ANALYSIS OF THE HACHINOHE LIBRARY DAMAGED BY '68 TOKACHI-OKI EARTHQUAKE

by

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1. Objective

The objective of this study was to analyze a reinforced concrete single story building, the Hachinohe library, damaged by '68 Tokachi-oki earthquake and to make clear the cause of the damage. As the first step of the study, the damage of the building was investigated precisely which has been reported in Reference 1.

Elastic and plastic behavior of the building during the earthquake was analyzed and the cause of the damage was examined in this paper. As a matter of convenience for comparing the results of the analysis with the damage, a part of the investigation of the damage was also transcribed.

2. Out-line of the investigation of damage

2.1 Structure

This building was constructed at Hachinohe city as a city library in 1961. As shown in Fig. 1, the structure consisted of reinforced concrete frames and shear walls located eccentrically. Dimensions of the structure were as follows:

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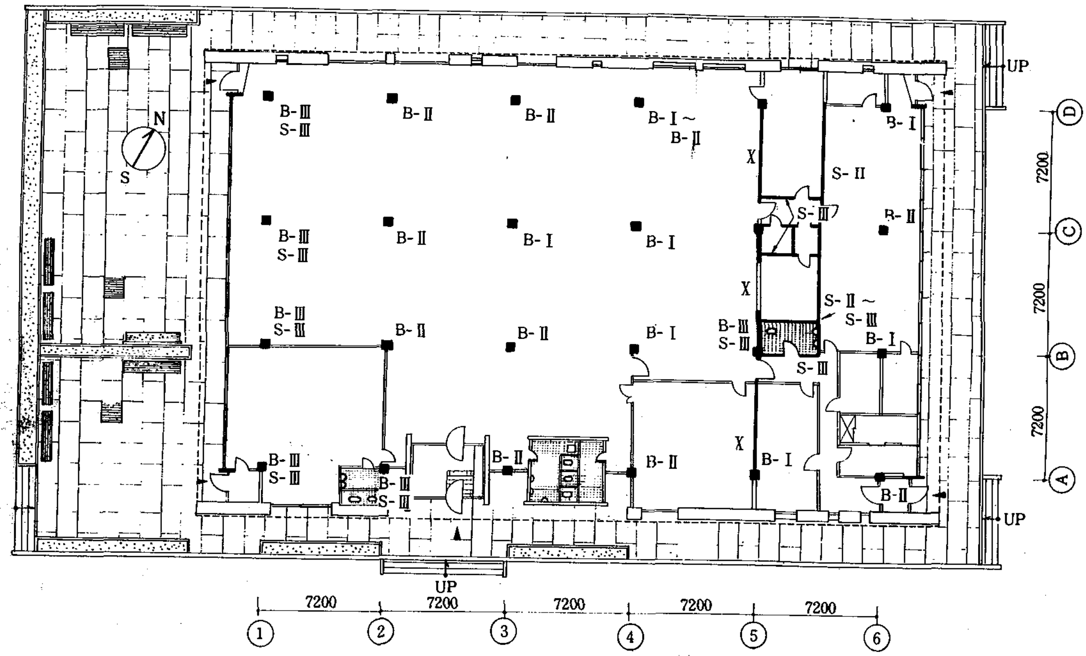

Fig. 1 Plan and degree of damage

span	: three spans of 7.2 m transversely and five spans of 7.2 m longitudinally.
story height	: 4.2 m from the surface of the footing beam to the top of roof.
beam	: 30 cm x (72 ~ 80) cm in cross section.
column	: 45 cm x 45 cm in cross section.
slab	: 15 cm in depth at the end and 12 cm at the center.
footing beam	: 35 cm x 80 cm in cross section.
footing	: individual footing.
wall	: 12 cm in thickness.
reinforcement	: round bar of 19 mm or 22 mm diameter with the specified yield strength of 2,400 kg per sq cm. Details are shown in Fig. 2.

