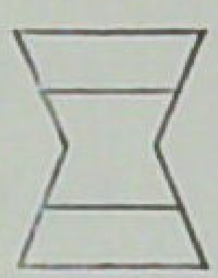
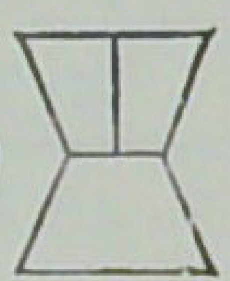
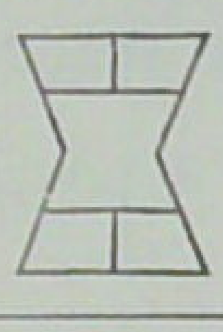
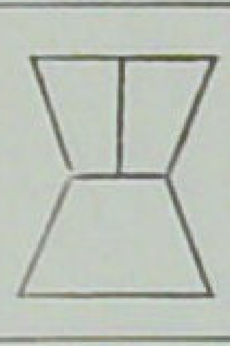
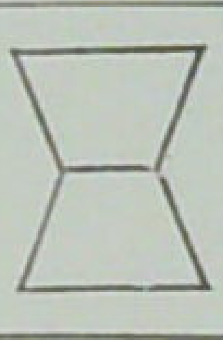
TABLE II.—Tensile Strength of the Brick Columns.

(*a*) Columns Nos. 1, 3, 6, 7, 9, 11, 14, 16, 17 and 19.*

No. of Column.	No. of Expt.	Sectional area. (sq. in.)	Tensile strength. (lbs. per sq. in.)	Time interval between the first application of load and the occurrence of fracture.	Date of the testing expt.
1	1	9.38	47.6	...	Dec. 14th, 1898.
	2	,,	58.5	...	
	3	,,	77.0	...	
	4	,,	38.2	...	
			mean 55.3		
3	1	10.3	59.8	1^{m} 13^{s}	,,
	2	9.4	47.7	0 34	
	3	,,	82.3	0 17	
	4	10.3	50.0	0 11	
	5	9.4	35.8	...	
			mean 55.1		
6	1	9.4	42.3	0 54	Dec. 16th, 1898.
	2	,,	62.6	1 17	
	3	10.3	64.1	1 37	
	4	6.9	10.8	1 57	
			mean 69.3		
7	1	7.56	52.1	...	Dec. 14th, 1898.
	2	8.25	24.4	...	
	3	6.88	52.9	...	
	4	,,	48.4	...	
	5	7.56	41.4	...	
			mean 43.8		
9	1	6.9	47.2	0 52	Dec. 16th, 1898.
	2	,,	53.7	...	
	3	,,	71.6	1 24	
	4	,,	62.6	...	
	5	8.3	58.3	1 16	
			mean 58.8		

No. of Column.	No. of Expt.	Sectional area. (sq. in.)	Tensile strength. (lbs. per sq. in.)	Time interval between the first application of load and the occurrence of fracture.	Date of the testing expt.
				m s	
11	1	10.6	21.6	0 9	Dec. 14th, 1898.
	2	,,	30.3	0 15	
	3	,,	44.9	0 41	
	4	,,	88.7	...	
	5	,,	49.1	1 10	
			mean 53.3		
14	1	10.0	36.4	0 55	,,
	2	,,	38.6	0 39	
	3	13.0	32.4	0 22	
	4	10.0	34.7	0 31	
	5	,,	36.4	0 46	
			mean 33.7		
16	1	10.0	53.2	...	,,
	2	,,	58.8	1 14	
	3	,,	47.0	...	
	4	,,	63.2	0 44	
	5	12.0	34.0	...	
			mean 51.2		
17	1	10.0	29.1	0 42	Dec. 16th, 1898.
	2	,,	61.6	1 34	
	3	,,	36.9	0 37	
	4	,,	19.6	0 9	
	5	12.0	40.1	1 4	
	6	10.0	60.4	0 20	
			mean 41.3		
19*	1	5.0	84.0	...	Dec. 14th, 1898.
	2	,,	45.9	...	
	3	,,	50.9	...	
	4	,,	39.2	...	
	5	,,	101.	...	
			mean 64.2		

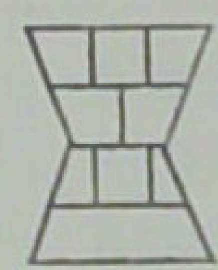
(*b*) Column No. 23.

Form of the brick-work test piece.	No. of expt.	Sectional area. (sq. in.)	Tensile strength. (lbs. per sq. in.)	Time interval between the first application of load and the moment of fracture.		Date of the testing expt.	Date of the construction of the column.	Remarks.
				m	s			
(A) 	1	9.69	65.3	1	37	Feb. 4th, 1899.	May 22nd, 1898.	Separated at the joint.
	2	,,	43.5		35			,,
	3	,,	76.2	1	27			,,
	4	,,	93.9	1	8			,,
		mean......	69.7					
(B) 	1	10.0	143.6	2	15	,,		Broke through brick at the section of minimum area.
	2	,,	163.7		,,			,,
	3	,,	169.5		——			,,
	4	,,	149.4	1	38			,,
		mean......	156.6					
(C) 	1	9.69	108.9	1	59	,,		Separated at the joint of the minimum section.
	2	,,	92.8	1	34			,,
	3	,,	141.4		——			,,
	4	,,	153.4	2	55			,,
		mean......	124.1					
(D) 	1	9.69	149.6		——	,,		Broke through brick at the section of minimum area.
	2	,,	142.7	1	37			,,
		mean......	146.2					
(E) 	1	10.0	122.7	1	31	,,		,,
	2	11.0	127.7	1	48			,,
		mean......	125.2					
(F) 	1	9.51	51.3		——	Feb. 26th, 1899.		Separated cleanly at the joint.
	2	9.46	32.5		——			,,
		mean......	41.9					
(G) 	1	9.51	29.9		——	,,		,,
	2	9.42	36.7		——			,,
		mean......	33.3					

The mean deduced from (B), (D) and (E), which gives the tensile strength of the bricks, is 142 lbs. per sq. in., while that deduced from (A), (C), (F) and (G), which gives the strength of the mortar and brick joint, is 77.1 lbs. per sq. in. The mean value of 37.6 lbs. per sq. in., obtained from (F) and (G), is to be regarded as representing the weakest strength of the column.

The value of 53.6 lbs. per sq. in., meaned from (A), (F) and (G), is to be used for the columns, Nos. 23 and 23′.

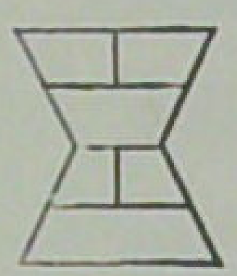
(*c*) Column No. 18.

Form of the brick-work test piece.	No. of expt.	Sectional area. (sq. in.)	Tensile strength. (lbs. per sq. in.)	Form of the brick-work test piece.	No. of expt.	Sectional area. (sq. in.)	Tensile strength. (lbs. per sq. in.)
	1	10 9	38.6		1	9.27	42.4
	2	9.62	50 2		2	9.61	52.4
	3	12.1	30.3		3	10.9	39.0
(A)	4	9.2	67.8	(B)	4	Simply dropped apart during fixing	0
	5	9.2	42.3		5	9.28	36.9
	6	10.1	69.6		6	9.35	44.0
	7	9.47	56.5		7	9.35	61.6
	8	9.29	22.8		8	9 41	30.9
	9	9.34	27.9		9	9.50	60.1
	10	11.3	38.6		10	9.41	20.7
		mean...	44.5			mean...	43.1

General mean = 43.8 lbs. per sq. in.

The date of construction of the column was March 20th, 18 98 ; and tha of the testing experiments Feb. 21st, 1899. The test pieces were fractured in general, partly through brick and mortar and partly by the separation of these two substances.

(*d*) Column No. 13.

Form of the brick-work test piece.	No. of expt.	Sectional area. (sq. in.)	Tensile strength. (lbs. per sq. in.)	Form of the brick-work test piece.	No. of expt.	Sectional area. (sq. in.)	Tensile strength. (lbs. per sq. in.)
	1	9.27	17.6		1	9.46	13.1
	2	9.61	57.5		2	10.0	55.5
(A)	3	9.51	54.1	(B)	3	9.57	24.9
	4	11.9	29.9		4	9.87	25.2
	5	Simply dropped apart during fixing.	0.		5	11.4	35.5
	6	9.46	13.1		6	9.27	46.5
	7	9.77	34.6		7	9·46	30.6
					8	9.61	42.1
		mean...	34.4			mean...	34.1

General mean = 34.2 lbs. per sq. in.

The date of construction was March 20th, 1898 ; and that of the testing experiment Feb. 26th, 1899. The brick pieces were fractured, in general, partly through brick and mortar and partly by the separation of these two substances.

The mean deduced from the eight smallest values of the tensile strength is 19.3 lbs. per sq. in , which is to be used for the calculation of the seismic stability of the column No. 13.′

(*e*) Column No. 20.*

No. of expt.	Sectional area. (sq. in.)	Tensile strength. (lbs. per sq. in.)	No. of expt.	Sectional area. (sq. in.)	Tensile strength. (lbs. per sq. in.)
1	5.55	65.1	6	5.01	62.9
2	4.94	47.6	7	4.84	35.9
3	5.09	67.9	8	5.00	62.2
4	6.18	38.2	9	5.00	84.6
5	5.10	48.4	10	5.10	66.5

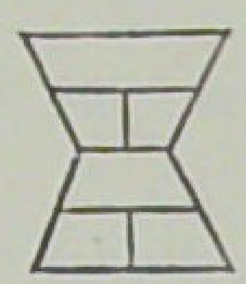
General average = 57.9 lbs. per sq. in.

The date of construction of the column was March 24th, 1898 ; and that of the testing experiments March 10th, 1899.

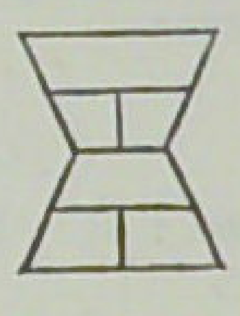
The form of the brick-work test pieces was as shown in the figure.

In the testing experiments, the load or stretching weight was applied always very gradually.

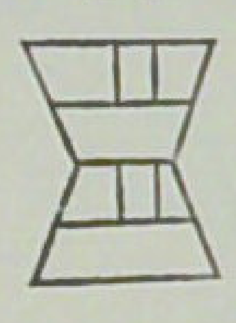
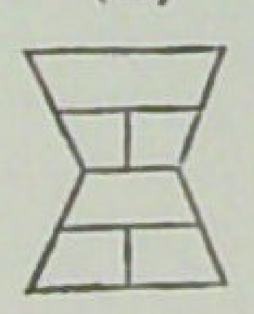
(*f*) Columns Nos. 10, 12 and 15.

No. of column, and the form of the brick-work test piece.	No. of expt.	Sectional area. (sq. in.)	Tensile Strength. (lbs. per sq. in.)	Date of construction of the column.	Remarks.
No. 10. 	1	9.44	52.2	March 26th, 1898.	Broke through brick and mortar.
	2	10.4	76.5		Broke through brick.
	3	8.90	38.0		Partly broke through brick and partly separated at the joint.
		mean....	55.6		

Tensile strength meaned from (1) and (3) = 45.1 lbs. per sq. in. This is to be used for the calculation of the stability of the column.

No. 12. 	1	10.5	77.4	April 2nd, 1898.	Broke through brick.
	2	9.22	28.5		Separated cleanly at the joint.
	3	,,	31.9		Separated at the joint, a portion of the mortar being also broken.
	4	,,	34.0		Separated at the joint, a portion of the brick face being scratched off.
	5	,,	39.0		Separated cleanly at the joint.
		mean....	42.2		

Tensile strength meaned from (2) ...... (5) = 33.4 lbs. per sq. in. This is to be used for the calculation of the stability of the column.

No. 15. (A) 	1	8.81	47.8	March 31st, 1898.	Separated at the joint, a portion of the brick face being scratched off.
	2	9.79	39.0		Partly broke through mortar and partly separated at the joint.
	3	10.4	38.2		Broke through brick.
	4	9.44	70.2		,,
	5	9.60	54.5		,,
		mean....	49.9		
(B) 	1	9.44	63.2		Broke through brick.
	2	10.6	17.0		Separated cleanly at the joint.
	3	9.79	52.8		,, ,,
	4	9.44	39.8		Partly broke through brick and partly separated at the joint.
	5	9.79	32.2		Separated cleanly at the joint.
	6	7.42	39.5		,, ,,
	7	7.42	31.2		,, ,,
	8	—	0		Simply dropped apart.
		mean....	39.4		
mean of (A) and (B)......			44.7		

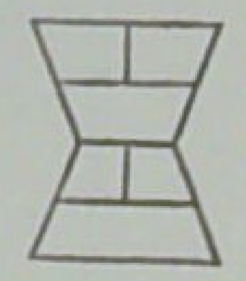
Tensile strength, meaned from (A) 1 and (B) 2, 3, 5, 6, 7, 8 = 30.2 lbs. per sq. in. This is to be used in the calculation of the stability of the column No. 15."

(*g*) Column No. 21.*

Date of expt.	No. of expt.	Sectional area. (sq. in.)	Tensile strength (lbs. per sq. in.)	Date of expt.	No. of expt.	Sectional area. (sq. in.)	Tensile strength, (lbs. per sq. in.)	Remarks.
March 8th, 1899.	1	5.12	65.1	April 5th, 1899.	11	5.04	56.2	Separated at the joint.
	2	4.93	41.8		12	4.75	82.1	Broke through bricks.
	3	4.74	30.4		13	6.50	55.1	Separated at the joint.
	4	5.00	12.2		14	4.75	51.8	,,
	5	5.27	61.5		15	5.30	64.5	Broke through bricks.
	6	5.09	31.8		16	4.75	49.3	Separated at the joint.
	7	,,	57.4		17	5.03	81.9	Broke through bricks.
	8	5.01	26.7		18	———	0	Simply dropped apart.
	9	5.09	49.2		19	4.75	32.4	Separated cleanly at the joint.
	10	5.14	50.2		20	,,	28.9	,,
					21	,,	38.8	Separated at the joint.

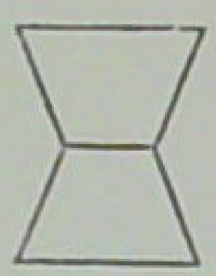
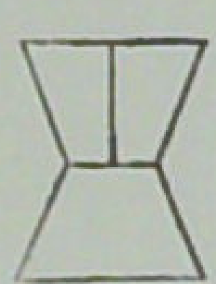
The average tensile strength = 46.1 lbs. per sq. in, which value is to be used for the calculation of the stability of the column No. 21.'* The 11 least among the above 21 determinations of the tensile strength, however, give an average value of 31.3 lbs. per sq. in., which is to be used in the calculation of the column No. 21.*

The form of the brick-work test piece was as shown in the figure.



In the testing experiments, the load or stretching weight was applied always very gradually.

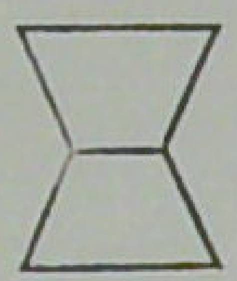
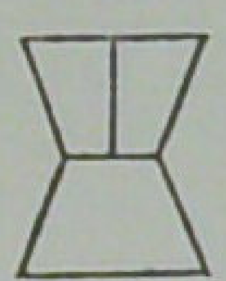
(*h*) Column No. 22.

Form of the brick-work test piece.	No. of expt	Section-al area. (sq. in.)	Tensile strength. (lbs. per sq. in.)	Remarks.
	1	9.05	149	Broke through mortar.
	2	9.04	113	"
	3	9.04	111	Separated at the joint, a portion of the brick face being scratched off.
(A)	4	9.44	125	Broke through mortar, partly also through brick.
	5	9.04	160	" "
	6	—	0	Simply dropped apart during the fixing.
	7	9.09	142	Broke through mortar.
	8	8.69	48.2	Separated clearly at the joint.
	9	9.26	137	Broke through mortar.
	10	8.93	58.7	Separated at the joint.
	11	9.26	34.5	Separated cleanly at the joint.
	12	9.15	75.8	Separated at the joint.
	13	8.96	153	Broke through mortar.
		mean	100.6	
	1	8.90	115	Broke through mortar.
	2	"	152	Partly broke through mortar and partly separated at the joint.
	3	"	123	Broke through mortar, a portion of the brick face being scratched off.
	4	"	156	Broke through mortar.
(B)	5	"	107	"
	6	8.96	115	"
	7	9.30	44.0	Separated cleanly at the joint.
	8	"	55.3	Separated at the joint.
	9	9.09	82.2	Separated at the joint, a portion of mortar being also broken.
	10	"	48.4	Separated cleanly at the joint.
	11	8.81	126	Broke through mortar.
	12	9.09	81.6	Separated cleanly at the joint.
		mean	100.4	

The general average value of the tensile strength =100.5 lbs. per sq. in. The date of construction of the column is April 3rd, 1898; and that of the testing experiments March 28th and 30th, 1899.

In the testing experiments, the load or stretching weight was applied always very gradually.

(*i*) Column No. 24.

Form of the brick-work test piece	No. of expt	Sectional area. (sq. in.)	Tensile strength. (lbs. per. sq. in.)	Remarks.
	1	9.2	209.	Separated at the joint, the brick face being almost entirely scratched off.
	2	9.46	61.	Separated at the joint, half of the brick face being scratched off.
	3	9.29	119.	Separated at the joint, about two-thirds of the brick face being scratched off.
(A)	4	9.31	145.	Separated at the joint.
	5	9.75	78.	Separated cleanly at the joint.
	6	9.26	132.	Separated at the joint.
	7	"	132.	"
	8	9.47	114.	"
	9	9.26	28.8	Separated cleanly at the joint.
	10	9.20	195.	Separated at the joint, the entire brick face being scratched off.
	11	9.20	208.	" "
	12	9.29	125.	Separated at the joint, a portion of the brick face being scratched off.
		mean...	128.9	
	1	9.29	160.	Broke partly through brick.
	2	9.12	233.	Broke through mortar.
	3	9.62	147.	Separated at the joint.
(B)	4	9.62	96.	Broke through mortar.
	5	9.71	202.	Broke through brick.
	6	9.47	145.	Separated at the joint.
	7	"	107.	Separated cleanly at the joint.
	8	8.99	47.3	" "
	9	11.2	128.	Broke through brick.
	10	9.20	138.	Broke through mortar.
	11	9.5	45.0	Separated at the joint.
	12	9.05	135.	" "
		mean ..	131.9	

General mean = 130.4 lbs. per sq. in.

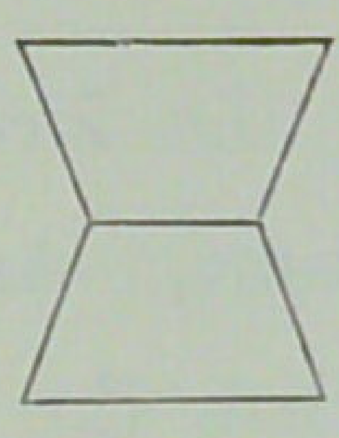
The date of construction of the column was May 22nd, 1898; and that of the testing experiments Feb. 6th, 1899.

