BEHAVIOR AND DESIGN OF WOODEN BUILDINGS
SUBJECTED TO EARTHQUAKE

By

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1 INTRODUCTION

In Japan wooden construction is a traditional one dating back 2000 years and about 90 percent of present Japanese buildings are constructed of wood. Therefore, almost all damage to buildings caused by the severe earthquakes in the past was damage to buildings of this type of construction and there have been made extensive investigation in this connection [1] from which the features of damage to the buildings have been clarified.

Construction of Japanese Wooden Buildings. The typical construction of Japanese wooden buildings is described below.

1) The column and horizontal member composing the frame are connected with each other by skilfully made tenon joints as shown in Figs. 1 and 2, and no metal connectors such as bolts or steel plates are, in general, employed in the construction. Diagonal bracings are not so extensively used, although their use as a means against earthquake forces has gradually increased during these thirty years.

2) The wall panel is usually the so-called "Shinkabe" which is made by covering bamboo lath peculiar to Japan with mud plaster and is thinner than the pillars around it; sometimes the wall is finished with Japanese style wooden sheathings. In recent years, foreign style wooden sheathings finished with mortar over metal lath have come to be used. This type of wall is called "Okabe" in Japanese.

3) The roof materials of buildings are Japanese tiles, cement tiles, asbestos cemente slates and metal plates, and thatched roofs are sometimes seen in the rural area. The weight of a heavy roof is about twice that of a light roof.

Characteristics of Earthquake Damage. The typical features of earthquake damage to wooden buildings are as follows:

1) At the joint connecting the column and girder, the failure usually occurs as shown in Figs. 1** and 2, accompanying the falling of finishings as shown in Fig. 3. As the inclination of the structure increases, its restoring force against distortion decreases due to the structural deterioration and roof weight and finally becomes negative, which results in the complete collapse of the building as shown in Fig. 4.

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**Figs. 1, 2, 3 and 4 were kindly offered by courtesy of Architectural
  Department, Faculty of Eng., and Earthquake Research Inst., Univ.
  of Tokyo.

2) In case of two story buildings, the first story structure usually suffers more damage and it is often seen that the lower structure falls down on the ground while the upper one survives the earthquake shock.

3) Earthquake damage to wooden buildings is considerably influenced by the ground on which they stand, and, in general, the softer is the subsoil, the greater is the rate of damage to the buildings. Furthermore, the damage due to differential settlements of foundations is observed for the wooden buildings standing on soft ground.

4) Sliding of the building as a whole is sometimes seen when there are no anchor bolts connecting the sill to the foundation. Also, the damage to superstructure is often observed when the foundation construction is poor to resist seismic force.

5) The relation between earthquake damage to wooden buildings and the maximum acceleration of ground motion obtained from several past large earthquakes is as follows:[2]

0.1g: cracking and falling-off of the wall finishings.

0.2g: slight distortion of structural frame with some deterioration of the connecting joints.

0.3g: most houses inclined, a few collapsed.

0.4g: most houses destroyed.

Vibrational Characteristics of Wooden Buildings. Dynamical behaviors of wooden buildings obtained from past investigations are summarized below.

1) Natural periods in small amplitude vibrations are generally in the range of 0.18 to 0.4 sec for one story buildings and of 0.25 to 0.6 sec for two story buildings. These variations are due to the roof weight and structural rigidity which mainly depends on the quantity of frames with walls or bracings.

2) The natural periods mentioned above increase with the amplitudes of vibration, amounting to twice to three times the said periods in the case of large amplitude vibrations.

3) During earthquakes the buildings vibrate with their fundamental periods and their amplitudes become large when those periods are close to the predominant periods of the ground on which they stand.

4) Fractions of critical damping observed in small amplitude vibrations of wooden buildings are in the range of 0.05 to 0.15.

2 GROUND CONDITION AND EARTHQUAKE DAMAGE

It is a well known fact that the damage of wooden buildings due to earthquake is closely related with ground conditions and the damage rate is evidently high where the alluvial soil is deeply deposited[3]. This fact is supposedly attributable to two reasons: (1) because of the characteristics of a soft ground that the periods of vibration extend over quite

a wide range without a distinct predominant period and the ground motion lasts for a long time, a building continues to be in a state of quasi-resonance with the ground even after it has been partially damaged and, consequently, its natural period has been lengthened, and (2) because of the insufficient bearing capacity of the ground, a large differential settlement takes place at the foundation of the building so that it induces excessive stresses in the superstructure which accelerate the collapse of the building.

In the following, the damage rates of wooden buildings in the Fukui Earthquake of 1948 and the Great Kanto Earthquake of 1923 will be reviewed from this standpoint and will be presented as an evidence of the inter-relationship between the earthquake damage and the ground conditions.

Fukui Earthquake. With respect to each of 60 villages in and around the Fukui Plain, the damage rate of wooden buildings, i.e. the ratio of the number of collapsed buildings to the total number of existing buildings, was investigated and the ground conditions of all investigated villages were divided into four categories:

I.   Entire village is located upon the Alluvium

II.   Approximately one-half of the village is located upon the alluvium and the other half upon the Bed Rock

III.   Bed Rock, geologically older than the Tertiary

VI.   Sand Dune

The distance from the epicenter to each investigated village is in the range of 3.1 to 15.5 km and, with respect to the Fukui Earthquake, another investigation[4] revealed that the relation between the damage rate and the distance from the epicenter could statistically be given by the equation

$$y = 1800R^{-2.26}$$

Then, in accordance with this equation, the damage rate in each village has been modified as if it were at the distance $R = 7.5$ km from the epicenter in order to eliminate the effect of the distance. If the damage rates thus modified are plotted against the four categories of the ground condition, Fig. 5 is obtained.