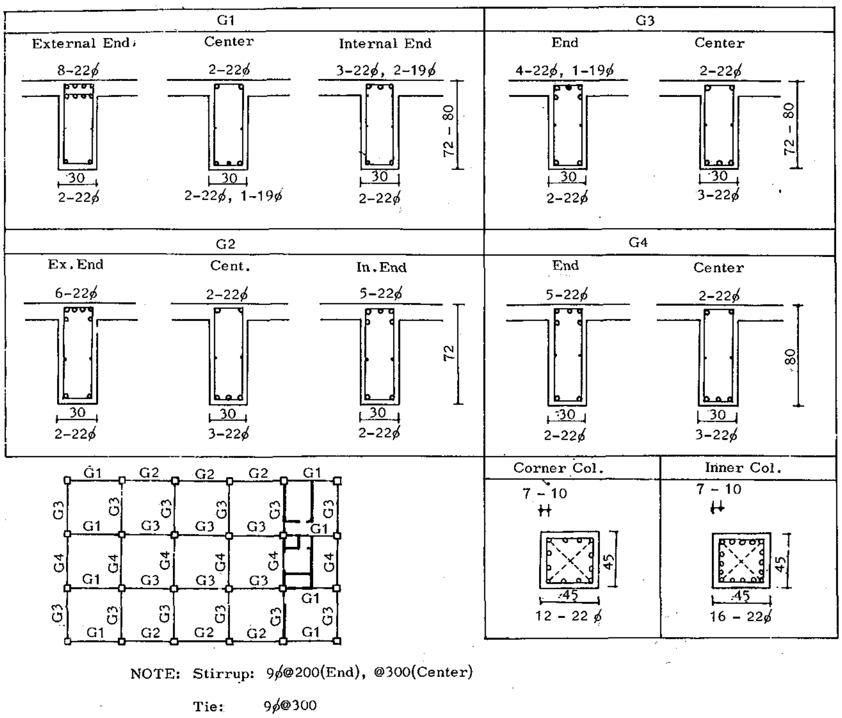

Fig. 2 Cross section and reinforcement

2.2 Damage

During the '68 Tokachi-oki earthquake, this building was heavily damaged. Signitures in Fig. 1 show the degree of damage of columns and walls in accordance with the standard shown in Table 1.

Among the damage of this building, the damage of columns was significant. Columns of the west frames were more seriously damaged than those of the east zone. Concrete was crushed and spalled at the top and/or at the bottom of the columns and the reinforcements were exposed as shown in Fig. 3. Large cross diagonal cracks were observed in the walls located in E-W direction. Among two walls of N-S direction, the east wall had great diagonal cracks. No crack could be found in beams and slabs.

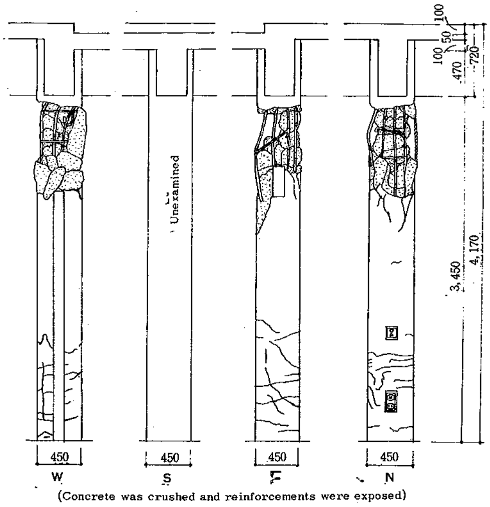

Fig. 3 Damage of columns (C_{1A})

Type of failure	Sign.	Degree of damage
Undamaged	0	Undamaged
Bending failure	B-I	cracking stage, hair crack was found.
	B-II	yield stage, tensile reinforcements took yielding
	B-III	or compressive concrete was crushed.
Shearing failure	S-I	ultimate stage, concrete was crushed completely and reinforcements were exposed.
	S-II	cracking stage, hair crack was found.
	S-III	yield stage, width of crack was over 1 mm.

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Table. 1 Signitures of degree of damage

Type of failure	Sign.	Degree of damage
Undamaged	0	Undamaged
Bending failure	B-I	cracking stage, hair crack was found.
	B-II	yield stage, tensile reinforcements took yielding or compressive concrete was crushed.
	B-III	ultimate stage, concrete was crushed completely and reinforcements were exposed.
Shearing failure	S-I	cracking stage, hair crack was found.
	S-II	yield stage, width of crack was over 1 mm.
	S-III	ultimate stage, large slipping or crushing was found.

2.3 Permanent displacement and Micro-tremor

Measured permanent displacements after the earthquake are shown in Fig. 4. Relative displacements between the top and the bottom of the columns were measured. From the pattern of the displacement diagram, it is supposed that this building had torsional vibration during the earthquake. Frame No. 1 which was located at the far west end left 2.5 cm of permanent displacement to the north and the No. 6 frame was deformed 1 cm in the opposite direction.