In the testing experiments, the load or stretching weight was applied always very gradually.

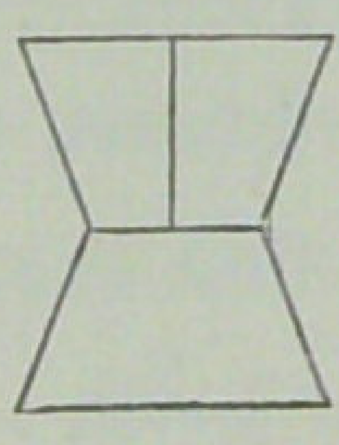
According to table II, the tensile strength of the brick columns used in the fracturing experiments varied between 33.7 and 130.4 lbs. per square inch, which wide difference perhaps depends to a certain extent on the quality of the bricks, all the columns having been constructed with the same mortar. The tensile strength of the five columns Nos. 1, 3, 6, 7 and 9, which were composed of $\frac{1}{8}$-bricks, varied between 43.8 and 69.3 lbs. per sq. in., the average value being 56.5 lbs. per sq. in. Again, the strength of the twelve columns Nos. 10, 11, 12, 13, 14, 15, 16, 17, 18, 19, 20 and 21, which were composed of $\frac{1}{2}$-bricks, varied between 33.7 and 64.2 lbs. per sq. in., the average value being 47.1 lbs. per sq. in. These $\frac{1}{8}$ and $\frac{1}{2}$ bricks were of the *ordinary first* or *second class* quality.

The tensile strength of the two columns Nos. 22 and 23, composed of ordinary second class bricks of normal dimensions, were respectively 100.5 and 77.1 lbs. per sq. in ; while that of column No. 24, composed of normal bricks of an extra-superior quality, was 130.4 lbs. per sq. in. These results seem to show that the tensile strength of the brick and mortar joint is raised with the good quality of bricks. I may here add that the tensile strength of the *bricks*, which composed columns Nos. 23 and 24, were respectively 142 and 303 lbs. per sq. in.

(a)



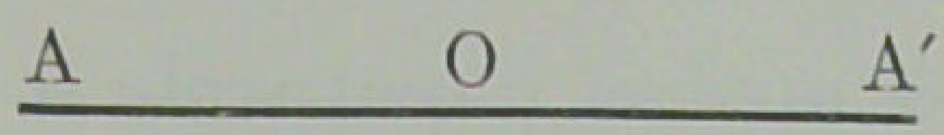
(b)



The tensile strength of the brick work seems, so far as the above figures show, to depend little on the size of the bricks themselves, there being for instance no great difference between the strength of the columns composed of $\frac{1}{8}$-bricks and those composed of $\frac{1}{2}$-bricks. Again, the brick work of the two columns Nos. 22 and 24 was tested in the two different forms, (a) and (b) ; (a) having only one horizontal joint and (b) one horizontal and one vertical joint. The two series of determinations however, gave practically identical results. (See table II, h, i.)

13. *Dependence of the tensile strength on the workmanship.*—The tensile strength of the brick and mortar joint for different portions of one and the same column varies sometimes within wide limits. Thus, even excluding the cases in which the test pieces simply gave way while being fixed to the testing machine, the maximum value of the strength amounted sometimes to five or six times its minimum value. (See table II.) This fact is probably to be attributed to the non-uniformity of the workmanship. Fig. 7 illustrates the broken faces of three test pieces taken from the column No. 24, the fracture having been caused in each case by the separation of mortar and brick. The relative strengths of the joints may be judged by the extent of the area of the mortar on which thin layers of the bricks were left adhering, and which in the first case covered nearly the whole face of the separated brick, in the second about one half, and in the third was limited to a few detached points. The tensile strengths for these three cases were, in correspondence with the above facts, respectively 209, 119 and 61 lbs. per sq. in. The importance of a strict superintendence in the construction of the brick-work can not be overestimated.

14. *Impulse.*—An *impulse* is an action in which the force is applied to a body during an infinitely short interval of time. As the breaking strength of a given body would be different when the force is impulsively applied from what it would be if the force were gradually applied, it is necessary to determine whether the earthquake motion is, in the calculation of the seismic stability of brick columns, to be regarded as an impulse or not. For this purpose let us first examine the relation between an impulse and the strain produced by it. The



well-known proposition, that a force when impulsively applied to an elastic body produces a strain double that caused by the same force when gradually applied, may be explained as follows. Let A diagramatically represent the position of equilibrium of a given body in its natural state, and O that assumed by it when acted on gradually by a force R. If now the force R act on the body impulsively, the result would be the same as if the body, whose position of equilibrium is at O, had been displaced to A. Consequently the dody would be carried to the position A′ at an equal distance on the opposite side of the equilibrium position *o* and thus vibrations would be set up. In other words, the strain produced by the impulsive application of the force R is double that produced by its gradual application.

In discussing the effect of a force on a work of construction, difficulty may occur in deciding whether the application of the force is to be regarded as *impulsive* or as *gradual*. The criterion is easily afforded by the above theorem, as follows.

1. If the application of a force to an elastic body be so slow that the latter assumes the position of equilibrium without being thrown into vibrations, the force may be regarded as acting *gradually*.

2. If the application of a force to an elastic body be so rapid that it is finished in a time interval infinitely small in comparison to the period of the corresponding natural vibration of the body, the force may be regarded as acting *impulsively*.

Fracturing by impulse.—What was stated above relates to the behaviour of an elastic body within the limit of its perfect elasticity. With regard to stones and bricks, however, the stress and strain relation is such that fracture immediately takes place when the limit of elasticity is exceeded; and consequently we may assume that the strength of a brick column against a force applied impulsively is half that against the same force applied gradually.

15. *The earthquake motion.*—In the application of equation (3) to the calculation of the seismic stability of a brick column,* the quantity a, or the destructive power of the earthquake motion, is, according to the definitions given in § 14, to be regarded as acting *gradually* and not *impulsively*; that is to say, the quantity F is to be understood as the ordinary tensile strength of the column, and not equal to its half. Thus in the fracturing experiments the period of the shaking table varied, as given in table IV, between 0,23 and 0,89 second and therefore the acceleration of the motion changed from zero to the maximum value in a time interval of 0,06 to 0,22 second. Such a motion, though certainly very violent, can not be considered as producing an impulsive effect to the brick column, the period of whose longitudinal vibration is naturally very short and which is broken by the tension across the plane of fracture produced in consequence of its flexure. The values of the tensile strength obtained by adding the load gradually, given in table II, are therefore to be employed in the calculation, by equation (3), of the seismic stability of the columns.

The above remark respecting the value of F applies also to the case of an actual destructive earthquake, the period of whose principal vibration is probably, as stated in § 10, one or two seconds and therefore longer than those of the shaking table in the present experiments.

16. *Experiments on the tensile strength of bricks.*—In illustration of what was said in § 15, the following experiments have been made.

The tensile strength of a number of bricks of one and the same

* The height of the column must be several times, but not infinitely, greater than its thickness and the range of the earthquake motion.

extra-superior quality was first determined, the result being as shown in the following table:

No. of brick	Sectional area. (sq. in.)	Tensile strength. (lbs. per sq. in.)	Remarks.
1	6.11	316	The load was applied in all the cases very gradually, and the bricks broke at, or near, the smallest section.
2	5.57	337	
3	5.57	365	
4	6.11	262	
5	5.66	241	
6	5.57	350	
7	5.57	237	
8	5.85	356	
9	6.11	257	

Mean..................................303.

I may here remark that, in the present experiments, bricks of extra-superior quality have been chosen, as such bricks would be comparatively uniform in strength. Having ascertained from the experiments on the bricks, Nos. 1—7, the average value of the tensile strength to be about 300 lbs. per sq. in., the brick No. 8 was fixed to the testing machine and a tension of 200 lbs. per sq. in. was suddenly applied on it by releasing the longer arm of the machine, which had previously been arrested. The brick was, however, not broken. The experiment was then twice repeated with tensions of 227 and 255 lbs. per sq. in.; still the brick did not give way. When determined by the usual method, the tensile strength of the brick was found to be 356 lbs. per sq. in.

The brick No. 9 was similarly tested by suddenly applying a tension, firstly, of 189 lbs. per sq. in., and secondly, of 217 lbs. per sq. in., but it did not break. The tensile strength was then determined by the usual method and found to be 257 lbs. per sq. in.

In each of these experiments, the application of the load was certainly finished in a very short interval of time, probably one-tenth of a second or so, still the result shows that the action was not impulsive, the brick remaining unaffected even when acted upon thus suddenly by a tension much greater than half its tensile strength.

17. *Remarks on the tensile strength of the brick columns.*—To determine the approximate values of the tensile strength F, to be used in the calculation of the seismic stability of the brick columns, the following two cases should be considered separately.

(a). When the column was fractured at a joint partly through mortar and brick, and partly by the separation of these two substances, the value of F was obtained by averaging all the determinations of the tensile strength of the brick-work test pieces, which were broken at the joint through the mortar, through the brick, or by the separation of mortar and brick.

(b). When the column was broken at a joint entirely by the separation of brick and mortar, the value of F has been obtained by averaging only those determinations of the tensile strength, in which the brick-work test pieces were broken by the separation of mortar and brick. The ten columns, Nos. 3, 10, 11, 12, 13′, 15,″ 21,* 22′, 23 and 23′, all belong to this class. For the two columns, Nos. 19* and 19″,* which were abnormally weak, the smallest among the different values of their tensile strength have been adopted.

It is unnecessary to remark that the brick-work test pieces gave generally higher values of the tensile strength when the joint gave way by the fracture through mortar or brick than when by the separation of these two substances.

18. *The fracturing experiments.*—The total number of the experiments is forty-three, the condition of the fracture of the columns being described in table III. The specimen diagrams of the motion of the shaking table are given in figs. 8—15.

TABLE III.—Fracturing Experiments.

(a) Columns without Concrete Base.

No.	Total height. (mm.)	Height above the iron fixing frame. mm.	Where and how fractured.
1	720	662	Broke at the 3rd joint, or 64 mm. from base, partly through bricks.
7	,,	,,	Broke at the base.
8	,,	,,	,, ,,
9′	720	604	Broke at the base, through bricks.
9″	604	532	Broke, by a clean separation, at the 3rd joint, or 43 mm. from base.
10	902	760	Broke, by a clean separation, at the base.
11	,,	720	Broke through joint at the base.
11′	720	650	Not broken.
13	1800	1590	Broke at the 1st joint, or 36 mm. from base, partly through bricks.
13′	1550	1330	Broke at the 1st joint, or 36 mm. from base.
13″	1300	1080	Broke through joint at the base.
13‴	1080	900	,, ,,
13⁗	900	720	Broke at the 3rd joint, or 110 mm. from base.
14	1800	1590	Broke at the 4th joint, or 144 mm. from base.
14′	1440	1230	Broke at the 3rd joint, or 110 mm. from base.
14″	1120	940	Broke at the 1st joint, or 36 mm., within the iron fixing frames.
14‴	970	830	Broke at the 4th joint, or 144 mm. from base.
15′	1370	1090	Broke at the 1st joint, or 36 mm. from base, through bricks.
15″	1040	760	Broke through joint at the base.
16	1744	1560	Broke at the 1st joint, or 35 mm. from base.
16′	1560	1360	Broke at the 5th joint, or 170 mm. from base.
17	1744	1530	Broke at the base.
18	,,	1570	Broke at the base, partly through bricks.
18′	1570	1400	Broke at the 1st joint, or 35 mm. from base, partly through bricks.
18″	1360	1190	Broke at the 2nd joint, or 70 mm. from base, partly through bricks.
19*	1810	1650	Broke at the base, where small cracks existed beforehand.
19′*	1630	1450	Broke through joint at the base.
19″*	1380	1140	Broke at the 3rd joint, or 109 mm., within the iron fixing frame.
20*	1810	1520	Broke at the 6th joint, or 217 mm. from base, partly through bricks.

No.	Total height. (mm.)	Height above the iron fixing frame. (mm.)	Where and how fractured.
20′*	1270	980	Broke at the 1st joint, or 36 mm. from base, through bricks.
21′*	1450	1160	Broke at the 1st joint, or 36 mm. from base, partly through bricks.
22	1580	1300	Broke at the 1st joint, or 69 mm. from base.
22′	1240	960	Broke through joint at the base.
23	1580	1380	" "
23′	1380	1200	Broke, by a clean separation, at the 2nd joint, or 138 mm. from base.
23″	1100	890	Not broken.
24′	1620	1320	Broke through joint at the base.

(*b*) Columns with Concrete Base.

No.	Height. (mm.)	Where and how fractured.
3	720	Broke through joint at the base.
6	730	Broke at the 6th joint, or 80 mm. from base, partly through bricks.
9	720	Broke at the base, partly through bricks.
12	915	Broke, by a clean separation, at the 4th joint, or 146 mm. from base.
21*	1810	Broke at the 10th joint, or 340 mm. from base, partly through bricks.
24	1587	Broke at the 1st joint, or 68 mm. from base, within the concrete.

The two columns Nos. 11′ and 23″, whose heights were proportionally small, could not be broken even with the strongest motion of the shaking table. All the others were, however, broken near the base ; thirty-nine columns giving way at, or very near to, the base while the remaining two Nos. 20* and 21* were broken at one-seventh and one-fifth of their heights from the base respectively. It is thus evident that the columns are seismically weakest at their bases.

As the brick column is fractured always at a joint, its seismic stability may be increased by using a good mortar, until the strength of the joint becomes equal to that of the bricks themselves. It may here be added that the columns composed of $\frac{1}{2}$- and $\frac{1}{3}$- bricks of inferior quality were sometimes broken partly by the fracture through brick and mortar and partly by the separation of these two substances, but the other columns, composed of bricks of normal dimensions, were, with the exception No. 24, all broken by a separation of brick and mortar at the joint.

19. *Results of the fracturing experiments.*—The discussion of the results of the fracturing experiments consists in comparing the actual intensity of motion of the shaking table with the calculated values of the seismic stability of the columns.

The intensity, or the maximum acceleration A, of the shaking table may be calculated by the following formula

$$A = \frac{4\pi^2 a}{T^2}, \qquad (6)$$

in which a and T are respectively the amplitude (semi-range) and the period of the shaking at the moment when the column was fractured. These two quantities are to be directly measured from the diagrams of the motion of the shaking table obtained by the arrangement described in § 7.

The seismic stability, or the least value of the acceleration α, capable of fracturing the column, may be calculated by means of equation (4) or (5) from its thickness (or dimension in the direction of the earthquake motion), area and form of section, height of centre of gravity of the portion fractured, and the tensile strength of the joint.

It may here be noted that for the calculation the section of fracture of a given column must first be determined. This will in practical cases be easily found. For example a column of uniform section is, as already pointed out, weakest at the base.

The result of the experiments is summarized in table IV.

IV. Summary of Results of the Fracturing Experiments.

$2x$ = thickness of column; $2y_0$ = its height above the section of fracture; F = its tensile strength; and α = acceleration necessary for fracturing the column. (Eq. 4 and 5).

$2a$ = range of motion of the shaking table; T = period; and A = maximum acc. or $\frac{4\pi^2 a}{T^2}$.

Date of Expt.	No. of column.	Calculation of α; eq. 4 or eq. 5.					Motion of the shaking table			Ratio.	Date of determination of F.
		$2x$ mm.	$2y_0$ mm.	Sectional area, sq. mm.	F lbs per sq. in.	α mm./sec.^{2}	$2a$ mm.	T sec.	A mm./sec.^{2}	A/α	
Oct. 12th, 1898.	1	95	560	96×120	55.3	21200	111.	0.37	16000	0.8	Dec. 14th, 1898.
	7	72	660	72×97	43.8	10100	97.	.57	5900	.6	,,
	8	72	,,	72×97	,,	,,	95.	.58	5580	.6	——*
	17	230	1530	230^2	41.3	5800	99.	.50	7850	1.3	Dec. 16th, 1898.
	19*	233	1620	233^2-131^2	cracks beforehand.	——	90.	.81	2710	...	Dec. 14th, 1898.
	19'*	,,	1440	,,	64.2	13300	107.	.40	13200	1.0	,,
	19''*	,,	1230	,,	39.2	11100	97.	.64	4680	.4	,,
	3	96	720	120^2	42.9	11000	109.	.52	8000	.7	,,
	6	97	650	97^2	69.3	22200	69.	.37	9840	.4	Dec. 16th, 1898.
	9	72	720	72×97	58.8	11400	53.5	.70	2170	.2	,,
	9'	,,	604	,,	,,	16200	70.5	.43	7520	.5	,,
	9''	,,	490	,,	,,	24400	76.5	.29	17900	.7	,,
Oct. 26th, 1898.	11	111	720	111^2	47.4	12400	43.4	.33	7900	.6	Dec. 14th, 1898.
	11'	,,	650	,,	53.3	17200	49.	.23	20000 not broken	...	,,
	23	230	1370	230^2	53.6	9850	44.5	.32	8650	1.0	Feb. 4th and 26th, 1899.
	23'	,,	1100	,,	,,	13900	46.	,,	8880	.6	,,
	23''	,,	894	,,	77.2	30200	56.	.23	21300 not broken	...	,,
	14	232	1440	110×232	33.7	5200	40.	.48	3430	.7	Dec. 14th, 1893.
	14'	,,	1120	,,	,,	8700	45.4	.32	8780	1.0	,,
	14''	110	970	,,	,,	5500	39.7	.49	3270	.6	,,
	14'''	,,	680	,,	,,	11100	107.	.48	8950	.8	,,
	16	230	1540	230^2	51.2	6900	108.	.51	8200	1.2	,,
	16'	,,	1190	,,	,,	11600	112.	.35	18000	1.6	,,
Jan. 27th, 1899	18	230	1570	230^2	43.8	5600	103.	.62	5960	1.1	Feb. 21st, 1899.
	18'	,,	1360	,,	,,	7400	105.	.50	8300	,,	,,
	18''	,,	1120	,,	,,	11000	112.	.39	14600	1.3	,,
	10	111	760	111^2	45.1	11800	103.	.59	5850	.5	March 27th, 1899.

* The tensile strength of col. No. 8 was unfortunately not determined, and consequently the value F = 43.8 for col. No. 7 has been adopted in the calculation of α, these two columns being exactly similar to each other.