The exact distribution of the depth of the Alluvium in the Fukui Plain and geotechnical properties thereof are not known. Nevertheless, Fig. 5 shows an evident tendency that, in general, the damage rate upon the Alluvium is far larger than that upon the Bed Rock, and that the damage rate in a village located on the Sand Dune is intermediate between the Alluvium and the Bed Rock.

Great Kanto Earthquake. Information on the number of wooden buildings in every block of Tokyo city collapsed and so heavily destroyed beyond any repair by the Great Kanto Earthquake is made available from the report of survey written by T. Matsuzawa[5], and by taking the ratio of these buildings to the total number of existing buildings at the date of the Earthquake the damage rate in each bolck has been determined.

On the other hand, for information on the thickness of Alluvial deposit under each block, the Subsoil May of Tokyo[6] has been resorted to. The Alluvial deposit in Tokyo area consists of sandy, silty and clayey materials and is highly varied, and mechanical properties thereof are in the ranges shown in the following Table.

	Sandy	Silty	Clayey
Number of Sample	130	9C	99
Cohesion (kg/cm^2)	0.06 - 0.26	0.12 - 0.34	0.09 - 0.35
Angle of Internal Friction (degree)	14 - 33	6 - 21	2 - 8

The total number of blocks, with which both the damage rate and the thickness of Alluvial deposit have thus been made known, is 1,464. Then, by sorting these data into intervals of 5 m of Alluvial thickness and by computing the averages of the damage rate and the Alluvial thickness within each interval, small circles in Fig. 6 can be obtained.

From Fig. 6 it will be seen immediately that the damage rate increases with the increase of the thickness of Alluvial deposit and, where the thickness exceeds about 30 m, the damage rate shows a tendency to increase rapidly. If we assume an equation of the type $y = ab^x + c$ for representing the relationship between the damage rate and the thickness of Alluvial deposit and determine unknown constants by the method of least square, the following equation can be obtained:

$$y = 0.982(1.086)^x + 0.26$$

Recently, H. Kawasumi has made an analysis to disclose the effects of individual, constituent layers of the Tokyo subsoil to the magnitude of seismic coefficient, and found that the effects of the layer of peat and the artificially filled top-soil are most remarkable.[7]

3 RELATION BETWEEN EARTHQUAKE DAMAGE TO WOODEN BUILDINGS AND QUANTITIES OF FRAMES THEREIN

In the great earthquake of Fukui of 1948, the authors investigated the relationship between the earthquake damage to wooden buildings and the quantities of framings with walls or bracings therein.[8] The total number of houses investigated was about 70, including one and two story buildings which were situated in and around Fukui City. They stand on deep Alluvium deposit and suffered severe shocks, especially in N-S direction, which caused various degrees of damage including slight and severe damage and complete destruction.

The results of investigation are shown in Figs. 7 and 8. These figures indicate the correlation between the features of damage to wooden buildings and the load carried by unit length of frame with wall therein, classified according to the kind of wall and its location in story.

In making those figures, the frames having diagonal bracings or walls with openings are calculated in terms of full wall panels; the rates follow the results obtained from static tests which are shown in Table 3. Although these figures represent considerable scattering, they lead to the following remarks:

1) The degree of damage increases as the load carried by unit frame length increases.

2) "Okabe" is expected to resist larger earthquake force than "Shinkabe".

3) The lateral force resistance of frame with wall located on the first floor of a two story building seems to be greater than that in one story building.

4) In the Fukui Plain, shocks in N-S direction might be stronger than those in E-W direction.

5) If the load carried by unit length of frame with "Shinkabe" in completely destroyed houses is assumed to be 300 kg/m or 540 kg/6 ft, the acceleration imposed on those houses might be in the range of 0.4 g to 0.6 g which can be estimated from Table 2 or Fig. 19.

4 STATIC AND DYNAMIC TESTS ON FRAMES WITH WALLS OR BRACINGS

It is considered that the principal lateral force resistance of wooden buildings comes from the frames with walls or bracings. Therefore several tests[9] [10] [11] using models have been carried out to study the static and dynamic behavior of those resisting elements, but no experiment on full size frame with vertical loading has been made so far. The purpose of this test[12] is to reveal the aseismic properties of the resisting frames which are commonly used in this country, using full-scale ones under real condition.

Frames with Walls or Bracings Tested. In Table 1 are listed all frames tested. Each pair of those was erected on a concrete floor and the weight corresponding to average vertical loading was imposed on the frames as shown in Figs. 9, 10, 11 and 12. The estimated vertical loads are 720 kg for 6 ft length frame, 1000 kg for 12 ft and 480 kg for 3 ft, respectively.

Static Tests. Horizontal racking load was alternatively applied to the beam connecting the tops of two frames erected in parallel up to the failure. Some of the results showing the relationship between horizontal load and distortion of the top of frame are illustrated in Figs. 13, 14, 15, 16, 17 and 18. From the test results, the relation between load and distortion for each frame is shown in Fig. 19. The correlation between dissipated work during one cycle loading and distortion of each frame is indicated in Fig. 20. The results of the static tests are summarized below.