Micro-tremor was observed on the roof to examine the vibrational characteristics after the earthquake. The period of 0.55 sec. predominated to the N-S direction and its mode was the torsional type, when the center of the rotation was located near the walls. In the case of the E-W direction, the period of 0.42 sec. predominated, when the mode was the type of translation.

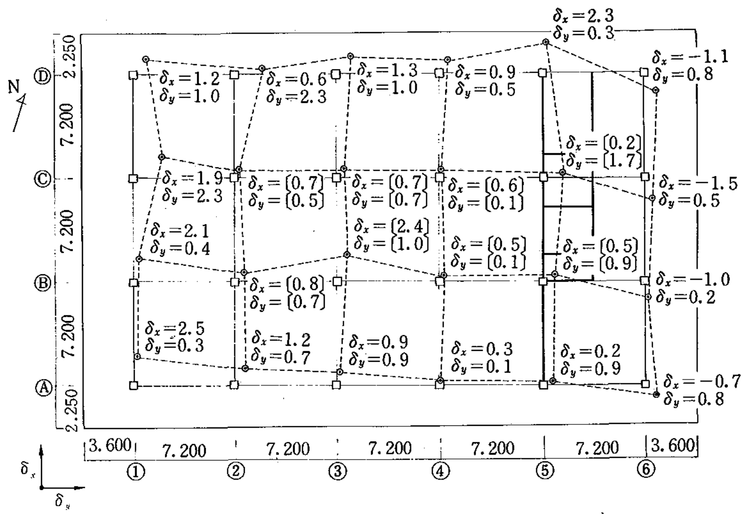


Fig. 4 Permanent displacement

2.4 Strength of concrete

The compressive strength of the concrete was estimated to be 180 kg per sq cm from the compressive test of four cylinders which were cut off by core boring from the columns. From the Schmidt hammer test, it was estimated to be 380 kg per sq cm. Considering these test results, the concrete strength of this building would not be less than the specified strength of 180 kg per sq cm.

3. Analysis

This building was designed in accordance with the building laws of Japan and the building code of A.I.J., so that the earthquake force was considered statically in the concept of seismic coefficient. The adopted value of the seismic coefficient was 0.2 for the frames, when the shear walls were not considered in computation. The seismic coefficient method has been adopted as a convenient method of the aseismic design for standard scale buildings. The building designed by this method had been considered not to be damaged slightly but to be cracked to some extent, or collapsed partially, in a severe earthquake. However, in the case of the Hachinohe library, the degree of the damage exceeded our expectation.

The analysis has been carried out so as to make clear the static and dynamic properties of this building, to examine the cause of damage and to offer useful data for the future aseismic design of reinforced concrete buildings.

3.1 Estimation of horizontal stiffness

As the estimation of horizontal stiffness affects considerably the results of the analysis, the stiffness of frames and walls was calculated respectively by three ways and nine model structures for analysis were assumed.

a) Stiffness of frames - - - - Methods of calculation and calculated values are shown in Tables. 2 and 3, respectively. Frame stiffnesses I and II are elastic stiffnesses and frame stiffness III is stiffness at the yield stage. Elastic stiffness was calculated by "the slope deflection method". The rigid zone of beam-column connection was not considered in the case of Frame stiffness I, while it was considered in the case of II.

Stiffness at the yield stage is the "secant modulus at the yield stage of the load-displacement relationship of the frame" shown in Fig. 5. As these frames were "type of column yielding", the stiffness at the yield stage was calculated approximately by the following method.

(stiffness at yield stage) = (elastic stiffness) x (reduction factor of rigidity of column section at yield stage)

where (the reduction factor) = (the secant modulus at yield stage)/(elastic rigidity) of the moment-curvature relationship of the column section.

b) Stiffness of walls - - - -
Wall stiffnesses I, II, and III correspond to Frame stiffnesses I, II and III, respectively. Bending and shear deformation were calculated by "the beam theory" in the case of wall stiffness I. Shear deformation and effect of opening were considered in the case of Wall stiffness II. The reduction factor was introduced in considering the opening effect of the walls, which was calculated as follows:

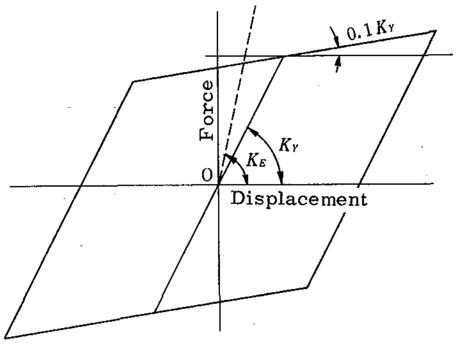


Fig. 5 Load-displacement relationship

(reduction factor) = (1.0 - (ratio of opening))
where (ratio of opening) = square root of (area of opening/ whole area of wall)

Wall III was adopted to analyze the plastic behavior after cracking. It was calculated by multiplying the elastic stiffness by the plastic reduction factor. The plastic reduction factor was assumed to correspond to the degree of damage as shown in Table 3. The reduction factor of 0.2 corresponds to the yield stage of the wall, when the shear deformation angle is assumed to be 3×10^{-3}.[2)]

Table. 2 Calculation of stiffness

Frame stiffness I	Elastic	Slope deflection method.
Frame stiffness II	Elastic	Slope deflection method, considering rigid zone
Frame stiffness III	Plastic	Frame stiffness I x Reduction factor at yield stage.
Wall stiffness I	Elastic	Beam theory, considering bending and shearing deformations.
Wall stiffness II	Elastic	Beam theory, considering shearing deformation and effect of opening.
Wall stiffness III	Plastic	Wall stiffness II x Reduction factor at yield stage.

Table. 3 Calculated stiffnesses (t/cm)

	Direction	Frame I or Wall I	Frame II or Wall II	Frame III or Wall III: Reduction factor	Frame III or Wall III: Stiffness	Frame III or Wall III: Average Stiffness*
Column 1	NS EW	7.94 7.86	12.2 11.7	0.46 "	3.65 3.62	3.64
2	NS EW	9.93 8.18	16.1 12.5	0.51 "	5.06 4.17	
3	NS EW	7.94 9.88	12.2 15.8	0.51 "	4.04 5.04	4.54
4	NS EW	7.94 9.56	12.5 15.1	0.51 "	4.04 4.88	
5	NS EW	9.93 10.1	16.1 17.0	0.51 "	5.06 5.15	5.06
6	NS EW	9.93 9.75	16.1 16.1	0.51 "	5.06 4.97	
Wall 1	NS	4,531	2,257	1.0	2,257	
2	NS	1,745	2,009	1.0	2,009	
3	EW	421.7	372	0.2	74	
4	EW	235.4	610	0.2	122	
5	EW	235.4	397	0.2	80	
6	EW	421.7	285	0.2	57	
7	EW	-	397	0.2	80	

* : These values were used as the stiffnesses of the Frame III.

3.2 Properties of model structure

a) Shear distribution of frames in static analysis - - - - Shear force distribution of the frames was examined by the usual static method. The adopted equation is as follows:

$$Q_x = \left\{ 1 + \frac{\Sigma C_{xi} \cdot e_y \cdot (e_y - y)}{J} \right\} \cdot \frac{C_x}{\Sigma C_{xi}} \cdot Q \quad \cdots\cdots\cdots\cdots (1)$$

where Q_x : shear force of frame under consideration to X axis.

Q : total shear force applied to the center of gravity.

C_x : stiffness of the frame under consideration.

C_{xi} : stiffness of the i-th frame.

J : Moment of inertia of stiffness with respect to the center of the stiffness.

e_y : distance between the center of stiffness and the center of gravity.

y : distance between the frame under consideration and the center of gravity.

The center of stiffness and the shear force distribution coefficients ($q_x = Q_x/Q$) are shown in Table 4. The center of stiffness was located about 12 meters east from the center of gravity in model structure B and C. When a horizontal force is applied to the center of gravity, the shear force of the frame is strongest at the far west frame considering the effect of the walls. However, it is remarkable that the shear force at the far west frame is less than that of each frame without walls, even if we consider the torsional effect of the walls located eccentrically.

b) Ultimate strength ---- Frames The ratio of the ultimate strength to the whole weight of the building was about 0.3, when the relative displacement of the roof to the base was about 2 cm. The ultimate strength means the sum of the shear force of the columns when all columns reach the yield stage by bending both at the top and at the bottom. The yield strength of the reinforcement and the compressive strength of the concrete were assumed to be 2,400 kg per sq cm and 200 kg per sq cm, respectively.