Date of Expt.	No. of column.	Calculation of a ; eq. 4 or eq. 5.					Motion of the shaking table.			Ratio.	Date of determination of F.
		$2x$ mm.	$2y_0$ mm.	Sectional area. sq. mm.	F lbs. per sq. in.	a mm./sec^2	$2a$ mm.	T sec.	A mm./sec^2	A/a	
Feb. 1st, 1899.	12	110	768	$\overline{111}^{2}$	33.4	8700	47.5	.58	2790	.3	March 27th, 1899.
	13	,,	1550	110×230	34.2	2100	46.	.76	1570	.8	Feb. 26th, 1899.
	13′	,,	1290	,,	19.3	1750	46.5	.72	1790	1.0	,,
	13″	,,	1080	,,	34.2	4400	46.5	,,	,,	.4	,,
	13‴	,,	900	,,	,,	6300	49.7	.42	5570	.9	,,
	13⁗	,,	610	,,	,,	13800	57.	.29	13400	1.0	,,
Feb. 14th, 1899.	21*	233	1470	$\overline{230}^{2}-\overline{123}^{2}$	31.3	5950	111.	.72	4250	.7	March 8th and April 5th, 1899.
	24	220	1620	$\overline{230}^{2}$	130.4	15100	126.	.46	11800	.8	March 6th and 30th, 1899.
	24′	,,	1350	,,	,,	21500	,,	.39	16800	.8	,,
Feb. 15th, 1899.	22	230	1240	$\overline{230}^{2}$	100.5	15300	123.	.44	12600	.8	March 28th and 30th, 1899.
	22′	,,	962	,,	64.0	13700	128.	.38	17600	1.3	,,
	20*	233	1300	$\overline{233}^{2}-\overline{133}^{2}$	57.9	14700	119.	.54	8000	.5	March 10th, 1899.
	20′*	,,	944	,,	,,	27900	123.	.45	11900	.4	,,
	21′*	230	1120	$\overline{230}^{2}-\overline{123}^{2}$	46.1	15100	115.	.60	6300	.4	March 8th and April 5th, 1899.
	15′	110	1040	110×232	44.7	4650	109.	.89	2720	.6	March 27th, 1899.
	15″	,,	756	,,	30.2	10200	118.	.51	8950	.9	,,

Mean..........................0.8

As will be seen from table IV, the double amplitude of the shaking table varied between 39,7 and 128 mm. and fairly represents the motion likely to occur in strong and destructive earthquakes, the maximum movement in the Tokyo earthquake of June 20th 1894 being, at Hongo, 73 mm. Further, the maximum acceleration varied between 1570 and 21300 mm. per sec. per sec., and therefore it may be regarded as giving the intensity of motion to be expected in any great earthquake. Thus, the maximum acceleration in the low and soft ground portions of Tokyo was at the time of the earthquake just referred to about 1000 mm. per sec. per sec.; while that in the most epifocal tract of the great Mino-Owari earthquake of Oct. 28th 1891 was probably about 10.000 mm. per sec. per sec.

The period of the shaking table varied from 0.23 to 0.89 second.

As columns Nos. 11′ and 23″ could not be broken even with the most violent motion of the shaking table, we may reasonably conclude that simple structures, like columns, walls or bridge piers, can when properly constructed resist any destructive earthquake motion.

Considering that experiments of this kind are very difficult to carry on and that the results would sometimes be little more exact than the determination of the order of the quantities concerned, the agreement of the seismic stability (α) of the column with the intensity of motion of the shaking table will be observed, from table IV, to be in general satisfactory. Thus the average ratio of A : α is 0,8, being sufficiently near to unity. The most difficult part in the calculation of α is the determination of F, the different joints of a brick column evidently having not necessarily one and the same tensile strength. As the columns tend to break at the weakest joint near the base, the values of F employed in the calculations given in table IV are probably somewhat greater than the actual strengths of the fractured joints. Again, it is well known that the tensile strength of the mortar increases with time, the variation continuing sometimes for two or three years. In the present case, the fracturing experiments were carried on 5 to 10 months after the construction of the columns; while the determination of their tensile strength was made from 20 days to 2 months after the fracturing experiments, thus giving values of the strengths possibly a little greater than what we want really to have. The ratio $^{A}/_{a}$ would be brought a little nearer to unity, if properly corrected for these two circumstances each of which tends to increase the value of a.

I conclude therefore that equations (3), (4) and (5) give, without sensible errors, the seismic stability of the columns and walls.

I give next a few remarks respecting equations (3), (4) and (5).

According to equations (4) and (5), the seismic stability (a) of a given uniform column is directly proportional to its thickness ($2x_0$) and inversely proportional to the square of its height ($2f$). It would thus seem that a very tall column is incapable of resisting strong earthquake motion.

Again, according to equation (3), the seismic stability of a uniform column is proportional to its tensile strength F, which latter has however been shown to be practically no other than the strength of the mortar joint. Consequently the stability of such a column can be raised n times by increasing its thickness n times, or by keeping the latter quantity unchanged and increasing the tensile strength of the mortar n times. This conclusion, which applies also to the walls, bridge piers and similar structures, indicates that sometimes both space and expense may be saved by reducing the thickness of a brick structure and employing a better quality of mortar in its construction.

20.—*Relation between the height of the column and the fracturing acceleration.* In table V, are collected the results of the experiments with columns Nos. 10–15″, which were all of one and the same thickness, viz. 110 mm. To reduce all the columns to the hypothetical case of a common tensile strength, say of 100 lbs. per sq. in., the fracturing acceleration (A) was in each case divided by the corresponding F and then multiplied by 100. As graphically shown in fig. 16, the modified fracturing acceleration $(A\times\frac{100}{F})$, will be seen to be approximately proportional to the inverse square of $2f$.

Again, fig. 17 illustrates the relation between the heights and the fracturing accelerations of columns Nos. 13–13‴′. The value of A will be seen to diminish very quickly with the increase of $2f$.

TABLE V.— Relation between the Heights of the Columns and the Fracturing Accelerations.

$2x$ = thickness of brick column. A = max. acc. of the shaking table.
$2y_0$ = height of column above the section of fracture.
F = tensile strength of column.

No. of Column	$2x$ mm.	Area of sec. sq. mm.	$2y_0$ mm.	A mm./sec.^{2}	F lbs. per sq. in.	$A \times \frac{100}{F}$ mm./sec.^{2}
11	111	$\overline{111}^2$	720	7900	47.4	16700
14″	110	110×230	970	3270	33.7	9750
14‴	,,	,,	680	8950	,,	26500
10	111	$\overline{111}^2$	760	5850	45.1	13000
12	110	$\overline{110}^2$	768	2790	33.4	8350
13	,,	110×232	1550	1570	34.2	4580
13′	,,	,,	1290	1790	19.3	9280
13″	,,	,,	1080	,,	34.2	5230
13‴	,,	,,	900	5570	,,	16300
13⁗	,,	,,	610	13400	,,	39300
15	,,	,,	1040	2720	44.7	6100
15″	,,	,,	756	8950	30.2	29600

21. *Hollow columns.*—Equation (5), which gives the seismic stability of a hollow square column of uniform section, may be written as follows :—

$$\alpha = \frac{2gF}{3w.\,\overline{2f}^2}\left(x_0 + \frac{x_1^2}{x_0}\right).$$

The α thus defined is greater for given values of x_0 and $2f$, than that defined by equation (4), which represents the stability of a corresponding solid square column. This leads to the conclusion that for given values of height and external dimensions, the hollow square column has a greater seismic stability than the solid one, the sides or walls in the former case being assumed to be sufficiently thick and rigid to warrant the employment of equation (5).

22. *Column of the uniform strength.*—For a column of uniform section equation (3) may be written :—

$$\alpha = \frac{IgF}{2x_0 ws.\,f^2}, \qquad (7)$$

s being the sectional area. Since I, x_0 and s remain constant for different sections, α is inversely proportional to the square of $2f$. Such a column is therefore weakest at the base. (See table III.) This conclusion applies equally well to brick or iron-pipe piers of bridges, as has been exemplified by the damage caused by the great earthquake of Oct. 28th 1891 to the Kiso, Nagara, Ibi and other railway bridges, which were all broken at their feet. Let us next calculate the form, which would give to the column a uniform seismic stability.

A column of the uniform strength must evidently terminate in a point or line top, as shown in fig. 18. Let the origin of coordinates be taken at the top, the axis of x horizontally, and the axis of y vertically downwards.

(a). Let the section be square. Let $2x_0$ be the length of the side of a given section and y_0 its distance from the top, and let V and f be the volume and the height of the centre of gravity of the portion of the column above that section. We have then the following equation :—

$$f = \frac{\int_0^{y_0} 4x^2 (y_0 - y)dy}{V}.$$

Whence equation (3) may be written :

$$a = \frac{4gF.x_0^3}{3wfV} = \frac{gFx_0^3}{3w\int_0^{y_0} x^2(y_0-y)dy}.$$

To make a constant in the above equation, we find, by Calculus of Variation, the following relation between x_0 and y_0 :

$$y_0^2 = \frac{10gF}{aw}x_0 \qquad (8)$$

This represents a parabola whose apex is at the origin of coordinates, and whose concavity is turned outwards as shown in the figure which has been drawn from the equation

$$y_0^2 = 64500x_0 \qquad \text{(units in inches.)}$$

obtained from the following data :—

a=1000 mm. per sec. per sec. (*i.e.* the intensity of motion in the low portions of Tokyo at the time of the earthquake of June 20th 1894) ;

F=40 lbs. per sq. in. (*i.e.* the average value of the tensile strength of the brick-work damaged in Tokyo by the same earthquake) ;

w=0.0608 lb.

(*b*). Let the section be rectangular, its breadth, or dimension normal to the earthquake motion, being $2b$. Equation (3) may be written :

$$a=\frac{4gFbx_0^2}{3wfV}=\frac{gFx_0^2}{3w\int_0^{y_0}x(y_0-y)dy},$$

in which $2x_0$ is the thickness or dimension in direction of the earthquake motion, y_0, V and f having the same signification as in (a).

The constancy of a requires again the following parabolic relation between x_0 and y_0 :

$$y_0^2=\frac{4gF}{aw}x_0 \qquad (9)$$

(c). Let the section be circular. We have in this case

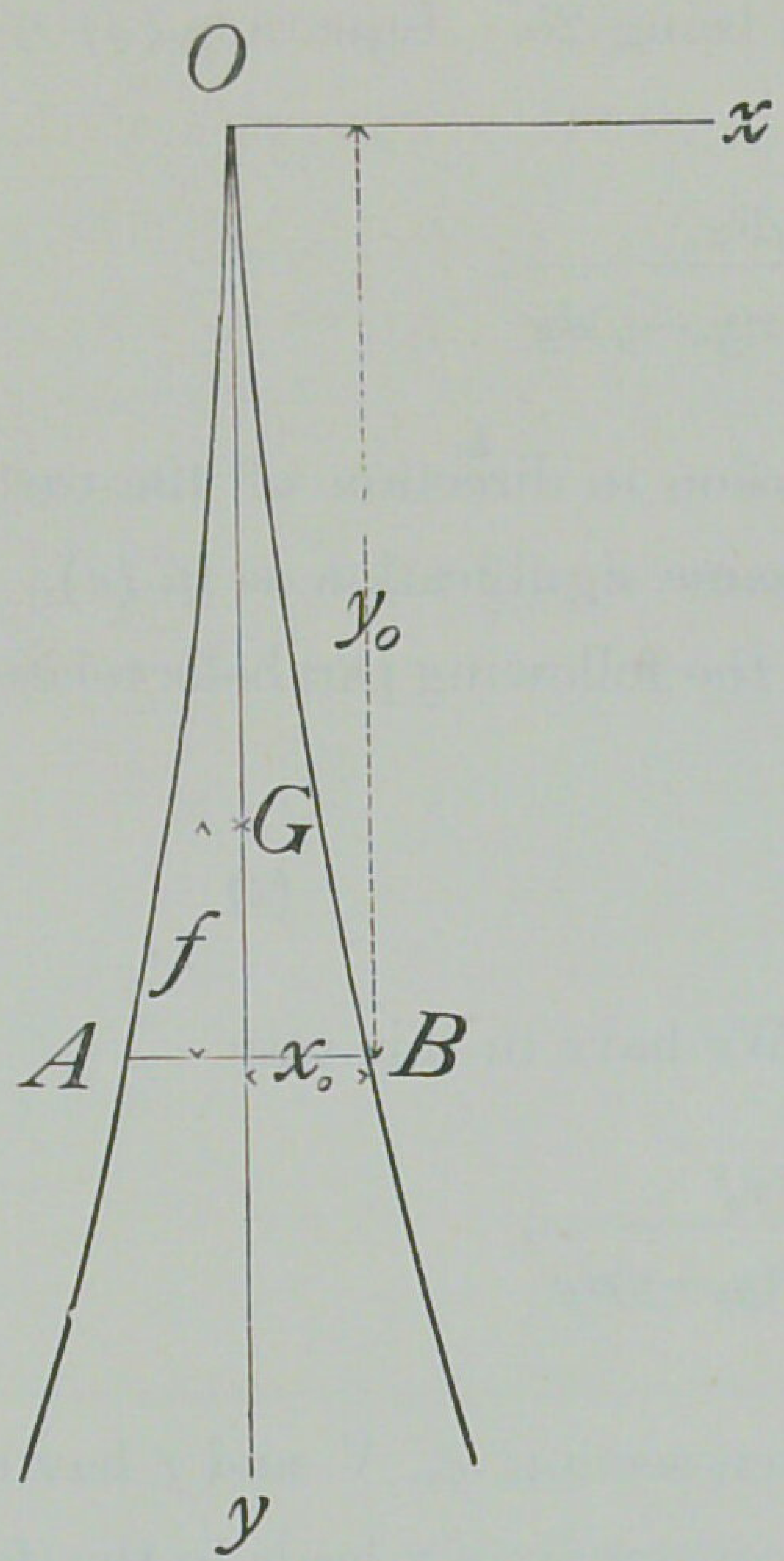
$$a=\frac{\pi gFx_0^3}{4wfV}=\frac{gFx_0^3}{4w\int_0^{y_0}x^2(y_0-y)dy},$$

in which $2x_0$ is the diameter of any given section, y_0, V and f having the same signification as before. The constancy of a leads to the following equation :

$$y_0^2=7\frac{1}{2}\cdot\frac{gF}{aw}x_0 \qquad (10)$$

Comparing the three equations, (8), (9) and (10), we see that for given values of height (y_0) and basal dimension ($2x_0$), the column of rectangular section has the highest seismic stability.

23. *Applications to brick bridge-piers.*—Let the breadth, or dimension of the section in direction of the earthquake motion, be constant and equal to $2b$. In this case we have to determine the top and base dimensions of the pier according to equation (9) in such a manner that the girders, rails and sleepers may be substituted for its truncated portion without much affecting the condition of uniformity of strength.



Let AOB represent the vertical section of a rectangular brick pier of uniform strength. If W be the weight of the column above any given section AB, and if f be the height of its centre of gravity, we obtain from equation (9),

$$W = 4bw\int_0^{y_0} x\,dy = \frac{abw^2 y_0^3}{3gF}, \qquad (11)$$

and
$$f = \frac{\int_0^{y_0} 4bx(y_0 - y)dy}{\text{vol. OAB}} = \frac{y_0}{4}. \qquad (12)$$

Suppose now the portion AOB to be removed, and the girders, rails and sleepers to be put on the top of the pier thus formed. The bending moment of the load with respect to the section AB, when the maximum acceleration of the earthquake motion is a, is

$$\frac{W'}{g} \times h\,a,$$

W′ being the weight of the load and h the height of its centre of gravity above AB. Equating this to the bending moment of the portion AOB of the original column with respect to AB, we obtain, from equations (11) and (12),

$$\frac{W'ha}{g} = \frac{a^2 b\, w^2 y_0^4}{3g^2 . 4F}; \text{ or } y_0^4 = \frac{12g\,F\,W'\,h}{a\,b\,w^2}; \qquad (13)$$

and
$$x_0^2 = \frac{y_0^4\, a^2 w^2}{16g^2 F^2} = \frac{3h\,W'\,a}{4b\,g\,F} \qquad (14)$$

Equations (13) and (14) give the width ($2x_0$) and the distance (y_0) from the apex of the parabola of the top (AB) of the pier, its form and basal dimension being then given by equation (9). The pier thus determined is somewhat stronger at its top than at its base, but it is to be considered as having nearly a uniform seismic stability when compared to ordinary vertical piers.

As an illustration let us take a case analogous to the Kiso Railway-bridge, supposing

$F = 65$ lbs. per sq. in.,

$w = 0.0608$ lb.,

$W' = 200$ tons,

$h = 10$ ft.,

$a = 3000$ mm. per sec. per sec.,

$f = 30$ ft.

If, further, the height of the pier be 30 feet, equations (13) and (14) give respectively

$y_0 = 674$ in.,

and $2x_0 = 5$ ft. 5 in.

Again, equation (9) becomes (writing x, y for x_0, y_0)

$y^2 = 13970\ x$, (x and y expressed in inches)

from which the thickness at the base is found to be 12 ft. 9 in. The form of the pier thus calculated is shown in fig. 19, the curvature of the two parabolic faces being so slight, that we may substitute inclined planes for the latter.

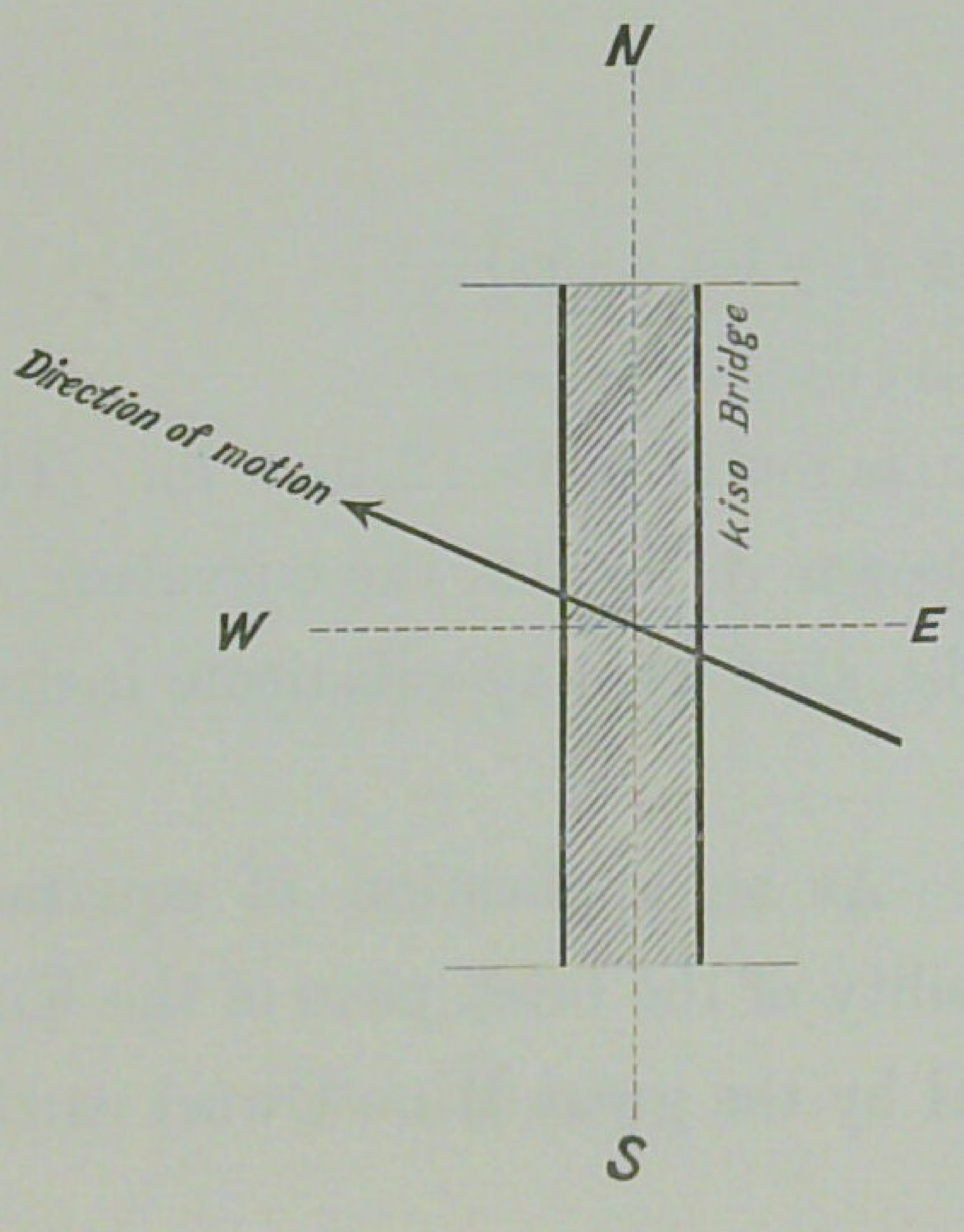
24. *The Kiso Railway-Bridge.*—As an application of equation (4), I shall calculate the seismic stability of the brick piers of the Kiso Railway-bridge, which was destroyed by the great Mino-Owari earthquake of Oct. 28th 1891.