1) The cracks of the wall finishings develop, in general, when the top displacements of the frames reach about 4 cm and then the finishings gradually fall off. A diagonal bracing with narrow section suffers buckling failure whereas one with wide section makes compressive failure of adjoining lumbers at both ends.

2) The strength of a frame with wall above the picture rail is about one-fourth that with full wall panel.

3) The strength of a frame without any bracing is very small and cannot be expected to resist earthquake force.

4) The strength and rigidity of "Okabe" are larger than those of "Shinkabe", and the superiority is based on the horizontal wood sheathing with nail joints.

5) The effectiveness of diagonal bracings is proved. Care should be taken to prevent their buckling failure as well as the compressive crushing of their adjoining lumbers.

6) The rigidity of a frame with wall panel is roughly proportional to the square of its length, i.e. the distance between two columns composing the frame.

7) As for the dissipated work during one cycle loading, the frames with diagonal bracings represent large value in comparison with other ones, and, in general, the energy to be absorbed by "Okabe" is larger than that by "Shinkabe".

Dynamic Tests.   Free vibration tests of the frames were carried out at various loading stages during the static tests. Under alternating loads in the plus and in the minus directions, how the natural periods of the frames become longer with the increase of the amplitudes is shown in Fig. 21. Furthermore the relationship between equivalent fraction of critical damping and ratio of load to its maximum value is shown in Fig. 22.   The results obtained from vibration tests are summarized below.

1) In general the natural periods of "Okabe" are less than those of "Shikabe".

2) The natural periods of frames with walls or bracings increase with the amplitudes of vibration. In case of "Okabe" the ratio of increase reaches twice to three times the periods for smaller amplitudes but it is not so remarkable in "Shinkabe". The natural periods of frames having bracings do not vary remarkably through the tests.

3) The equivalent fractions of critical damping of "Shinkabe" are in the range of 0.25 to 0.1, decreasing with increase of amplitudes due to the falling of infilled mud-plaster. On the other hand, the equivalent fractions of critical damping of "Okabe" increase with amplitudes due to the properties of nail joints for wood sheathings, ranging 0.1 to 0.3. Those values of frames with bracings are generally small and less than 0.1.

5   CONSIDERATIONS AND PROPOSAL OF ASEISMIC DESIGN

Permissible loads.   The horizontal loads to be allowed for frames should be dertermined by considering the features of damage permissible which is considered as the falling of finishings after cracks develop. These loads, from the results of static tests, correspond to the displacements of 4 cm at the tops of frames, i.e. the angular deformation of 1/75 radian. This permissible load for each frame will be obtained from Fig. 19 as shown in Table 2, and the equivalent value per unit length of frame is shown in Table 3. Furthermore, the relative rigidity of each

frame is also shown in the last column of the same Table.

Story-rigidity in Two Story Buildings. The following load per unit floor area including both dead and live load is considered, on an average, to participate in the vibration of buildings during earthquakes:

	Light-roofed	Heavy-roofed
Roof level	100 kg/m^2	150 kg/m^2
2nd floor of two story bldg.	180	180

Now consider a two-mass-system which has mass M and rigidity k_2 for the second story, and 1.5 M and k_1 for the first story, respectively. Assuming $2\pi\sqrt{M/k_2}$ = 0.3 sec and each fration of critical damping with respect to first or second mode is equal to 0.1, the elastic response of the system can be obtained from the velocity spectrum of an earthquake. If the displacement of each story is calculated by using average velocity spectrum curves[13], and the ratio of the story rigidities which produces equal relative distortions are sought, the relation k_1/k_2 = 1.75 will be obtained. This means that in the first floor of a two story building larger quantity of frames should be required than in the second story but it results in larger load carried by unit frame length in the first story. This is not inconsistent with observed results shown in Figs. 7 and 8.

A Proposal on Aseismic Design. From a viewpoint that earthquake force resistance of a wooden building comes from frames with wall panels or bracings and based on the principle to restrict the damage within a small limit, a proposal on earthquake resistant design of wooden buildings is presented here. In this case the coupling effect of the structure and ground should be considered.

In the 1948 Fukui Earthquake, the city situated on thick Alluvium deposit and near epicenter suffered severe shocks, the maximum horizontal acceleration of which was estimated to be over 0.4 g. In the great earth-quake of Tokyo in 1923, the maximum horizontal acceleration estimated was 0.2 to 0.25 g at the downtown area on the Alluviam and 0.1 to 0.15 g at the uptown area on the Diluvium. Wooden buildings standing on the Tertiary Bed Rock suffered least damage, generally speaking, as had been observed in the past earthquakes. If the ground condition is classified into four kinds as shown in Table 4, the range of each predominant period is consider-ed to be as shown in the same Table.

The response of wooden buildings to earthquakes depends on the vibra-tional characteristics of the ground and dynamical properties of the structure. To avoid the quasi-resonance during earthquakes, the natural period of a building with its elongated value should be taken not to coincide with the predominant period of the ground. On the other hand it has been proved that the natural period of a wooden building is mostly governed by the quantity of frames with walls or bracings.[14] [15]

In the Fukui Earthquake, it is seen, from Fig. 7, that the maximum load carried by unit length of frame which restricted the damage to slight inclination of the building would be 300 kg/m, i.e. about 30 cm/m^2 for a one story building with light roof and about 45 cm/m^2 for one with heavy roof. Based on those values and considerations indicated above, the following method of aseismic design of wooden buildings is proposed:

1) In wooden buildings the quantity of frames with walls or bracings (the length per unit floor area) indicated in item 2) should be provided in a well balance in each principal direction to withstand earthquake force. In this case the effective length of each frame should be calculated by multiplying its real length by the rate shown in Table 3.