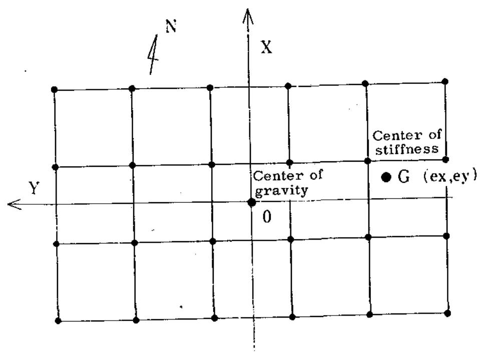


Fig. 6 Coordinate

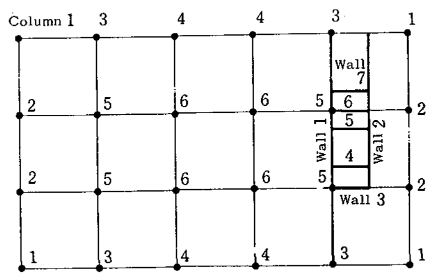


Fig. 7 Signitures of columns and walls

Table. 4 Model structures and their properties

Signiture of model	Stiffness		e_y (m)		Shear force coefficient of No. 1 Frame	Natural period (sec.)		
	Frame	Wall	NS	EW		Fundamental	Second	Third
A-1	I	---	0	0	0.167	0.37 (trans. NS)	0.37 (trans. EW)	---
A-2	II	---	0	0	0.167	0.29 (trans. NS)	0.29 (trans. EW)	---
A-3	III	---	0	0	0.167	0.52 (trans. NS)	0.52 (trans. EW)	---
C-4	I	I	0	11.4	0.120	0.32 (rotation)	0.14 (trans. EW)	0.053 (trans. NS)
C-5	II	I	0	11.2	0.134	0.27 (rotation)	0.13 (trans. EW)	0.053 (trans. NS)
C-6	I	II	0.63	11.9	0.117	0.31 (rotation)	0.14 (trans. EW)	0.062 (trans. NS)
C-7	II	II	0.60	11.6	0.133	0.26 (rotation)	0.11 (trans. EW)	0.062 (trans. NS)
C-8	III	III	0.57	12.2	0.108	0.44 (rotation)	0.23 (trans. EW)	0.063 (trans. NS)
B-9	III	III(NS) zero(EW)	0	12.1	0.118	0.52 (trans. EW)	0.46 (rotation)	0.063 (trans. NS)

e_y: Distance between the center of gravity and the center of stiffness.
trans. NS: Mode of translation to N-S direction.
trans. EW: Mode of translation to E-W direction.

<u>Frames with walls located eccentrically</u> - - - - Considering the torsional effect of the walls, the far west frame reached a yield stage when the seismic coefficient was 0.36 ~ 0.41.

c) Dynamic properties of model structure

As this building had a rigid roof construction, we adopted the three degrees of freedom system of vibration. According to the coordinate system shown in Fig. 6, the differential equation of free vibration is:

$$[M]\{\ddot{u}\}+[K]\{u\}=\{0\} \quad \cdots\cdots\cdots\cdots(2)$$

where,

$$[M]=\begin{bmatrix} m, & 0, & 0 \\ 0, & m, & 0 \\ 0, & 0, & I \end{bmatrix}$$

$$[K]=\begin{bmatrix} \Sigma C_{xi}, & 0 \quad , & e_y \cdot \Sigma C_{xi} \\ 0 \quad , & \Sigma C_{yi}, & e_x \cdot \Sigma C_{yi} \\ e_y \cdot \Sigma C_{xi}, & e_x \cdot \Sigma C_{yi}, & J+e_x^2 \Sigma C_{yi}+e_y^2 \Sigma C_{xi} \end{bmatrix}$$

$$\{\ddot{u}\}=\begin{Bmatrix} \ddot{x} \\ \ddot{y} \\ \ddot{\theta} \end{Bmatrix} \qquad \{u\}=\begin{Bmatrix} x \\ y \\ \theta \end{Bmatrix}$$

x, y : roof displacements with respect to the base in X and Y coordinate, respectively.

θ : rotational angle with respect to the center of gravity.

M : whole mass.

I : moment of inertia of mass with respect to the center of gravity.

C_{xi}, C_{yi}, : stiffnesses to X and Y coordinate, respectively.

e_x, e_y : distances of eccentricity to Y and X coordinate, respectively.