(a). *Construction of the bridge.* The bridge consisted of nine 200 ft. span girders of wrought iron each weighing 156,7 tons. As the weight of the rails and sleepers was 8,9 tons per 200 ft., the load supported by a single pier amounted to 165,6 tons. The height of the girders was 19 ft. 4,8 in., and the height of their centre of gravity about $9\frac{1}{2}$ ft.

The foundation was formed by a pair of circular wells 12 ft. in diameter, joined together at the top by an arch on which the pier or abutment was constructed. (See figs. 20 and 21.) The mortar was composed of 1 part of cement and 2 or 3 parts of sand.

The height, thickness and width of the pier, which was provided at each end with a triangular buttress, were respectively 30 ft. 9 in., 10 ft. and 21 ft.

(*b*). *The intensity of the earthquake motion.* With reference to the intensity of the earthquake motion at the Kiso bridge, I give next the result of my observations at the towns of Ichinomiya (in the province of Owari) and Kasamatsu (in the province of Mino), both in the vicinity of the bridge.



At Ichinomiya, about 4 km. from the bridge, the maximum horizontal acceleration was between 2500 and 3500 mm. per sec. per sec., the direction of motion being WNW and ESE.

At Kasamatsu, situated immediately to the north of the bridge, the maximum horizontal acceleration was about 4000 mm. per sec. per sec., the direction of motion being WNW and ESE.

The direction of motion in the vicinity of the Kiso bridge thus seems to have been WNW and ESE, and the greatest displacement of the ground probably took place towards WNW, as the majority of stone lanterns and similar bodies were overturned in that direction. Assuming the maximum acceleration of the earthquake motion at the bridge (whose length was parallel N and S) to be the same as that at Kasamatsu, viz. 4000 mm. per sec. per sec., we obtain the following results.

The intensity of motion in the N and S direction, i.e. normal to the piers, $=4000 \times \sin 22°\frac{1}{2}=1500$ mm. per sec. per sec.

The intensity of motion in the E and W direction, i.e. normal to the length of the bridge $=4000 \times \cos 22°\frac{1}{2}=3700$ mm. per sec. per sec.

(*c*). *The damage to the bridge.* Each of the wells suffered more or less displacement, while the abutments and the piers were without exception fractured. As will be seen from figs. 22 and 23, each of the piers was broken at, or very near to the arched junction with the well and displaced 3 to 8 in. towards south and west; the abutments were cleanly broken horizontally at the junction with the arch at the back.

Piers (or abutments) like those of the Kiso bridge are apparently strong enough, but their seismic stability may turn out when calculated to be very low, as the fracturing force which is equivalent to the product of the combined mass of the pier, girders, etc., and the maximum acceleration of the earthquake motion, is also very great.

(*d*). *Wells.* The employment of a pair of circular wells joined together by an arch is seismically very bad, its defects being two fold. Firstly, the two wells would be caused to assume independent movements, necessitating the formation of cracks at the junction with the pier. Secondly, the arch reduces very much the sectional area of the pier at its weakest place, the base. Thus in the case of the bridge under discussion, the width of the basal section of the pier was 21 ft. while the diameter of the arch was 7 ft. 4 in., the seismic stability of the pier for earthquake motion normal to its plane, being reduced by one-third of its total value. An elliptical well on the other hand is free from these defects and is seismically a very strong construction.

(e). *The tensile strength of the brick-work of the Kiso Bridge.* After the great earthquake of Oct. 28th 1891, the authorities of the Imperial Railway Department made a determination of the tensile strength of the brick-work of the bridge. The result is shown in table VI, the average value of the strength being 62.4 lbs. per sq. in.

TABLE VI.—Tensile Strength of the Brick-Work of the Piers and Abutments of the Kiso Railway Bridge, Damaged by the Great Earthquake of Oct. 28th 1891.*

Place whence brick specimen was taken.	Tensile strength per. sq. in. (lbs.)	Remarks.
No 6 pier	103.	
No. 5 pier	43.	
"	59.	Broke partly through bricks.
No. 8 peir	57.6	
No. 7 pier. South part of up stream side of arch.	141.	Broke at joint.
No. 5 pier. North part of up stream side well.	88.	Broke through bricks
No. 5 pier. North part of up stream side of middle layer.	65.	Broke at joint.
No. 5 pier. North part of down stream side of arch.	168.	Broke partly through bricks and partly along mortar joint.
No. 3 pier. South side surface of middle layer.	0	Simply dropped apart.
No. 2 pier. South side surface of arch.	0	"
No. 1 pier. South side surface of arch.	——	Held on to 63 lbs. per sq. in., did not break.
South abutment. North side surface beneath bedstone.	36.	Clean separation at joint.
No. 2 pier. North side surface of middle layer.	26.	Broke through bricks.
No. 2 pier. North side surface of arch.	81.	Clean separation at joint.
No. 2 pier. South side surface of middle layer.	41.	"
South abutment. South interior part beneath bedstone.	49.	Separation at joint.
No. 1 pier. North side surface of middle layer.	41.	Clean separation at joint.
No. 1 pier North side surface of arch.	93.	Broke partly through brick and partly through mortar.
No. 3 pire. North side furface beneath bedstone.	63.	Clean separation at joint.
No. 3 pier. South side surface beneath bedstone.	60.	"
No. 3 pier. North side surface of arch.	54.	Broke entirely through bricks.
No. 2 pier North side surface at base of pier.	92.	Broke at joint.
No. 2 pier. South side surface at base of pier.	0	Simply dropped apart.
South abutment. North side surface of middle layer.	51.	Clean separation at joint.
No. 3 pier. North side surface of middle layer.	98.	Broke entirely through bricks.
No. 1 pier. South side surface at base of pier.	51.	Clean separation at joint.
No. 3 pier. South side surface of arch.	62.	"

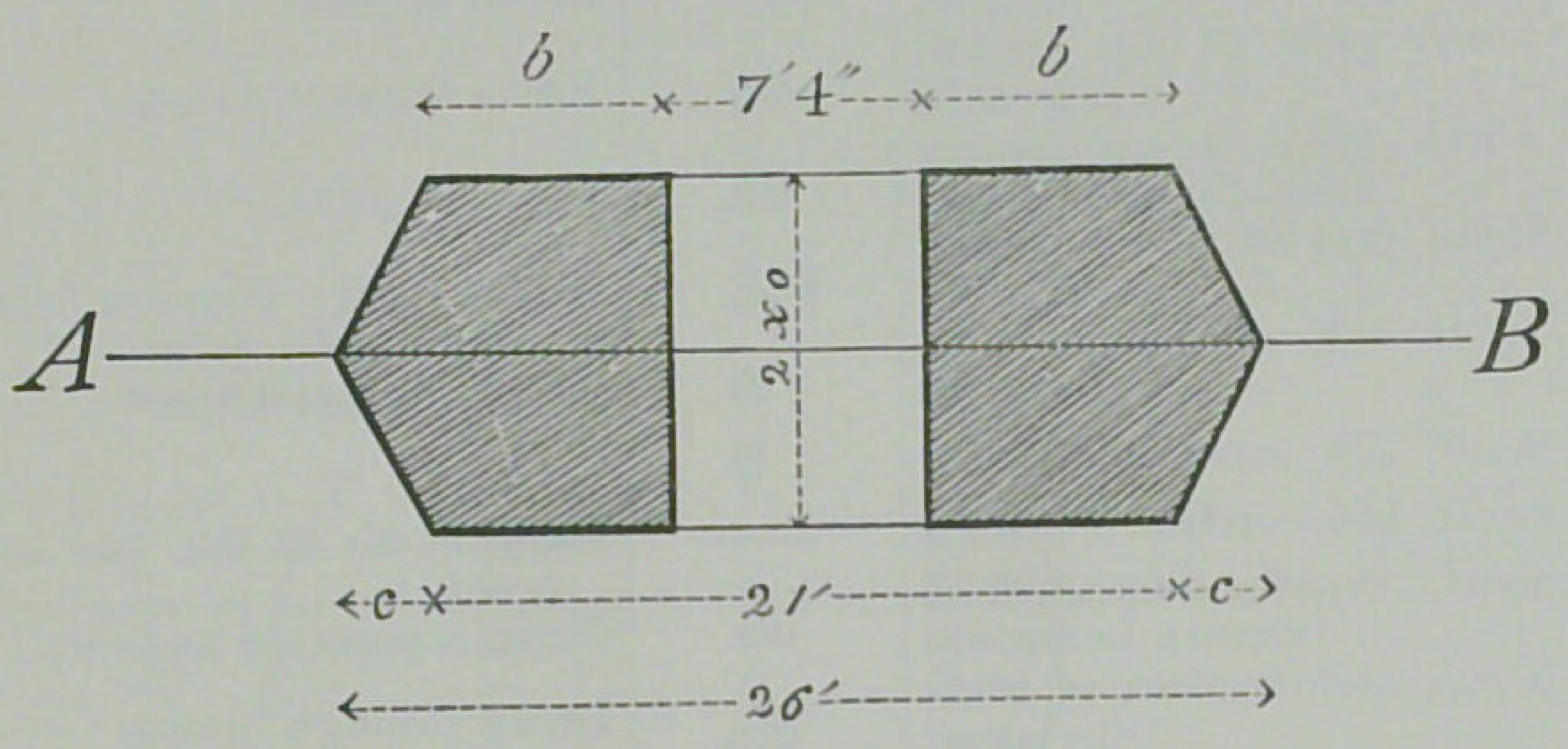
Average value of the tensile strength per. sq. in. =62.4 lbs.

* Determined by the Imperial Railway Department.

(*f*). *Calculation of the seismic stability of the pier.*—The pier was theoretically weakest, and actually broke at the junction with the wells. The seismic stability or acceleration of the earthquake motion (α) necessary for fracturing normally the pier at this section can be calculated by the formula :

$$\alpha = \frac{g\,\mathrm{F}\,\mathrm{I}}{\mathrm{W} f x_0}.$$

The signification of the several symbols in this equation is as follows.



$2x_0 = 10'$ $c = 2'\,6''$
$b = 6'\,10''$

g is the acceleration due to gravity, $=9800$ mm.

x_0 ,, half thickness of the pier, $=60$ in.

F ,, tensile strength of the brick-work per sq. in., $=62.4$ lbs. (See table VI).

I ,, moment of inertia of the section of fracture with respect to its longer axis AB (see the figure), or

$$\mathrm{I} = \frac{4bx_0^3}{3} + \frac{2x_0^3c}{6} = \frac{2x_0^3}{3}\left(2b + \frac{c}{2}\right)$$

$$= \frac{2 \times 60^3}{3}(14'\ 8'' + 15'')$$

$=25.770.000.$ (expressed in inches).

W is the weight of the whole structure above the top of the well, or

W=(weight of the girders, rails, etc.)
+(weight of the brick pier)
=165,6 tons + (0,0608 lb. × vol. of the pier)
=371.000 lbs. + (0,0608 lb. × 11.019.700 c. in.)
=1.051.800 lbs.

f is the height of the centre of gravity of the same, = 294 in.

Substituting these values in the above equation, we find

$$\alpha = \frac{9800 \times 62{,}4 \times 25.770.000}{1.051.800 \times 294 \times 60} = 850 \text{ mm. per sec. per sec.}$$

This value of α, which represents the seismic stability of the pier normal to its plane, will be seen to be far smaller than the actual N and S component intensity of the earthquake-motion at the Kiso-gawa, namely, 1500 mm. per sec. per sec. This shows how small the seismic stability of ordinary bridge piers is.

If the foundation of the pier had consisted of a single elliptical well, instead of a pair of circular ones joined by an arch, the seismic stability of the same pier would have been 1270 mm. per sec. per sec., the loss of the strength due to the existence of the arch thus amounting to 1270−850=420 mm. per sec. per sec.

III. Chimneys.

25.—*Chimneys* mean here only brick factory-chimneys.

Chimneys are broken by earthquake motion usually a little above the middle height, it being extremely rare that they receive damage near the base. This fact has already been observed at the time of the great Mino-Owari earthquake of Oct. 28th 1891, and has been explained by Mr. K. Nakamura, Director of the Central Meteorological Observatory, on the assumption that these columns are weakest at the height corresponding to the centre of percussion with respect to the base.

The top portion of a chimney, when thrown down, has never been projected to a great distance from the base. Again, the broken portion very often does not fall down but remains at its original height and presents the phenomenon of rotation. It is probable also that an earthquake motion of great amplitude is required to overturn columns or structures of large dimensions.

Of the 49 factory chimneys * broken at the time of the Tokyo earthquake of June 20th 1894, only eleven had the top portion thrown down, as shown in the following table.

Chimney.	Total height. *Shaku*.†	Height of the section of fracture. *Shaku*.	Distance between the centre of the chimney base and the point of projection *Shaku*.	Distance between the chimney foot and the point of projection. *Shaku*.	Direction of projection.
Tokyo Hōhei-Kōshō	70.4	56.9	16.1	11.1	——
Paper-tile Factory, Fukagawa.	42.	32.2	23.	20.	S60°W
Onagigawa Brick Factory, Fukagawa.	132.	68.	25.	17.	S75°W
Okada Seimai-sho, Fukagawa.	60.	40.	20.	17.	E
Tokyo Shinji-kan.	150.	130.	20.	13.	N10°W
Do.	120.	111.8	13.	7.	SW
Kakizawa Factory, Honjō	70.	47.3	15.	9.	NE
Okura Factory, Fukagawa.	75.	49.	30.	24.	N73°E
Tanaka Factory, Fukagawa.	78.	60.	17.	11.	S75°W
Sakurada Beer Company.	50.	28.5	15.	11.	S55°E
Tokyo Seifun Company, Fukagawa.	50.	30.	22.	18.	N40°E

* See the report by Messrs. Mano, Tanabe and Yasunaga, 5th Vol. of the Report of the Earthquake Investigation Committee.

† 1 *shaku* = 300 mm.

According to the above table, the maximum distances of the point of projection of the broken top from the centre and the exterior side of the chimney base were respectively 30 and 24 ft., there being no definite relation between these distances and the heights of the chimneys.

The forty-nine chimneys already referred to were broken at the $\frac{24}{100}$th to $\frac{94}{100}$th part of their heights, which varied between 39 and 150 *shaku*. The mean height of the plane of fracture deduced from all these forty-nine cases, which is at the $\frac{67}{100}$th part of the chimney height, may be regarded as defining statistically the probable point of fracture of chimneys.

Again, according to the report by Messrs. Mano, Tanabe and Yasunaga, the tensile strength of the joint of the brick-work, tested in the cases of eleven chimneys, varied between 19.3 and 58.2 lbs. per sq. in., giving an average of 36.1 lb. per sq. in. It is however, to be noted that this value represents the mean strength of the stronger portions of the brick-work, those test pieces whose strength was nearly zero and too weak to be tested, having been excluded from the experiment.

26. *Period of vibration of brick columns.*—The following table gives the result of experiments made on the free vibration of the six brick columns Nos. 13, 13''', 13'''', 15, 21* and 24 (tables I and III); the double amplitude and the period having been measured from the diagrams traced by a rigid pointer attached to the top of each column, which was set in motion by striking it with the hand. See figs. 24-27.

No. of column.	Thickness. mm.	Height. mm.	Maximum range of motion. mm.	Average period. sec.	Remarks.
13	110	1700	9.6	0.16	The square of the ratio of the heights of the columns, Nos. 13 and 13'''= 2,66; the ratio of their periods =2,4.
13'''	,,	990	3.	.072	
13''''	,,	810	2.	.053	The square of the ratio of the heights of the columns, Nos. 13''' and 13'''' =1.49; the ratio of their periods =1.4.
15	,,	1800	11.	.20	
21*	233–131	1810	4.5	.11	The column No. 21* is hollow, all the others being solid.
24	220	1587	1.9	.10	

The *height* in the third column of the table, which gives the mean of the whole height of a column and its height above the iron fixing frame (fig. 6), is intended to represent the effective length coming into play in the free vibration.

From the above table the period of vibration of a column will be seen, as ought to be, nearly proportional inversely to the thickness and directly to the square of the height. Thus the thickness of the two columns Nos. 15 and 21*, whose heights are nearly equal, are in the ratio of 1 : 2, while their periods are in the ratio of 2 : 1. Again, the periods of the three columns Nos. 13, 13''' and 13,'''' whose sections are equal to one another, are roughly in the ratios of the squares of their heights.

The period of free lateral vibration of the six columns experimented upon, which varied between 0.053 and 0.20 second, is rather slower than might have been supposed. These numbers may be used for estimating roughly the period of vibration of a given column. Let us suppose, for instance, the height and thickness of a square brick column to be respectively 30m. and 3m. the quality of the brick-work being similar to that of column No. 24. The period of the given column will then be about 2.6 seconds.

27. *Period of vibration of chimneys.* In the 21st volume of the Report of the Earthquake Investigation Committee, there is an interesting report by Messrs. Mano and Tanakadate on the vibration of a chimney which belonged to the former astronomical observatory of the University. The section of the chimney, whose height was 19,6 *shaku*, was nearly uniform and 2,7 *shaku* square.

According to the experiments by Messrs. Mano and Tanakadate, the period of vibration of the chimney increased somewhat with the amplitude. Thus for instance, in one experiment the average period of vibration was 0.55 second, while the period of the maximum motion, whose range (at the top of the chimney) was 90 mm., was 0,85 second. In another case, the average period was 0.62 second, while the period of the maximum motion, whose top range was 162 mm., was, 0.99 second. From these results it may be concluded that the period of vibration of the chimney in question at the time of a great earthquake would be about one second.