The required quantities of frames in wooden buildings are as follows:

a) For buildings on Kind 4 ground; Alluvium, 30 meters or more in thickness, made-land or swampy ground,

	Light-roofed, in cm/m^2	Heavy-roofed, in cm/m^2
One story bldg. or 2nd floor of two story bldg.	30	45
1st floor of two story bldg.	50	75

b) For buildings on Kind 3 ground; Alluvium, less than 30 meters in thickness,
    4/5 times the values shown in the Table

c) For buildings on Kind 2 ground; Diluvium,
    2/5 times the values shown in the Table

d) For buildings on Kind 1 ground; Tertiary or older strata,
    1.5/5 times the values shown in the Table

The natural periods of wooden buildings which have the quantities of frames with walls or bracings prescribed above are considered to be in the range shown in Table 4, where the period during severe earthquakes is assumed to be twice as long. Considering the quasi-resonace effect which amplifies the ground acceleration[16], the earthquake forces to be imposed on structures are generally considered to be in proportional to the quantities of frames prescribed above.

Comment on the Japanese Building Code Requirements. The principle and method of the present Japanese Code Requirements[17] coincide with those presented by the authors in the preceding paragraphs. The required quantity of framings in the Code is generally the same as that proposed by the authors for wooden buildings on firm ground.

However, value required by the Code for buildings standing on soft ground are about 2/3 to 1/2 of those proposed; the former values should be increased up to these values if the damage is to be restricted within a small limit. With respect to the strength rate of each frame, there are some over-estimations on the frames with bracings in the Code, as can be seen in Table 3.

6 CONCLUDING REMARKS

In the present paper, the authors investigated earthquake damage to wooden buildings in connection with the ground conditions and quantities of frames with walls or bracings therein, from which principal lateral force resistance came. On the other hand, static and dynamic tests on such frames of full size were carried out to reveal their seismic behavior. The results thus obtained are as follows:

1) The ground conditions under wooden buildings influence the earthquake damage to a great extent and the softer is the subsoil, the greater is the damage to the buildings. When the thickness of Alluvium under buildings is 30 m or more, significant damage is observed.

2) The earthquake damage to wooden buildings is related with the quantity of frames with walls or bracings provided and the damage increases with the decrease of quantity of frames.

3) From the test results, it is proposed that the angular distortions of frames against earthquake forces be permitted within 1/75 radian, at which they represent the resisting force indicated in Table 3. Furthermore, the prolongation of the natural periods of resisting elements and the variations of damping and absorbing energy in relation with the increase of amplitudes were obtained for each frame as shown in Figs. 20, 21 and 22.

4) The tests proved that the frames with wall panels having horizontal sheathings or diagonal bracings are highly effective in comparison with the frames with Japanese walls of mud plaster over bamboo lath.

Thus, based on the results of investigations and experiments, an aseismic design method for wooden buildings which requires certain quantities of frames with walls or bracings is proposed. The required values vary according to the roof weight, story where the frames are located and conditions of ground on which buildings stand. Also, some comment is given concerning the aseismic regulations on wooden buildings prescribed in the present Japanese Building Code.

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NOMENCLATURE

R : Distance from the epicenter in kilo-meters

x : Thickness of Alluvial deposit in meters

y : Damage rate in per cent

P : Horizontal racking load on frame in kg

X : Horizontal distortion of frame in cm

r : Ratio of horizontal racking load to its maximum value

W : Dissipated work in frame during one cycle loading in t.mm

T : Period of frame or building in sec

he : Equivalent fraction of critical damping
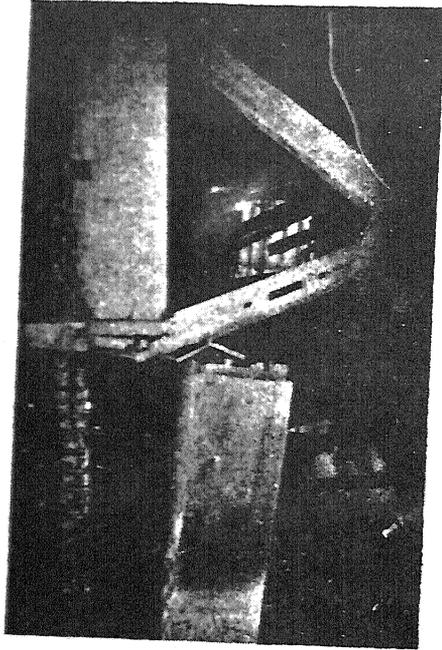
FIG. 2