J : moment of inertia with respect to the center of stiffness.

Natural periods and modes computed by Eq. (2) are shown in Table 4. Computed elastic periods were 0.29 ~ 0.37 sec, in the case of frame structure without walls and 0.26 ~ 0.32 sec. in the case of considering the walls. These results show that this building was rather flexible as for a single story reinforced concrete building. Using the secant modulus at the yield stage, the natural periods were 0.52 sec. in the frame structure and 0.44 sec. in the frame and wall structures. Considering the effect of the walls, the fundamental mode was the rotational type in all cases except Model B.

Examples of mode are shown in Fig. 8, where they were normalized so that the sum of each mode is united. Taking into account the effect of the walls, the modes were very similar to Fig. 8 in all cases and their normalized first mode was nearly equal to that of the frame structure.

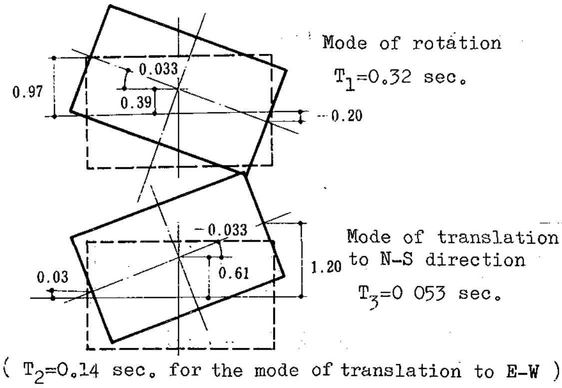


Fig. 8 Mode of vibration (Model C-4)

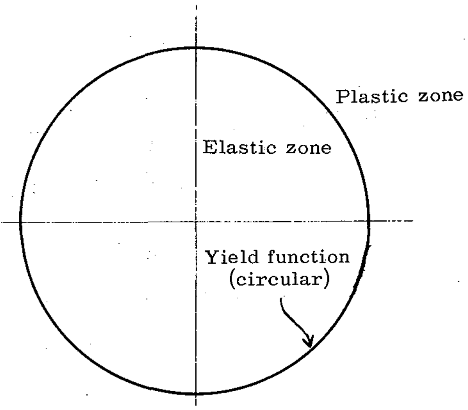


Fig. 9 Yield function

3.3 Response analysis

It has been supposed from the results of analysis mentioned above that this building would be apt to be affected by the ground motion having the frequency characteristics of near 10 c.p.s. in elasticity. Judging from the response spectrum computed from the acceleration records of the ground motion observed at Hachinohe harbor, which was 4 km away from this building, the effect of the ground motion on this building would be greater than the seismic coefficient of 0.2 for the design.

However, a yielding of structure does not always follow the collapse of a building if it has a ductile characteristic in a plastic zone. In this section, the linear and nonlinear response analyses were carried out varing the estimations of stiffness of the structure, damping constant and earthquake ground motion, because it was difficult to determine the only reasonable condition for analysis even if we neglected the interactions between building and ground soil.

a) The differential equation and method of computation

Using the same coordinates and notations as adopted in equation (2), the following equation was used.

$$[M]\{\ddot{u}\} + \frac{2h}{\omega_1}[K]\{u\} + [K]\{u\} = -[M]\{\ddot{u}_0\} \quad \cdots\cdots(3)$$

where,

$$\{\ddot{u}_0\} = \begin{Bmatrix} \ddot{x}_0 \\ \ddot{y}_0 \\ 0 \end{Bmatrix}$$

x, y : accelerations of ground motion to X- and Y - axes, respectively.

h : damping constant respect to the fundamental period.

ω_1: angular frequency of the fundamental mode

The equation was solved numerically by an electric computer using the linear acceleration method. The time interval of computation was 1/200 sec. where the stiffness and damping were assumed to be constant. In nonlinear analysis, the new plastic stiffness matrix was computed at each step according to the assumed load-deformation characteristics, whenever a part of columns and walls were in plastic zone.

b) Assumptions of the analysis

The load-deformation characteristic in nonlinear analysis was assumed for every direction shown in Fig. 5, where the stiffness after yield point was assumed to be 10 percent of that before yielding. The yield function in the column cross section was assumed to be circular as shown in Fig. 9. The elasto-plastic relationship was adopted for walls.