Let us now imagine for the sake of illustration, a solid brick square column, whose height and thickness are five times those of the chimney above considered, siz. 98 and 17,5 *shaku* respectively. If the material of the supposed column be exactly similar to that of the chimney, the period of vibration of the former may be inferred from that of the latter to be about 5 seconds. This result, though only roughly approximate, shows that the period of large brick columns and consequently also of chimneys, would be very slow. As a matter of fact slow oscillations are usually perceived at the top of large chimneys.

28. *Height of the weakest section of chimneys.*—From § 27, it follows that a large brick chimney behaves at the time of a destructive earthquake as if the "centre of percussion" really existed, the period and the height of the chimney being many times greater than the period and range of the earthquake motion. In this case, the centre of percussion, which is at the same time a *steady point*, marks theoretically the weakest place of the chimney. (See § 25.) I shall next give a few practical illustrations.

(*a*). *The chimney of the Goryō-kyoku Factory at Ōji*—This chimney, whose height was 100 *shaku* and whose section was circular, was broken by the earthquake of June 20th 1894 at the height of 61 *shaku*. (See fig. 28.)

Theoretically the centre of percussion of this chimney was at the height of 54,4 *shaku*; while the calculated position of the section of fracture would be according to § 25 at a height of $100 \times \frac{67}{100} = 67$ *shaku*. These two values must, in calculations of this kind, be regarded as sufficiently close to one another, and give the mean height of 60,7 *shaku* for the most probable section of fracture, which conclusion has almost exactly been fulfilled in the actual case. I am, however, inclined to regard this result as indicating the accidental weakness at the particular height of 61,7 *shaku*, since the chimney might have been broken more easily 2,9 *shaku* higher at the height of 64,6 *shaku*, where there had existed a discontinuity in the thickness of the brick-work.

(*b*). *The Great Chimney of the Imperial Steel Works.*—The height of this chimney, now in the course of construction, is 80 m., its external slope being $\frac{3.75}{100} = 0,0375$. The section is circular, the outer diameters at the base and the top being respectively 7,80 and 4,36 m. (fig. 29.)

The centre of percussion of the chimney is found by calculation to be at the distance of 42,6 m. from base, or a little above the middle height. On the other hand the calculated coefficient of $\frac{67}{100}$ (§ 25), gives the weakest point at the height of $80 \times \frac{67}{100} = 53,2$ m. Taking the mean of these two results, we get the value of 48 m. for the probable height of fracture. As, however, there is a discontinuity in the thickness of the wall 2 m. lower, (AB in the accompanying figure), the natural conclusion is to suppose the weakest section to be at the height of 46 m.

29. *Seismic stability of chimneys.* According to the results obtained above, chimneys seem to be seismically weakest at the height y' given by the following equation:

$$y' = \frac{1}{2}\left\{h + \left(y_0 \times \frac{67}{100}\right)\right\}, \tag{15}$$

in which y_0 is the total height, and h the height of the centre of percussion. It is to be remarked that this equation implies uniformity of construction throughout the chimney.

Having determined by equation (15) the probable height of fracture of a given chimney, we can calculate its seismic stability by equation (3) :

$$\alpha=\frac{g\mathrm{FI}}{x_0 fw\mathrm{V}}\,, \tag{3}$$

in which the several symbols have the same signification as before, the only difference being that the section of fracture is no longer at the base. When the section is circular, we obtain

$$\alpha=\frac{\pi g\mathrm{F}(d_2^4-d_1^4)}{32d_2 fw\mathrm{V}}\,, \tag{16}$$

in which d_2 and d_1 are respectively the outer and the inner diameters of the section of fracture. As examples I shall calculate the seismic stability of the two chimneys already considered.

(*a*) *The chimney of the Goryō-kyoku Factory at Ōji.* In this case, we have :—

The height of the section of fracture=61,7 *shaku*,

d_2=9,2 *shaku*,

d_1=5,5 ,,

V=volume of the portion fractured=1110 cubic *shaku*,

f=height of the centre of gravity of the same portion above the plane of fracture=17,3 *shaku*,

F=54.4 lbs. (adopting the result obtained by Messrs. Mano, Tanabe and Yasunaga, from the testing experiments on the brick-work of this chimney).

Substituting these values in equation (16), we find

α=8,41 *shaku*=2550 mm. per sec. per sec.,

which gives the seismic stability of the chimney. As already remarked however, there is reason for supposing the strength of the brick-work at the section of fracture to have been imperfect, the tensile strength of F=54.4 lbs. per sq. in. being consequently too high. It is probable that the chimney was broken actually by an intensity of earthquake motion much lower than the above value of α.

(*b*). *The Great Chimney of the Imperial Steel Works.* We have :—

The height of the probable section of fracture = 46 m.

d_2 = 5,585 m.

d_1 = 4,865 m.

f = 14,5 m.

wV = weight of the portion to be fractured = 269,31 French tons.

Assuming F to be 34,6 French tons per sq. m., (which corresponds to the tensile strength of 50 lbs. per sq. in.), we obtain

$$\alpha = 634 \text{ mm. per sec. per sec.}$$

It is to be remarked that the value of F used in the calculation is much higher than the average in ordinary cases. (See § 25.) The seismic stability α will, however, be seen to be much smaller than that of the Oji chimney, due doubtless to the difference in height. It is probable that very tall chimneys can never resist great earthquakes. If possible therefore, the reduction of the height of chimneys is seismically very important.

I may here add that the design of this great chimney of the Imperial Steel Works has subsequently been changed, the height having been reduced to 70 m. without altering the thickness of the brick-work.

IV.—Overturning of Columns.

30. Overturning experiments were made in connection with the determination of the intensity of motion of destructive earthquakes. Consequently the dimensions of the columns employed in the

experiments have been of magnitude comparable to those of the stone lanterns, tomb-stones, etc., observed in actual cases.

The details of the columns, whose number amounted to 42, are given in table VII. Of these, two were iron pipes, 8 solid or hollow brick columns, 18 hollow columns or boxes of wood, and the remaining 14 solid columns of wood.

The bases of the wooden columns, as illustrated in Fig. 30, were made slightly concave, so as to make them sit well. For brick columns, a similar precaution has been taken.

The height of the columns varied between 1150 and 242 mm. and the dimension of the section between $\overline{300}^2$ and $\overline{90}^2$ sq. mm.

TABLE VII.—List of the Columns Overturned.

No.	Dimensions* (in mm.)
	(I) Iron pipes, (7 mm. thick).
k_1	(diameter) 152; (height) 940
k_2	(„) 150; („) 480
	(II) Solid Square Columns of Brick.
l_1	230×230×1150
l_2	230×230×700
l_3	230×230×660
	(III) Hollow Square Columns of Brick.
m_1	230×230×1100, (54 mm. thick).
o_1	185×185×950, (45 „ „).
n	233×233×900, (50 „ „).
m_2	230×230×790, (54 „ „).
o_2	183×183×600, (45 „ „).
	(IV) Hollow Square Columns of Wood, (1 cm. thick.)
A_1	300×300×900
A_2	„ „ ×605
B	274×274×817
C_1	242×242×970
C_2	„ „ 727
C_3	„ „ 484
D_1	210×210×850
D_2	„ „ 633
D_3	„ „ 420
E_1	180×180×907
E_2	„ „ 727
E_3	„ „ 544
E_4	„ „ 362
F_1	151×151×910
F_2	„ „ 754
F_3	„ „ 602
F_4	„ „ 450
F_5	„ „ 304

No.	Dimensions* (in mm.)
	(V) Solid Square Columns of Wood.
G_1	120×120×970
G_2	„ „ 843
G_3	„ „ 727
G_4	„ „ 595
G_5	„ „ 480
G_6	„ „ 362
G_7	„ „ 242
H_1	90 × 90 ×726
H_2	„ „ 665
H_3	„ „ 637
H_4	„ „ 544
H_5	„ „ 450
H_6	„ „ 361
H_7	„ „ 270

* The dimensions are the outside dimensions.

31. *Method of the experiments.*—The method of the experiments consisted in putting the columns on the shaking table and overturning them by giving proper movements to the latter. The shaking intensity deduced from the diagram of motion of the table was then compared with the theoretical values of the acceleration necessary for overturning the columns. Specimen diagrams are shown in figs. 31 and 32.

In the experiments, the range of motion of the shaking table varied between 29.5 and 120 mm., and the period between 0.4 and 1.47 seconds. Further, the maximum acceleration of motion varied between 750 and 10.700 mm. per sec. per sec. The column C_3, however, could not be overturned even with the utmost intensity of motion at our disposal, which fact shows the difficulty of overturning certain stable objects by earthquakes. The result of the experiments is shown in table VIII.

TABLE VIII.—Summary of results of the Overturning Experiments.

$2x$ = thickness of column ;
$2y$ = its height ;
$\alpha = \dfrac{x}{y} g$ = acceleration, which may overturn the column.

$2a$ = range of motion of the shaking table ;
T = its period ; A = its maximum acceleration.

No. of Column.	$2x$ mm.	$2y$ mm.	$\alpha=\dfrac{x}{y}g$ mm./sec.^{2}	$2a$ mm.	T sec.	A mm./sec.^{2}	Ratio $\dfrac{\alpha}{A}$
l_1	230	1150	1960	100	0.89	2480	0.8
m_1	230	1100	2050	110	1.11	1760	1.2
o_1	185	950	1910	108	1.27	1310	1.5
n	233	900	2540	111 110	1.00 .90 mean	2190 2670 2430	1.0
m_2	230	790	2860	110 111	.96 .87 mean	2350 2890 2620	1.1
l_2	230	700	3220	104	.69	4280	0.8
l_3	230	660	3400	114	.77	3780	0.9
o_2	183	600	3040	111	.83	3180	1.0
k_1	152	940	1580	32 113 110	.77 1.14 1.47 mean	1060 1720 1000 1260	1.3
k_2	150	480	3[illegible]60	114 113 32 114	.90 .92 .61 .95 mean	2770 2640 1700 2480 2400	1.3

No. of Column.	$2x$ mm.	$2y$ mm.	$\alpha=\frac{x}{y}g$ mm./sec.^{2}	$2a$ mm.	T. sec.	A. mm./sec.^{2}	Ratio $\frac{\alpha}{A}$
A_1	300	900	3260	114	0.79	3600	0.9
B	274	818	3290	114	.79	3600	
				116	.66	5240	
					mean	4420	0.7
C_1	242	970	2450	113	.88	2870	
				112	1 00	2210	
					mean	2540	1.0
C_2	242	727	3260	114	.77	3780	
				116	.62	5950	
				114	.81	3420	
				111	.85	3020	
					mean	4040	0.8
C_3	242	484	4900	119	.51	9000 (not overturned)	
				120	.47	10700 (not overturned)	—
D_1	210	850	2420	113	.88	2870	
				,,	.91	2700	
				110	.96	2350	
				$33\frac{1}{2}$	.46	3120	
					mean	2510	1.0
D_2	210	634	3250	114	.78	3700	
				,,	.81	3420	
				111	.86	2960	
				,,	.82	3250	
					mean	3330	1.0
E_1	180	908	1940	112	1.01	2160	
				,,	1.09	1860	
				32	.56	2010	
				$34\frac{1}{2}$	.34	5900(?)	
				$32\frac{1}{2}$	.48	2780	

No. of Column.	$2x$ mm.	$2y$ mm.	$\alpha=\frac{x}{y}g$ mm./sec.^{2}	$2a$ mm.	T sec.	A mm/sec.^{2}	Ratio $\frac{\alpha}{A}$
E_1	180	908	1940	33	0.48	2830	
				35	.41	4100	
					mean	2620	0.7
E_2	180	727	2430	100	.83	2860	
				,,	.86	2660	
				113	.98	2320	
				112	1.04	2040	
				110	.99	2220	
				111	.96	2370	
				110	.94	2450	
					mean	2420	1.0
E_3	180	544	3240	114	.77	3780	
				,,	.80	3510	
				,,	.86	3040	
					mean	3440	0.9
E_4	180	362	4870	116	.62	5950	
				,,	.63	5750	
					mean	5850	0.8
F_1	152	910	1640	113	1.14	1720	
				110	1.33	1230	
				$31\frac{1}{2}$	.71	1230	
				$30\frac{1}{2}$	.67	1340	
				$32\frac{1}{2}$	.53	2280	
				30	.97	630	
				$29\frac{1}{2}$	.75	1030	
				$30\frac{1}{2}$	.75	1070	
				30	.69	1240	
					mean	1310	1.2
F_2	152	754	1960	112	1.09	1860	
				30	.84	840	
				$31\frac{1}{2}$	.59	1780	
				32	.55	2080	
				$32\frac{1}{2}$	.63	1610	
				31	.48	2650	

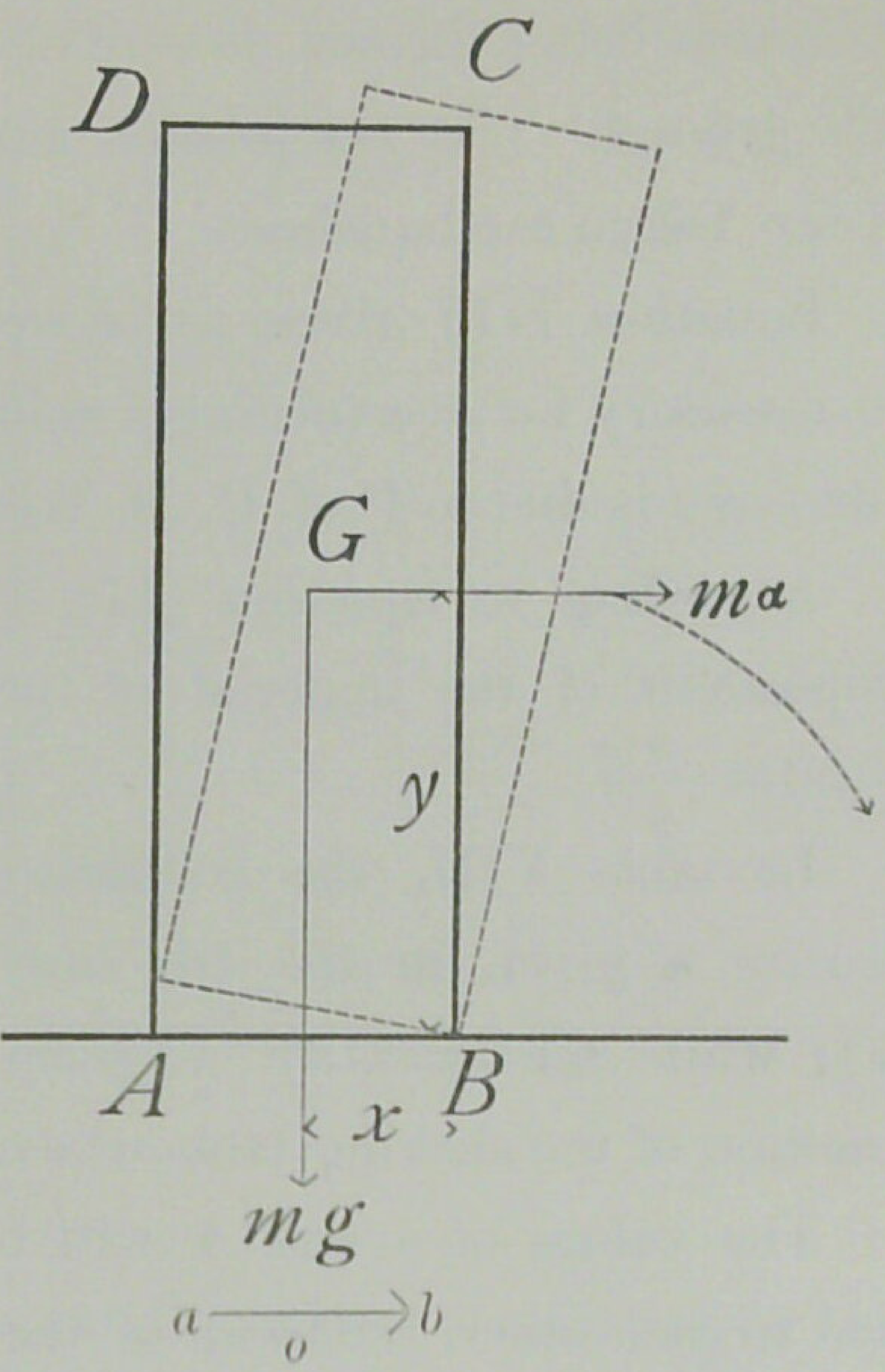
No. of Column.	$2x$ mm.	$2y$ mm.	$a=\frac{x}{y}g$ mm./sec.^{2}	$2a$ mm.	T sec.	A mm./sec.^{2}	Ratio $\frac{a}{A}$
F_2	152	754	1960	29½	0.75	1030	
				30½	.69	1260	
					mean	1640	1.2
F_3	152	602	2470	112	1.04	2040	
				31	.60	1700	
				29½	.65	1370	
				31½	.57	1910	
				32	.45	3100	
					mean	2020	1.2
F_4	152	450	3310	114	.86	3040	
				113	.87	2950	
				32½	.50	2560	
				33	.46	3070	
				32½	.57	1980	
					mean	2720	1.2
F_5	152	304	4900	117	.66	5290	
				116	.66	5240	
					mean	5270	0.9
G_1	120	970	1210	110	1.30	1280	
				31½	.82	920	
				,,	.81	950	
				,,	.84	880	
					mean	1010	1.2
G_2	120	844	1390	110	1.30	1280	
				31½	.91	750	
				,,	.91	750	
				,,	.84	880	
					mean	920	1.5
G_3	120	728	1620	111	1.28	1330	
				31½	.81	950	
				,,	.84	880	
				,,	.75	1100	
					mean	1070	1.5

No. of Column	$2x$ mm.	$2y$ mm.	$\alpha=\frac{x}{y}g$ mm./sec.^{2}	$2a$ mm.	T sec.	A mm./sec.^{2}	Ratio $\frac{\alpha}{A}$
G_4	120	596	1970	112	1.13	1720	
				108	1.27	1310	
				31½	.74	1130	
				,,	.80	970	
				,,	.68	1340	
					mean	1290	1.5
G_5	120	480	2450	112	.99	2250	
				110	1.06	1940	
				110	1.01	2170	
				32	.60	1750	
				34½	.45	3350	
				32	.65	1490	
				55	.81	1500	
				56	.64	2700	
					mean	2140	1.1
G_6	120	362	3260	101	.83	2900	
				101	.77	3350	
				,,	.81	3030	
				114	.89	2840	
				113	.87	2940	
				114	.84	3180	
				32½	.51	2460	
				,,	.50	2560	
				32	.64	1540	
				56½	.63	2800	
				57½	.54	3880	
					mean	2860	1.1
G_7	120	242	4860	116	.65	5400	
				,,	.68	4950	
				,,	.66	5240	
					mean	5200	0.9
H_1	90	726	1220	111	1.18	1570	
				,,	1.30	1290	
				31½	.90	770	
				,,	.91	750	