TYPICAL FEATURE OF DAMAGE TO FRAME JOINT
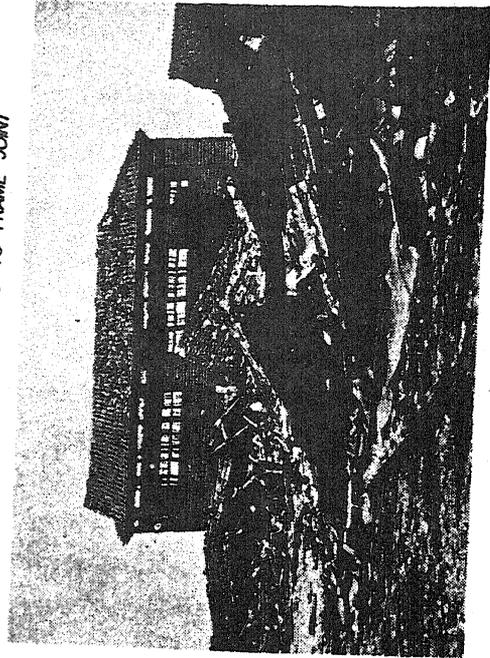
FIG. 4

TYPICAL FEATURE OF DAMAGE TO WOODEN BUILDING
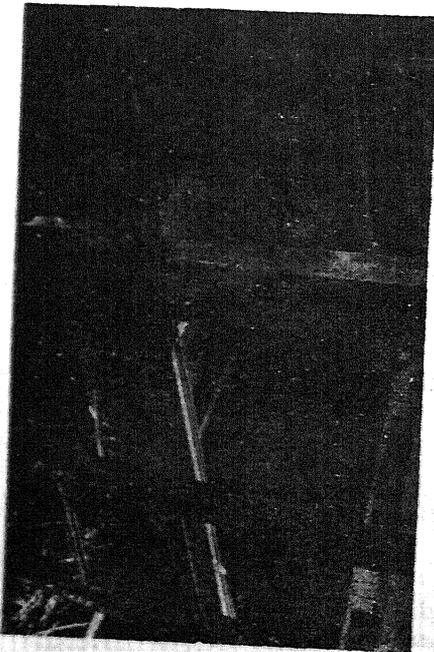
FIG. 1

TYPICAL FEATURE OF DAMAGE TO FRAME JOINT
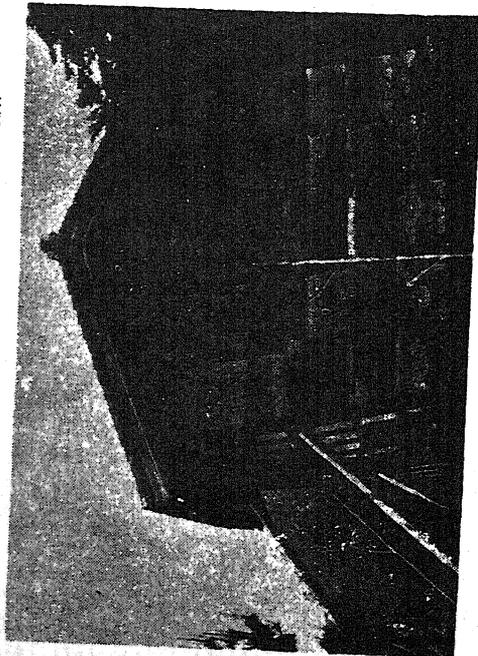
FIG. 3

TYPICAL FEATURE OF DAMAGE TO WOODEN BUILDING
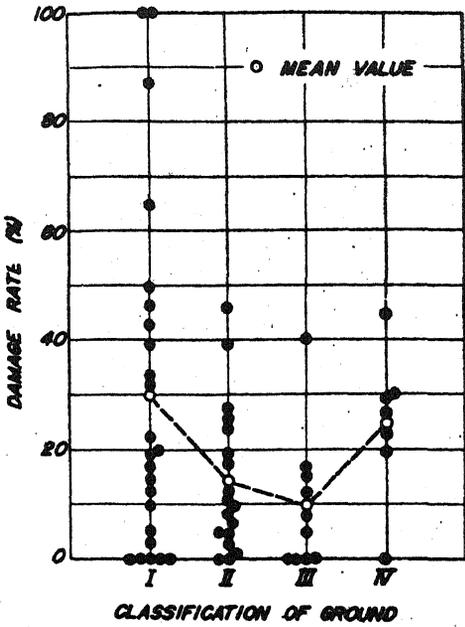
FIG. 5
DAMAGE RATES OF WOODEN BUILDINGS
ON DIFFERENT GROUNDS
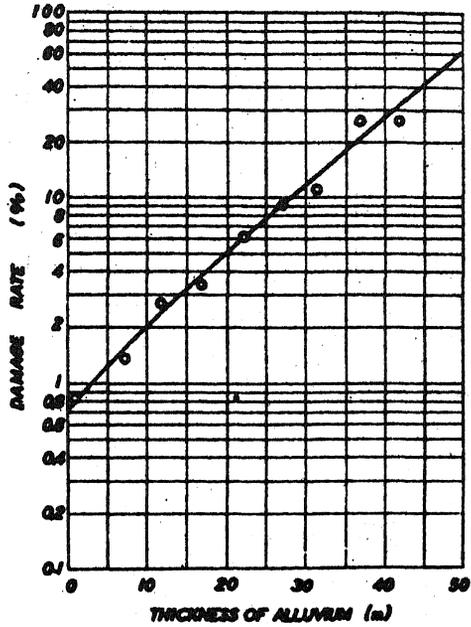
FIG. 6
RELATIONSHIP BETWEEN DAMAGE RATE
AND THICKNESS OF ALLUVIAL DEPOSIT
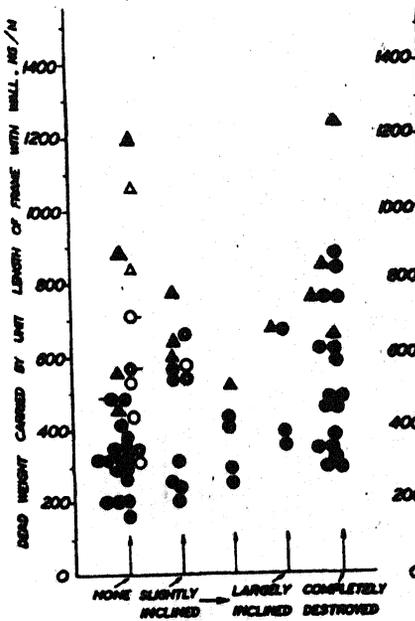
FIG. 7
RELATION BETWEEN DAMAGE AND DEAD
WEIGHT CARRIED BY UNIT WALL LENGTH,
IN N-S DIRECTION
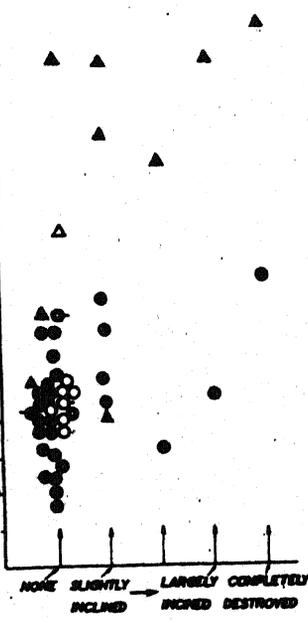
FIG. 8
RELATION BETWEEN DAMAGE AND DEAD
WEIGHT CARRIED BY UNIT WALL LENGTH,
IN E-W DIRECTION
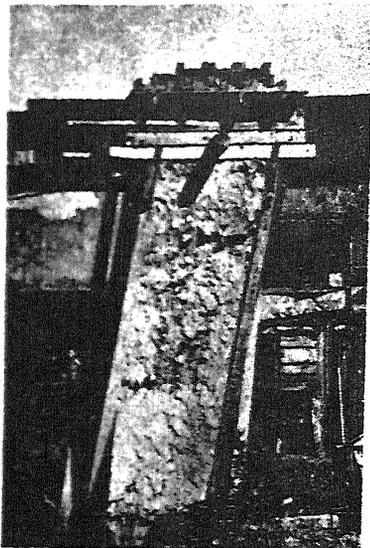
FIG. 9