No of Column.	$2x$ mm.	$2y$ mm.	$\alpha = \frac{x}{y} g$ mm/sec.^{2}	$2a$ mm.	T sec.	A mm/sec.^{2}	Ratio $\frac{\alpha}{A}$
H_1	90	726	1220	$31\frac{1}{2}$	.88	800	
				,,	.77	1060	
					mean	1040	0.9
H_2	90	666	1330	111	1.22	1470	
				,,	1.26	1380	
				$31\frac{1}{2}$	.88	800	
				,,	.89	790	
				,,	.91	750	
				,,	.88	800	
				,,	.85	830	
					mean	974	0.9
H_3	90	638	1390	112	1.18	1[illegible]90	
				111	1.21	1490	
				$31\frac{1}{2}$	.81	950	
				,,	.84	880	
				,,	.82	930	
					mean	1180	0.9
H_4	90	544	1620	112	1.10	1840	
				,,	1.19	1570	
				$31\frac{1}{2}$	.88	880	
				,,	.81	950	
				,,	.72	1200	
				,,	.74	1130	
				$30\frac{1}{2}$	.65	1420	
					mean	1290	1.3
H_5	90	454	1940	112	1.14	1710	
				,,	1.12	1770	
				$31\frac{1}{2}$	.71	1230	
				,,	.74	1130	
				,,	.75	1100	
				$32\frac{1}{2}$	.50	2560	
					mean	1600	1.2
H_6	90	362	2440	113	.98	2320	
				,,	1.06	1990	

No. of Column.	$2x$ mm.	$2y$ mm.	$a = \frac{x}{y} g$ mm/sec.^{2}	$2a$ mm.	T sec.	A mm./sec.^{2}	Ratio $\frac{a}{A}$
H_6	90	362	2440	32	0.65	1490	
				,,	.73	1180	
				,,	.63	1590	
				32	.63	1590	
				31½	.41	3700	
				,,	.58	1850	
				,,	.61	1670	
					mean	1930	1.3
H_7	90	270	3260	115	.85	3140	
				116	.80	3590	
				110	.95	2400	
				35	.45	3390	
				33	.53	2320	
				32½	.60	1780	
				31½	.44	3200	
				32	.40	3950	
				32½	.54	2200	
					mean	2890	1.1

General mean 1.07



The overturning of a column by earthquake motion is a complex phenomenon, incapable of exact solution by a simple formula. Let us however, simplify the question by supposing the period of the earth-motion not very short in comparison with the proper period of rocking of the column. In this case, the body will not be overthrown at once, but will continue to move with the ground so long as the intensity of motion of the latter is not sufficiently great. Let ABCD be a column resting at AB on the ground. Let G be the centre of gravity of the body, y its height, and x the horizontal distance between it and a corner edge B. If at a given moment the ground be moving from the equilibrium position o towards the extremity b, the acceleration (a) will be directed towards o. Impressing now the acceleration a in the opposite direction, uniformly to the column and the ground, we reduce the latter to a state of rest. The motion of the body relative to the ground however, is not thereby altered, the result being that the body is impressed with the acceleration a in direction $o\,b$. The least value of the earth acceleration necessary for overturning the column whose mass is m is therefore given by the equation :—

$$m\ a\ y = m\ g\ x$$

or

$$a = \frac{g\ x}{y}. \qquad (17)$$

the column being thrown towards the same direction as the motion of the ground. (See the present author's Notes on the Mino-Owari and the Tokyo earthquakes.)

Equation (17) gives, as is well known, the horizontal acceleration necessary for overturning a column at rest. Its introduction to seismology is due to Prof. C. D. West.

According to equation (17), the overturning acceleration α is independent of the material of the columns and depends simply on the ratio $\frac{x}{y}$.

In table VIII, the columns are arranged in groups. The quantity α given in the 4th row has been calculated by equation (17); while the quantity A given in the 7th row is the intensity of motion of the shaking table which actually overturned the body.

The values of α and A will be seen in general to be practically equal to each other, the mean of the ratios α : A being 1,07: 1. It must be remembered however, that this conclusion is limited by the condition that the period of the earthquake motion is not very small in comparison to the period of rocking of the columns.

32. *Remarks on equation (17).*—That the overturning acceleration α is independent of the nature of the material of the columns is shown in table IX. Thus the columns, n, m_2, l_2, l_3, o, k_1 and k_2 were of brick or iron ; while the others D_1, D_2, E_3, F_1 and F_2 were wooden boxes. α was, however, nearly the same for the columns of both kinds, when their exterior dimensions were approximately equal.

TABLE IX.—SHOWING THE NON-EXISTENCE OF RELATION BETWEEN THE MATERIAL OF A COLUMN AND ITS OVERTURNING ACCELERATION.

($2x$, $2y$, α and A have the same signification as in Table VIII.)

Column	Material.	$2x$ mm.	$2x$ mm. mean	$2y$ mm.	$2y$ mm. mean	$\alpha = \frac{x}{y} g$ mm./sec.^{2}	α mean	A mm./sec.^{2}	A mean
n	brick	233	232	900	845	2540	2700	2430	2530
m_2	,,	230		790		2860		2620	
D_1	wood (hollow)		210		850		2420		2510
l_2	brick	230	214	700	653	3220	3220	4280	3570
l_3	,,	230		660		3400		3780	
o	,,	183		600		3040		3180	
D_2	wood (hollow)	210	195	635	590	3250	3250	3330	3390
E_3	,,	180		544		3240		3440	
k_1	iron pipe		152		940		1580		1260
F_1	wood (hollow)		152		910		1640		1390
k_2	iron pipe		150		480		3060		2400
F_2	wood (hollow)		152		450		3310		2720

33. *An absolute scale of destructive earthquakes.*—In the investigation of the great Mino-Owari earthquake of Oct. 28th 1891, I estimated the intensity of the earthquake motion from the observation of numerous stone lanterns, tomb-stones, etc., with whose dimensions those of the columns in the present experiments were comparable.* As, further, the period of the principal motion in this destructive earthquake was not very short but probably one or two seconds, the values of the maximum acceleration at various places calculated by means of equation (17) must be very near to the truth.

* F. Omori; Note on the great Mino-Owari earthquake of Oct. 28th 1891.

The following *absolute scale of destructive earthquakes*, or the relation between the maximum acceleration of the earthquake motion and the damage produced, has been deduced chiefly from analysis of the Mino-Owari earthquake, the intensity being arbitrarily divided into seven classes *I–VII*. It is to be noted that the scale applies principally to Japan.

I. *Maximum acceleration=300 mm. per sec. per sec.*—The motion is sufficiently strong that people generally run out of doors. Brick walls of bad construction are slightly cracked; plasters of some old *dozo* (godowns) shaken down; furniture overthrown; wooden houses so much shaken that cracking noises are produced; trees visibly shaken; waters in ponds rendered slightly turbid in consequence of the disturbance of the mud; pendulum clocks stopped; a few factory chimneys of very bad construction damaged.

II. *Maximum acceleration=900 mm. per sec. per sec.*—Walls in Japanese houses are cracked; old wooden houses thrown slightly out of the vertical; tomb-stones and stone lanterns of bad construction overturned, etc. In a few cases, changes are produced in hot springs and mineral waters. Ordinary factory chimneys are not damaged.

III. *Maximum acceleration=1200 mm. per sec. per sec.*—About one factory chimney in every four is damaged; brick houses of bad construction partially or totally destroyed; a few old wooden dwelling houses and ware houses totally destroyed; wooden bridges slightly damaged; some tomb-stones and stone lanterns overturned; *shoji* (Japanese paper-covered sliding doors) broken; roof-tiles of wooden houses disturbed; some rock fragments thrown down from mountain sides.

IV. Maximum acceleration=2000 mm. per sec. per sec.—All factory chimneys are broken; most of the ordinary brick buildings partially or totally destroyed; some wooden houses totally destroyed; wooden sliding doors and *shoji* mostly thrown out of the grooves; cracks 2 or 3 inches in width produced in low and soft grounds; embankments slightly damaged here and there; wooden bridges partially destroyed; ordinary stone lanterns overturned.

V. Maximum acceleration=2500 mm. per sec. per sec. All ordinary brick houses are very severely damaged; about 3 % of the wooden houses totally destroyed; a few *tera*, or Buddist temples, thrown down; embankments severely damaged; railway lines slightly curved or contorted; ordinary tomb-stones overturned; *ishigaki*, or masonry walls, damaged here and there; cracks 1 or 2 feet in width produced along river banks; waters in rivers and ditches thrown over the banks; wells mostly affected with changes in their waters; landslips produced.

VI. Maximum acceleration=4000 mm. per sec. per sec. Most of the *tera*, or Buddhist temples, are thrown down; 50 to 80 % of the wooden houses totally destroyed; embankments shattered almost to pieces; roads made through paddy fields so much cracked and depressed as to stop the passage of wagons and horses; railway lines very much contorted; large iron bridges destroyed; wooden bridges partially or totally damaged; tomb-stones of stable construction overturned; cracks a few feet in width formed in the ground, accompanied sometimes by the ejection of water and sand; earthenware buried in the ground mostly broken; low grounds, such as paddy fields, very greatly convulsed, both horizontally and vertically, sometimes causing trees and vegetables to die; numerous landslips produced.

VII. Maximum acceleration much above 4000 mm. per sec. per sec. All buildings, except a very few wooden houses, are totally destroyed; some houses, gates, etc., projected 1 to 3 feet; remarkable landslips produced, accompanied by faults and shears of the ground.

In the above scale of the seismic intensity, the earthquake motion has been assumed to be entirely horizontal. This supposition would not, except in places very near to the epicentre, cause sensible errors in the result. (See the note on the Mino-Owari earthquake.)

The comparison of the present absolute scale with the Rossi-Forel system and that employed by our Central Meteorological Observatory will be seen from the following schedule.

Absolute scale of destructive earthquakes. (Acc. in mm. per sec. per sec.)	The intensity scale employed by the Central Meteorological Observatory.	Rossi-Forel Scale.
	Slight.	I II
	Weak.	III IV V
——— I .... 300 mm./sec.²	Strong.	VI VII
II .... 900 III .... 1200 IV .... 2000 V .... 2500 VI .... 4000 VII ....>4000	Violent.	— VIII IX X — —

It may here be noted that the number of (wooden) houses destroyed by an earthquake is not necessarily proportional to the intensity of the latter. The following list is the result obtained by taking means from numerous observations.

	Percentage of (wooden) houses destroyed in a town or village.		Maximum (horizontal) acceleration of the earthquake motion.	
(1)	2 or 3	%	2600	mm. per sec. per sec.
(2)	15	,,	3400	,,
(3)	50	,,	3900	,,
(4)	80	,,	4500	,,
(5)	100	,,	Infinite	

The meaning of (5) is that a few wooden houses could not be totally destroyed by an earthquake, however violent.

34. *Large buildings.*—Equation (17) can not be applied to pagodas, *bell-temples* and other buildings, whose dimensions are very much greater than the range of motion of the ground. As their rocking periods are long, the earthquake motion is to be regarded, with respect to these bodies, as acting in *shocks*. They will never be overthrown, unless the range of motion of the ground amount to several feet.*

35. In the present paper, I have confined myself to the consideration of some of the simplest brick structures. Hollow columns and other structures shall form the subject of the second series of fracturing experiments.

Tokyo. May, 1899.

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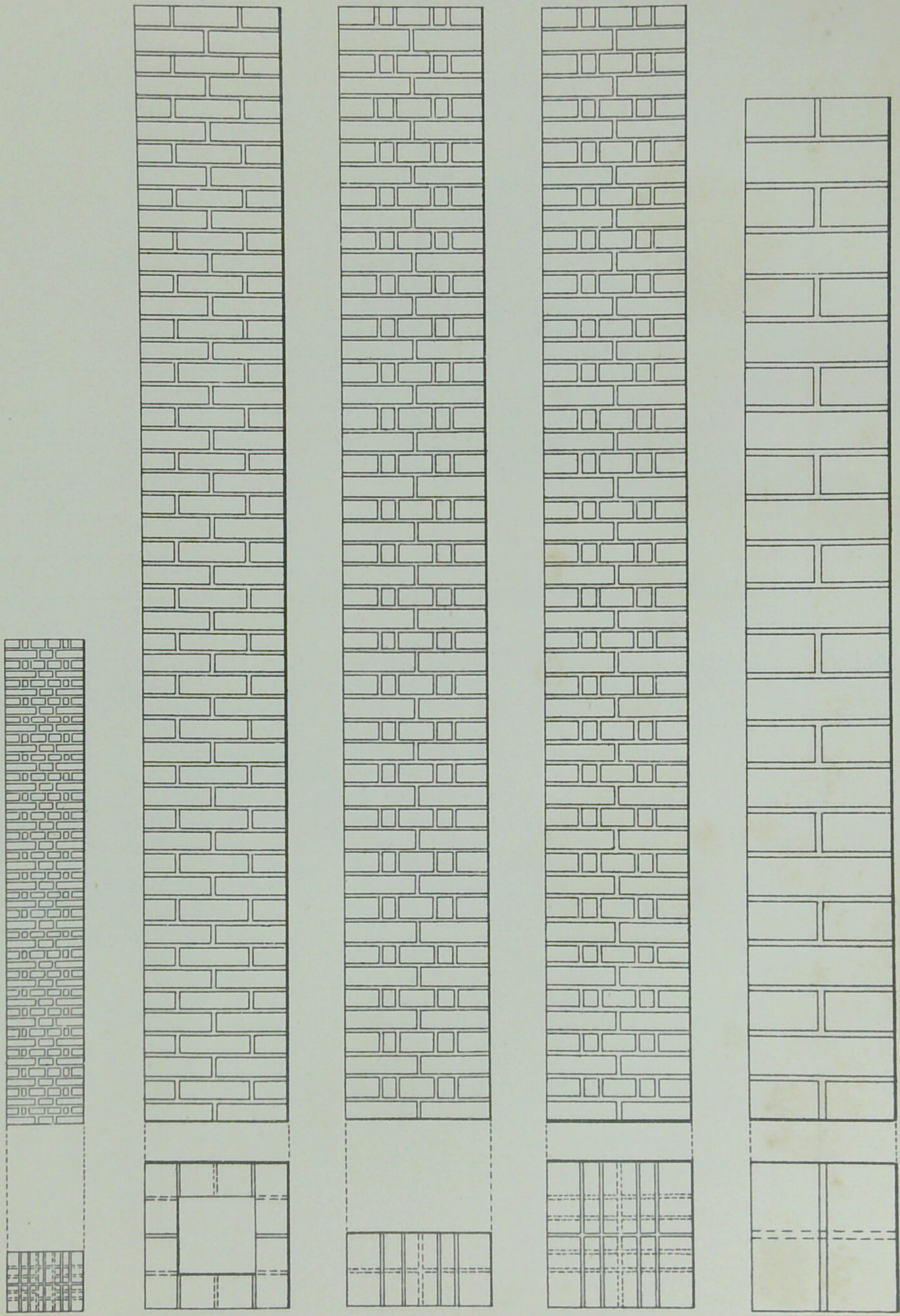
* See F. Omori: *On the overturning of Columns.* Seis. Jour. Japan. Vol II.

Fig. 4. Brick Columns.



Scale $\frac{1}{10}$

Fig. 5. Brick Column With Concrete Base.

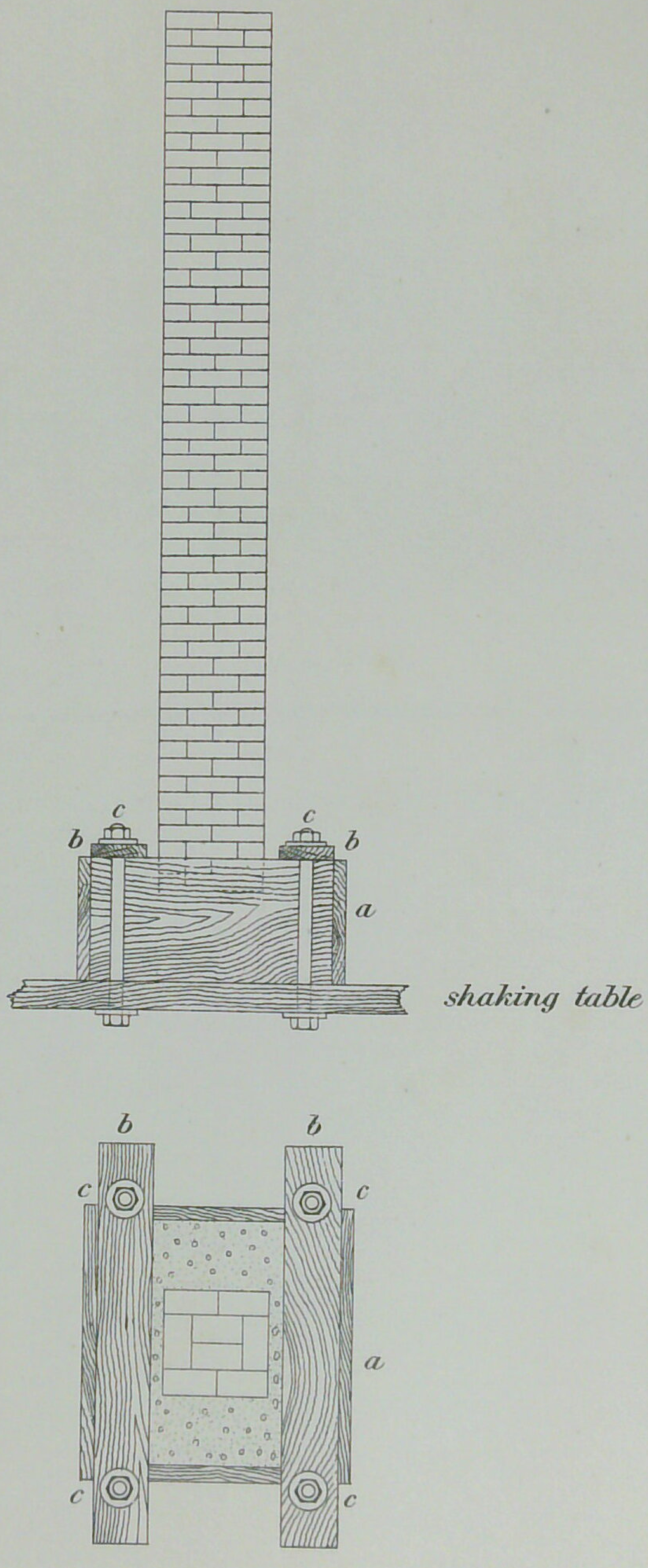


Fig. 6. Iron Frame for fixing the Columns to the Shaking Table.

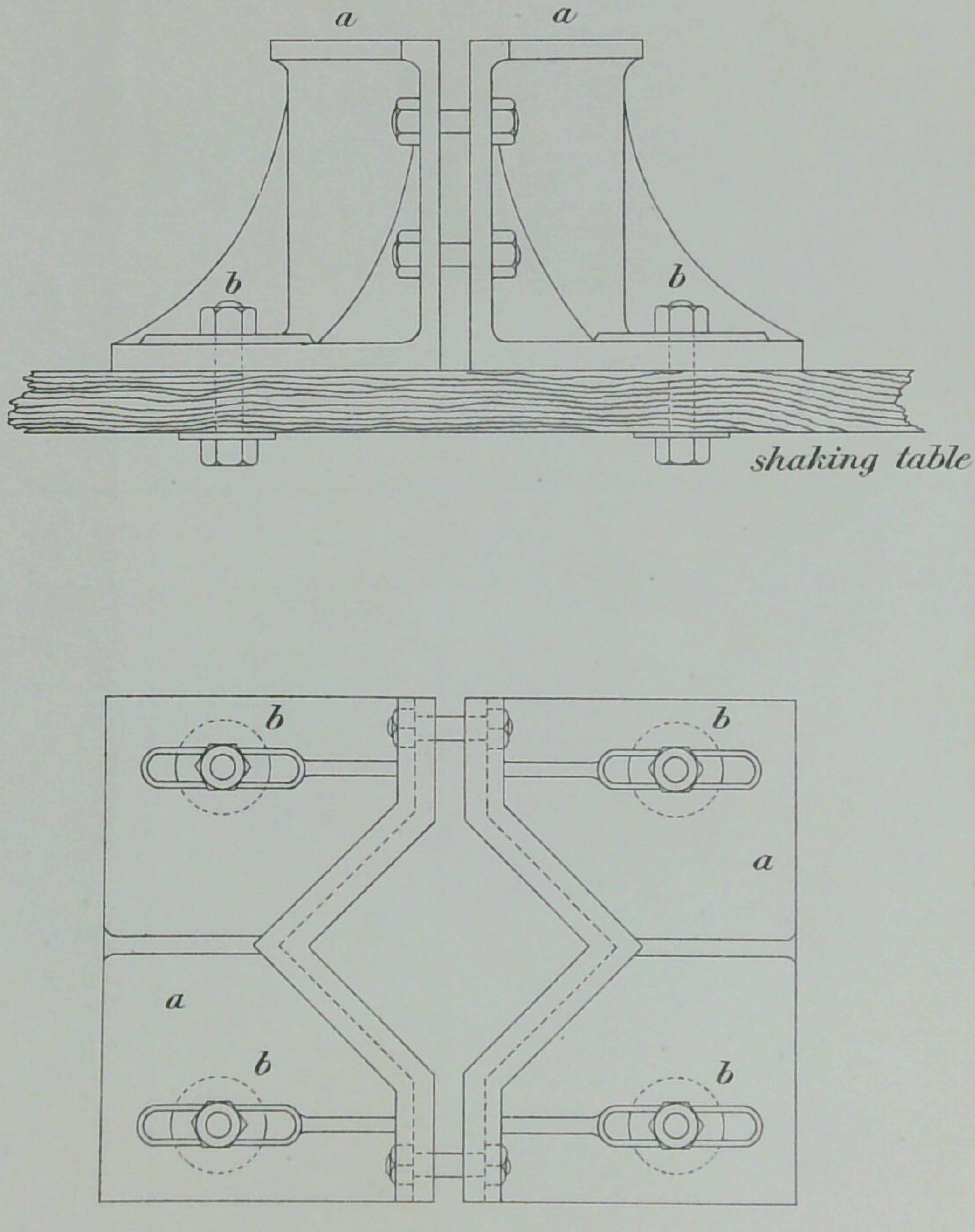
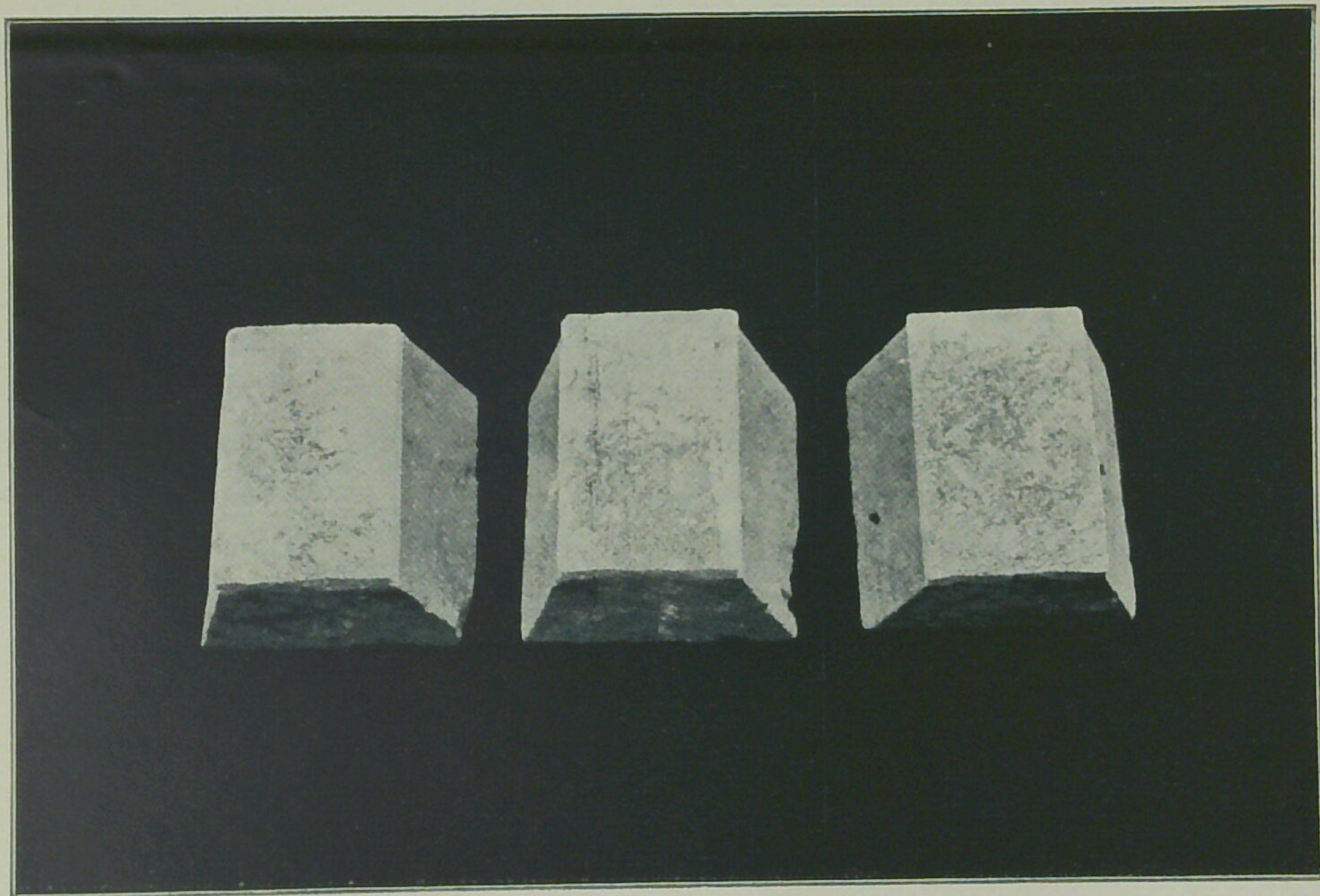


Fig. 7.



(c) (b) (a)

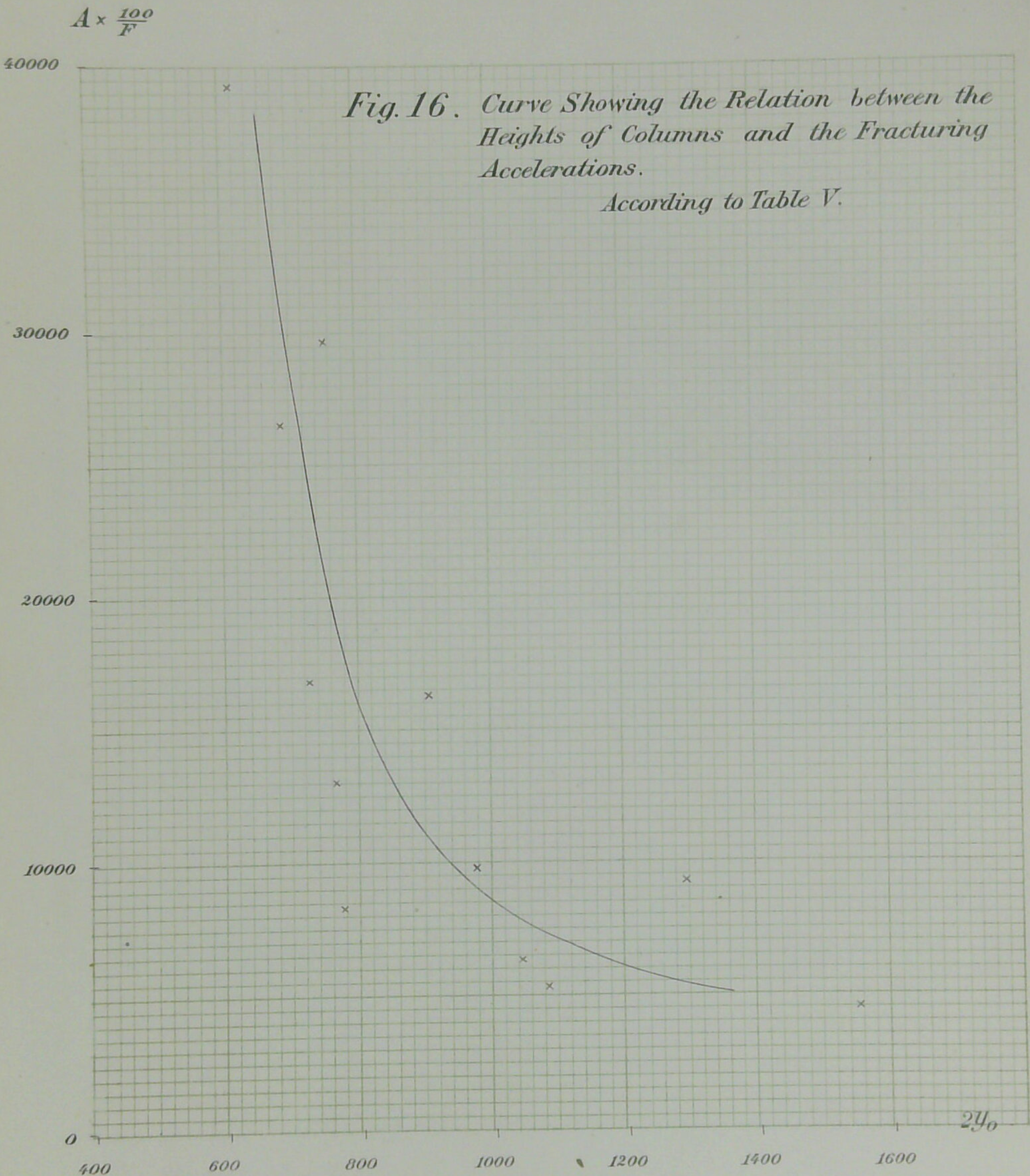


Fig. 16. Curve Showing the Relation between the Heights of Columns and the Fracturing Accelerations.

According to Table V.

$2y_0$ = *height of the column above the section of fracture*

$A \times \frac{100}{F}$ = *fracturing acceleration, in mm per sec. per sec.*

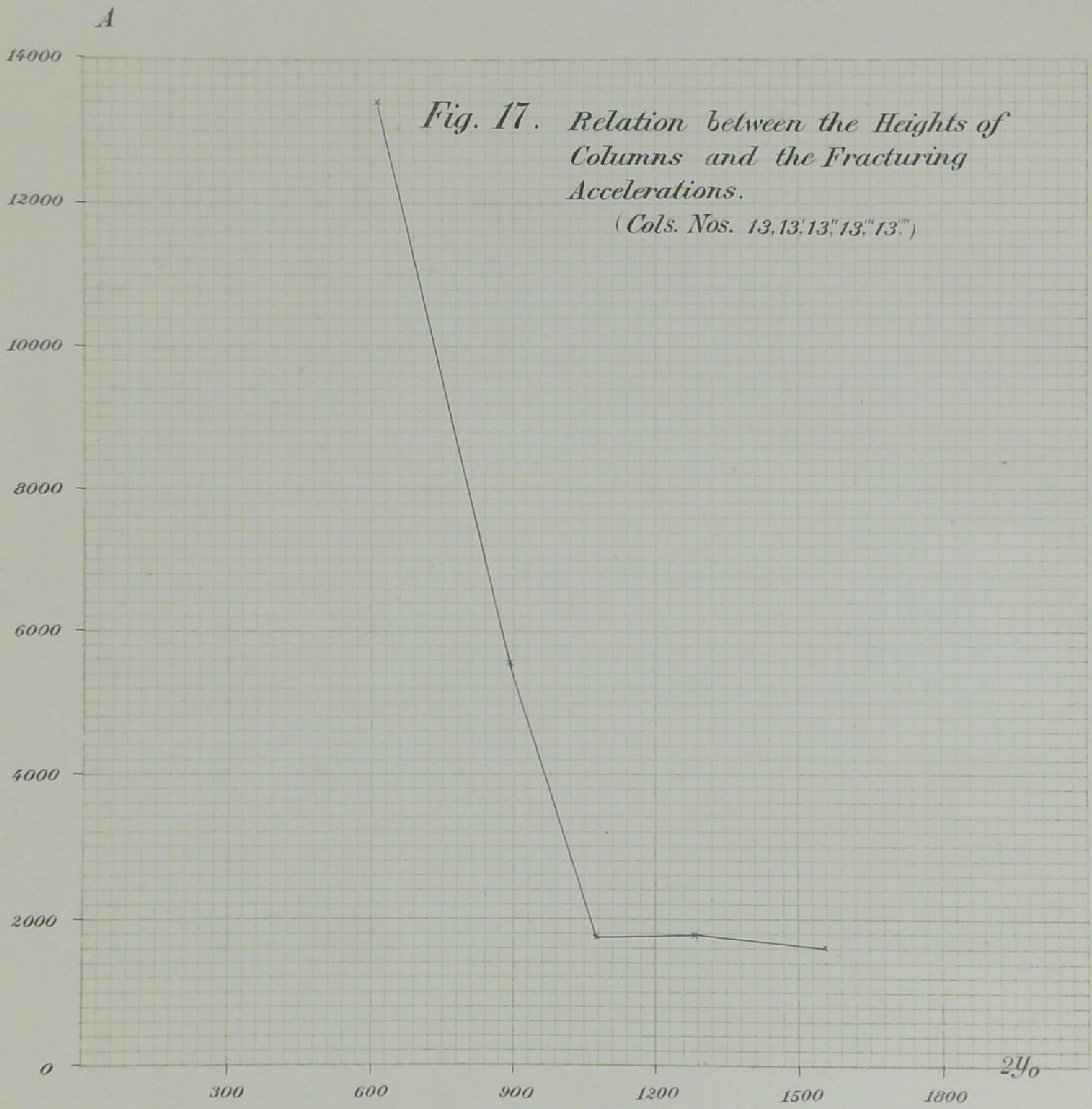


Fig. 17. Relation between the Heights of Columns and the Fracturing Accelerations.
(Cols. Nos. 13, 13', 13'', 13''', 13'''')

$2y_0$ = height of the column above the section of fracture
A = fracturing acceleration, in mm per sec. per sec.

Fig. 18. A Column of Uniform Strength.