TEST ON WALL PANEL SB 3
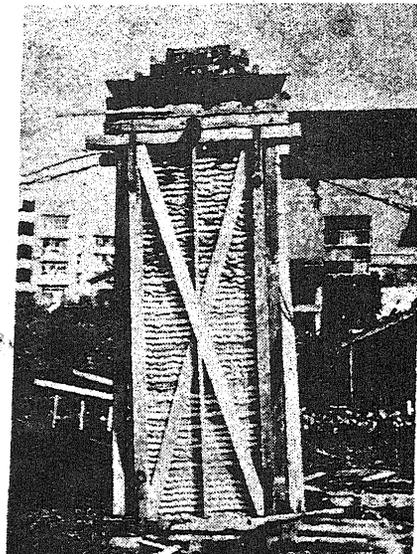
FIG. 10

TEST ON WALL PANEL OD 3
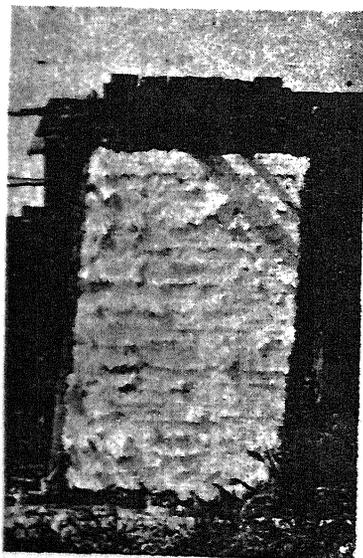
FIG. 11

TEST ON WALL PANEL OB 6
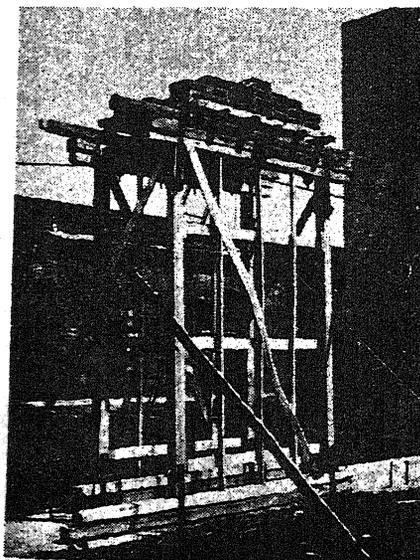
FIG. 12

TEST ON FRAME FWB 3
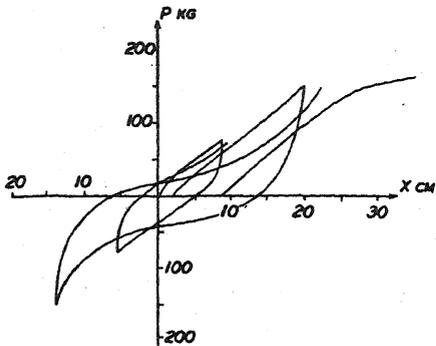
FIG. 13
LOAD – DISTORTION CURVE
FOR WALL PANEL SA 6
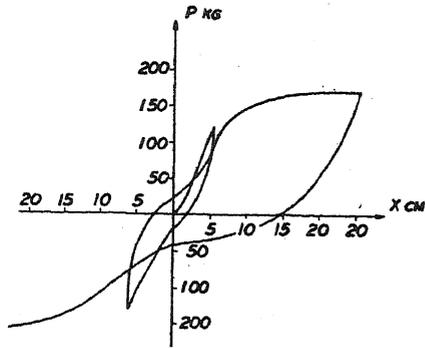
FIG. 14
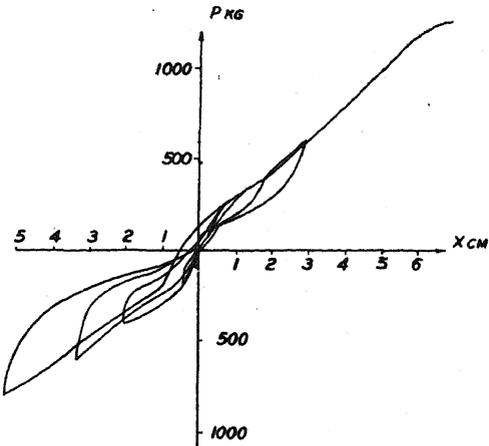LOAD – DISTORTION CURVE
FOR WALL PANEL SB 6