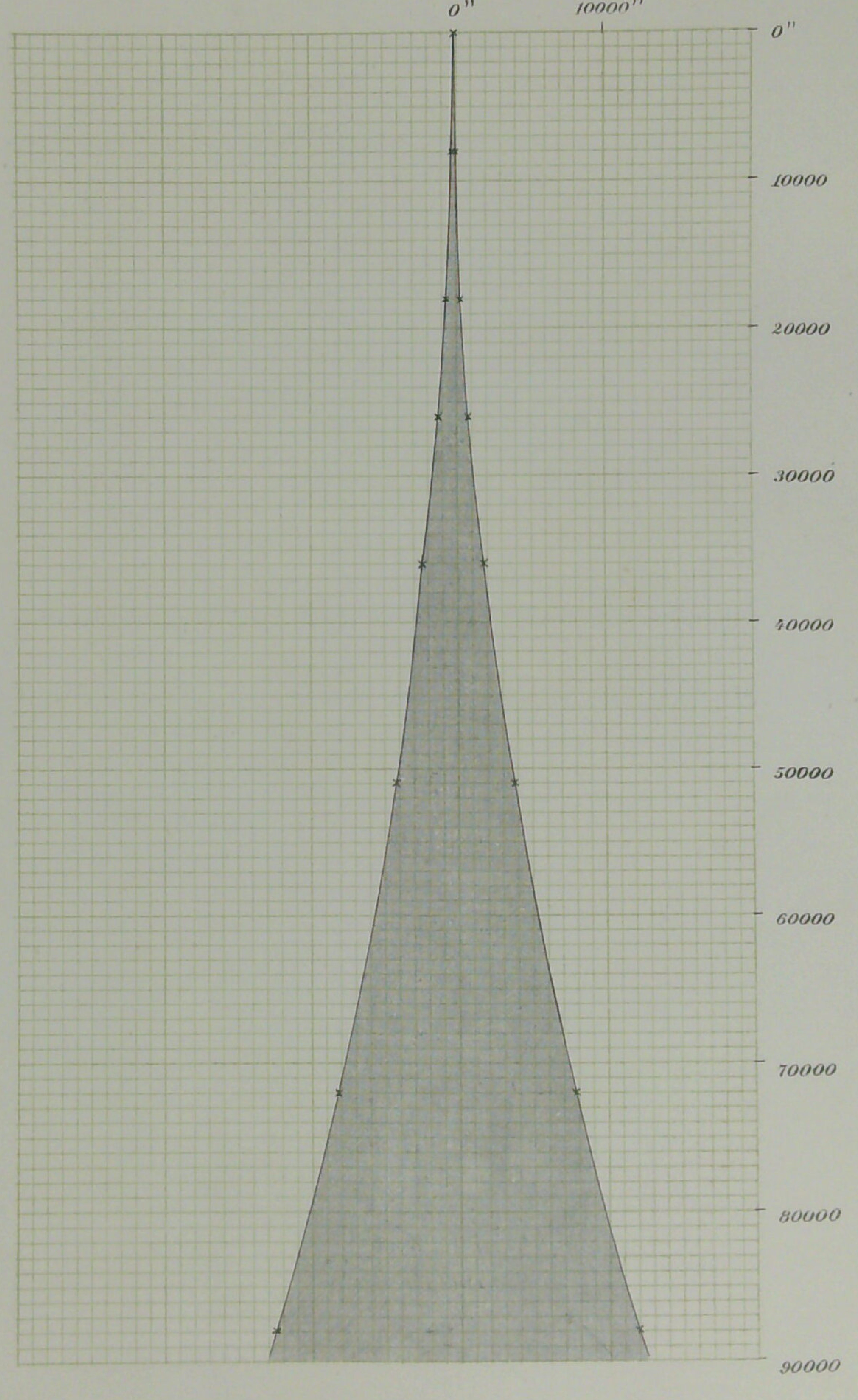
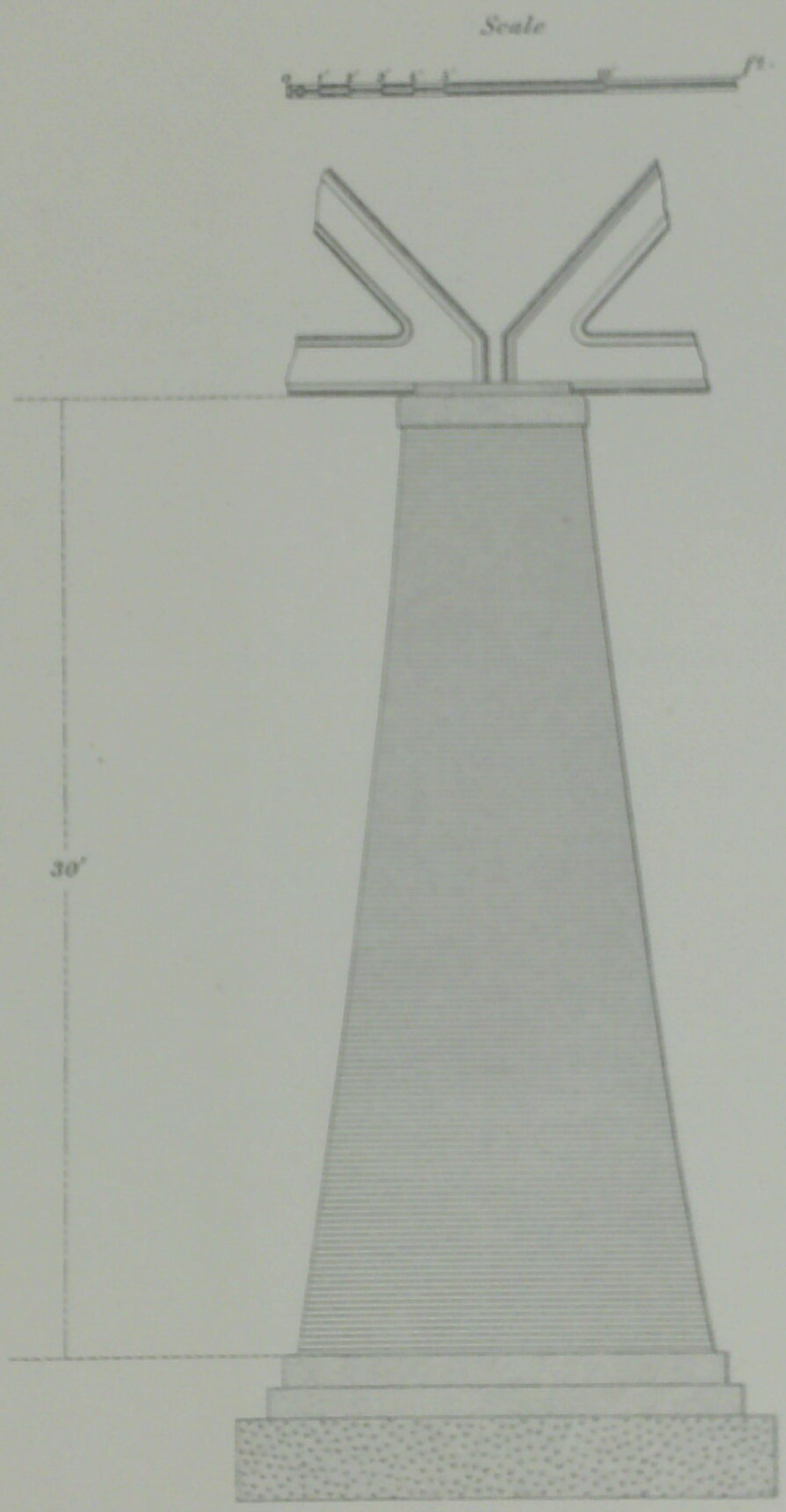


Fig. 19. A Bridge Pier.



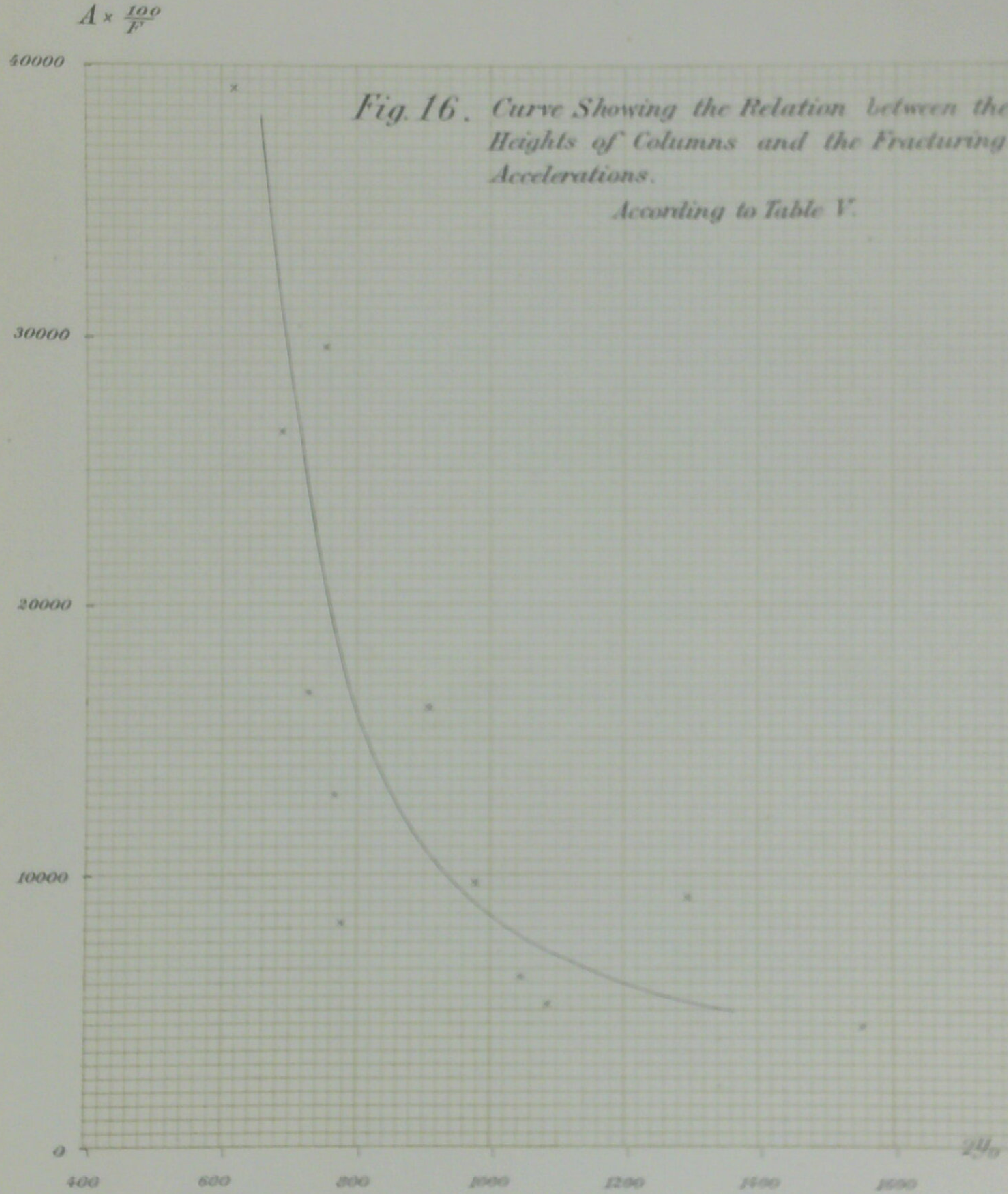


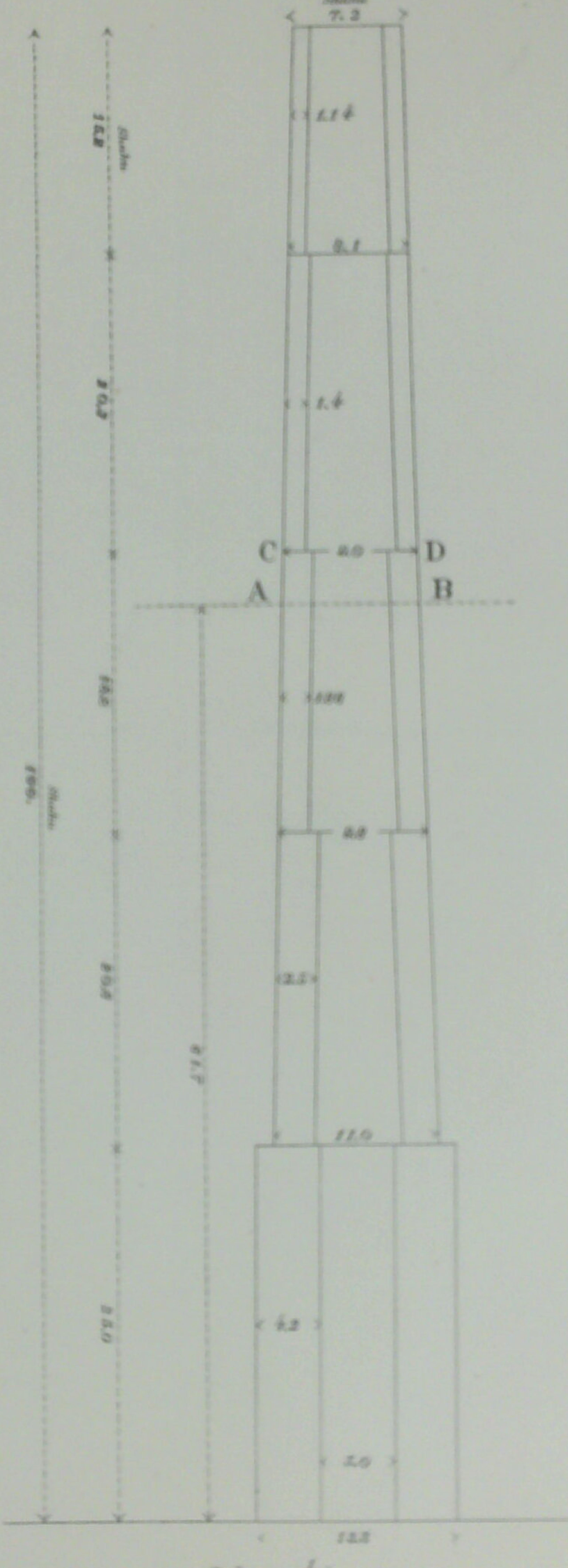
Fig. 16. Curve Showing the Relation between the Heights of Columns and the Fracturing Accelerations.

According to Table V.

$2y_0$ = height of the column above the section of fracture

$A \times \frac{100}{F}$ = fracturing acceleration, in mm per sec. per sec.

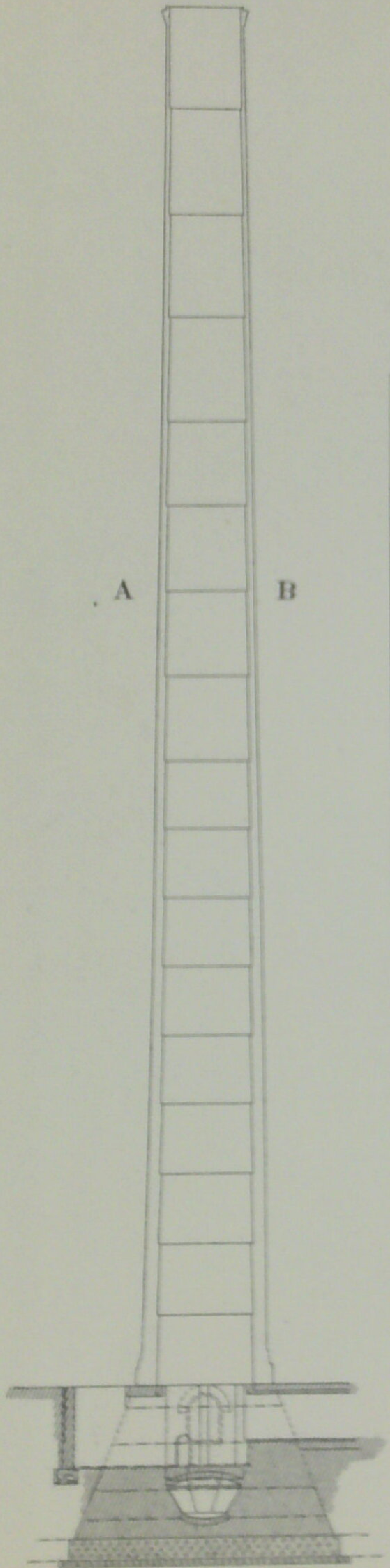
Fig. 28. Chimney of Oji Goryokyoku Factory.



Scale $\frac{1}{150}$

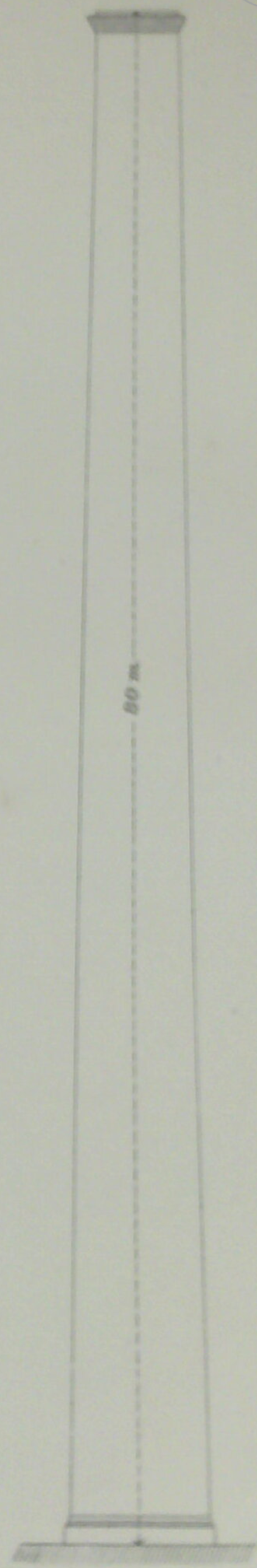
(AB—Section of fracture.)

Fig. 29. The Great Chimney of the Imperial Steel Works.



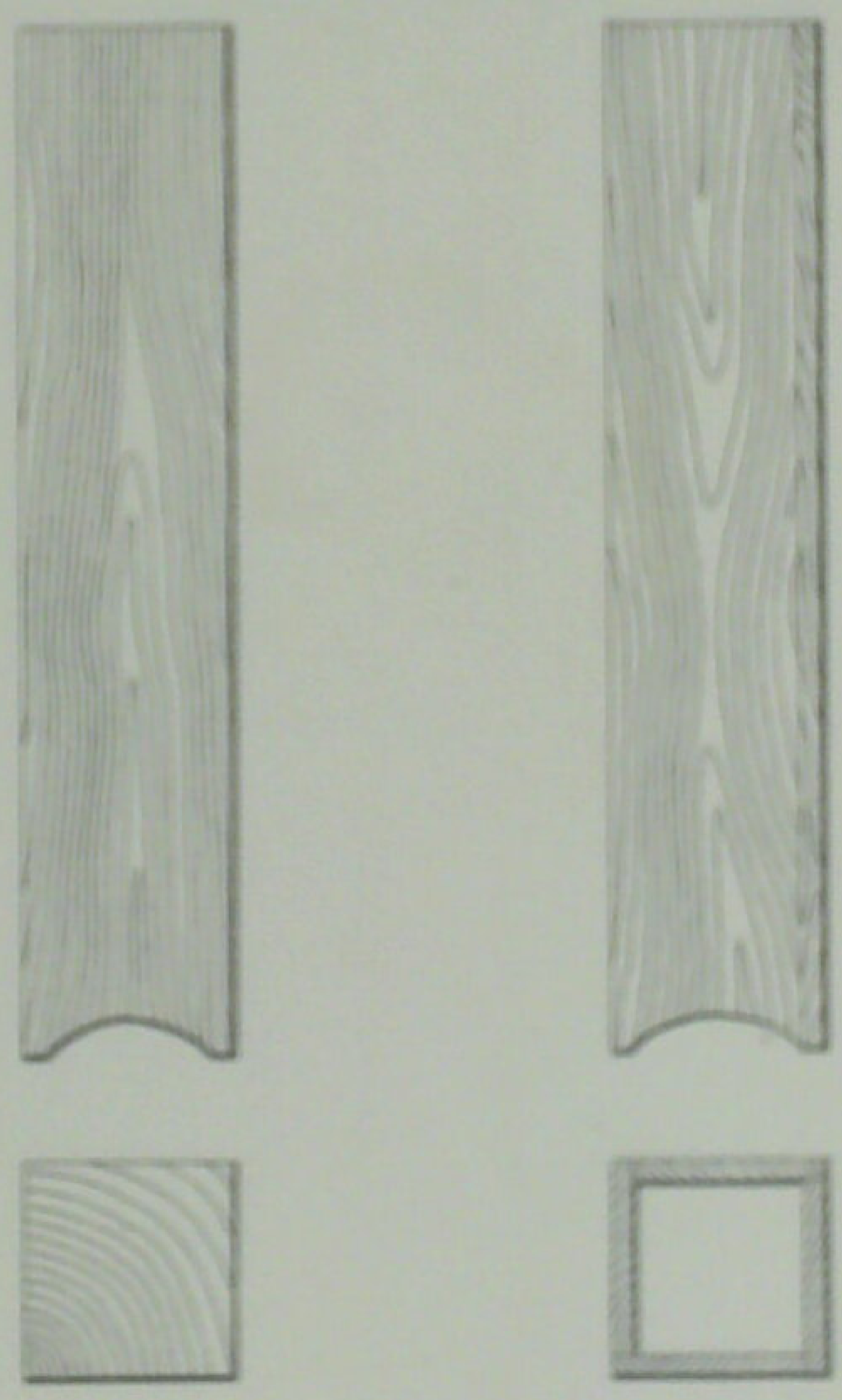
Structure of the Chimney.

Height m	Thickness of brick work m
80	[illegible]
74	[illegible]
68	[illegible]
62	[illegible]
56	[illegible]
51	[illegible]
46	[illegible]
41	[illegible]
36	[illegible]
32	[illegible]
28	[illegible]
24	[illegible]
20	[illegible]
16	[illegible]
12	[illegible]
8	[illegible]
4	[illegible]
1.5	[illegible]
0	[illegible]



Scale $\frac{1}{400}$

Fig. 30. Wooden Columns.



ERRATA.

The "Publications"; No. 4, page 111, the 12th line from top.

for $h=10$ ft., *read* $h=20$ ft.

The "Bulletin"; Vol. II, No. 2, page 196.

for $a=\dfrac{lgf}{x_0 fW}$, *read* $a=\dfrac{lgF}{x_0 fW}$.

The "Bulletin"; Vol. III, No. 2, page 38, the 20th line from top.

for $1^h\ 20^m\ 17^s$, *read* $5^h\ 20^m\ 17^s$.

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