FIG. 15
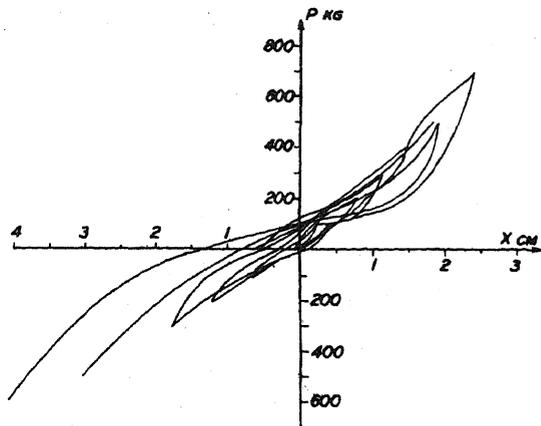LOAD–DISTORTION CURVE
FOR WALL PANEL SD 6

FIG. 16
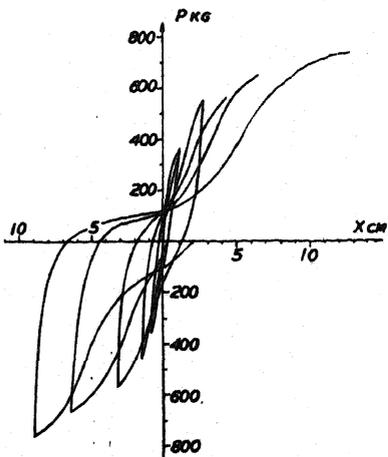LOAD–DISTORTION CURVE
FOR WALL PANEL OB 6

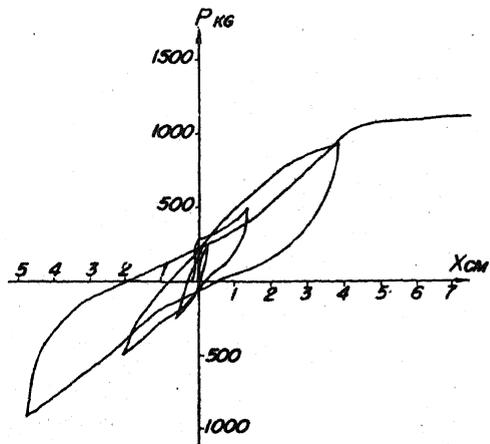FIG. 17
LOAD – DISTORTION CURVE
FOR WALL PANEL OC12

FIG. 18
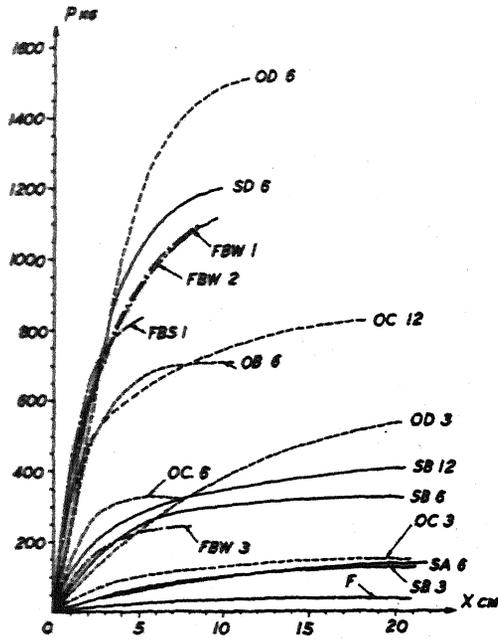LOAD–DISTORTION CURVE
FOR FRAME FBW2

FIG. 19

RELATIONSHIP BETWEEN LOAD
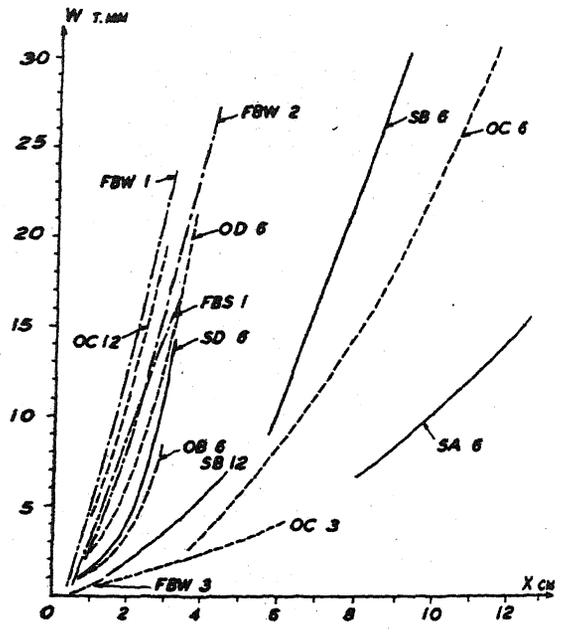AND DISTORTION

FIG. 20

RELATIONSHIP BETWEEN DISSIPATED
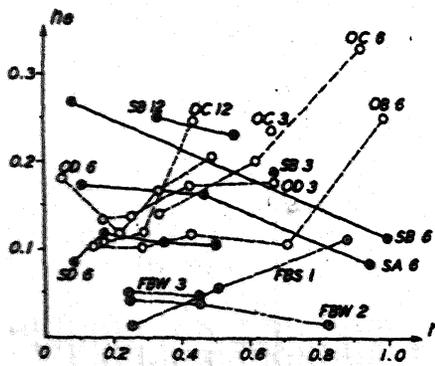WORK AND DISTORTION

FIG. 22

RELATIONSHIP BETWEEN FRACTION
OF CRITICAL DAMPING AND RATIO
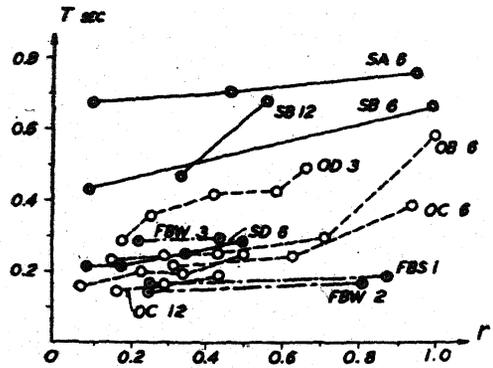OF LOAD TO ITS MAXIMUM VALUE

FIG. 21

RELATIONSHIP BETWEEN NATURAL
PERIOD AND RATIO OF LOAD
TO ITS MAXIMUM VALUE

Table 1

Symbol of frame	Length of frame, in ft	Wall area	Wall finishing		Bracing
			interior	exterior	
SA6	6	above picture rail	mud-plaster	mud-plaster	none
SB6	6	full	mud-plaster	mud-plaster	none
SD6	6	full	mud-plaster	Japanese sheathing	X, 1/5 pillar
OB6	6	full	plaster on wood sheathing	plaster on wood sheathing	none
OC6	6	full	plaster on wood sheathing	wood sheathing	none
OD6	6	full	plaster on wood sheathing	wood sheathing	X, 1/3 of pillar
FBW1	6	frame only	—	—	X, equal in size to pillar
FBW2	6	frame only	—	—	X, 1/2 of pillar
FBW3	6	frame only	—	—	Diagonal, 1/5 of pillar
FBS1	6	frame only	—	—	X, ϕ16 steel bar
F6	6	frame only	—	—	none
SB3	3	full	mud-plaster	mud-plaster	none
OC3	3	full	plaster on wood sheathing	wood sheathing	none
OD3	3	full	plaster on wood sheathing	wood sheathing	X, 1/3 of pillar
SB12	12	full	mud-plaster	mud-plaster	none
OC12	12	full	plaster on wood sheathing	wood sheathing	none

Table 2

Symbol of frame	Length of frame, in ft	Load at 1/75 rad. distortion, in kg	Max. load, in kg
SA6	6	50	160
SB6	6	220	330
SD6	6	920	1200
OB6	6	630	700
OC6	6	320	330
OD6	6	1020	1800
FBW1	6	850	1150
FBW2	6	850	1150
FBW3	6	210	230
FBS1	6	800	800
F6	6	35	40
SB3	3	60	150
OC3	3	90	150
OD3	3	190	600
SB12	12	260	450
OC12	12	600	800

Table 4

Kind of ground	Predominant period of ground, in sec	Natural period of bldg., in sec	
		one story	two story
Kind 1: Tertiary or older strata	0.1 – 0.2	0.35 – 0.7	0.45 – 0.9
Kind 2: Diluvial	0.2 – 0.4	0.3 – 0.6	0.4 – 0.8
Kind 3: Alluvial	0.4 – 0.7	0.2 – 0.4	0.3 – 0.6
Kind 4: Alluvial, 30 meters or more in thickness, made-land	0.4 – 0.6 – 1.0	0.18 – 0.35	0.25 – 0.5

Table 3

| | Kind of framing | Load at 1/75 rad. distortion, in kg | | Rate |
		per 6' length	per 1 m length	
(1)	Frame with "Shinkabe"	220	120	1 (1)
	Frame with "Okabe", sheathed on one side	320	180	1.5 (1.5)
	Frame with "Okabe" sheathed on both sides	630	350	3 (3)
(2)	Frame with a lumber bracing of 1/5 of pillar	210	115	1 (1)
	Frame with X lumber bracing of 1/3 of pillar	700	390	3 (3)
	Frame with X lumber bracings of 1/2 of pillar	850	470	4 (6)
	Frame with X lumber bracings equal in size to pillar	850	470	4 (6)
	Frame with X steel bar bracings having diameter of 16 mm	800	445	4 (6)
(3)	Frame with walls in item (1) and bracings in item (2) combined			Sum of values of the items concerned

Note: Rates in parentheses show the values prescribed in the
present Building Code Requirements.