Since no ground motion was observed at the site of this building, three earthquake records were used for calculation. They were the acceleration record observed at Hachinohe harbor in the '68 Tokachi-oki earthquake (called HACHINOHE record in this paper), EL CENTRO record in 1940 (the maximum accelerations are 325 gals in N - S component and 220 gals in E - W component) and the modified TAFT record in 1952 (the maximum accelerations were modified to 384 gals in N - S component and 325 gals in E - W component). Besides these records, the modified HACHINOHE record of which the amplitude of the acceleration was multiplied by 1.5 was used.

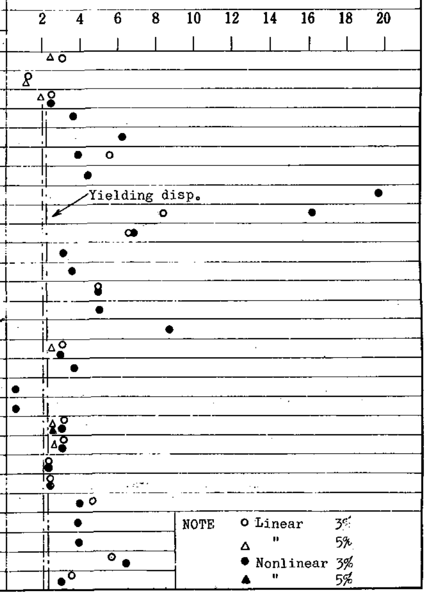
Viscous damping was adopted concluding the damping effect of ground soil. The damping constant was assumed to be 0.03 or 0.05 with respect to the fundamental angular frequency which was computed at each step.

c) Results of computation

i) Maximum response displacement

The maximum displacements shown in Fig. 10 included the results of both linear and nonlinear response analyses. The maximum displacement has the following tendency which can be classified according to the assumed model structures.

No.	Model structure					Period (sec)			Ground motion		
	Sign.	N-S		E-W		T_1	T_2	T_3	Designation.	Direction of struc.	
		Frame	Wall	Frame	Wall					N-S	E-W
1	A-1	Elastic	——	Elastic	——	0.37	0.37		HACHI	NS1.0*	——
2	A-2	〃	〃	〃	〃	0.29	0.29		〃	〃	——
3	A-3	Plastic	〃	Plastic	〃	0.52	0.52		〃	〃	——
4	〃	〃	〃	〃	〃	〃	〃		m HACHI	NS1.5	——
5	〃	〃	〃	〃	〃	〃	〃		〃	EW1.5	——
6	〃	〃	〃	〃	〃	〃	〃		HACHI	NS1.0	EW1.0
7	〃	〃	〃	〃	〃	〃	〃		m HACHI	NS1.5	〃
8	〃	〃	〃	〃	〃	〃	〃		〃	〃	EW1.5
9	〃	〃	〃	〃	〃	〃	〃		EL CENTRO	NS1.0	EW1.0
10	〃	〃	〃	〃	〃	〃	〃		m TAFT	〃	〃
11	B-9	〃	Elastic	〃	〃	0.52	0.46	0.063	HACHI	NS1.0	——
12	〃	〃	〃	〃	〃	〃	〃	〃	m HACHI	NS1.5	——
13	〃	〃	〃	〃	〃	〃	〃	〃	HACHI	NS1.0	EW1.0
14	〃	〃	〃	〃	〃	〃	〃	〃	m HACHI	NS1.5	〃
15	〃	〃	〃	〃	〃	〃	〃	〃	〃	〃	EW1.5
16	C-8	〃	〃	〃	Plastic	0.46	0.23	0.063	HACHI	NS1.0	——
17	〃	〃	〃	〃	〃	〃	〃	〃	m HACHI	NS1.5	——
18	〃	〃	〃	〃	〃	〃	〃	〃	HACHI	——	EW1.0
19	〃	〃	〃	〃	〃	〃	〃	〃	m HACHI	——	EW1.5
20	〃	〃	〃	〃	〃	〃	〃	〃	HACHI	NS1.0	EW1.0
21	〃	〃	〃	〃	〃	〃	〃	〃	〃	NS-1.0	〃
22	〃	〃	〃	〃	〃	〃	〃	〃	〃	EW1.0	NS1.0
23	〃	〃	〃	〃	〃	〃	〃	〃	〃	EW-1.0	〃
24	〃	〃	〃	〃	〃	〃	〃	〃	m HACHI	NS1.5	EW1.0
25	〃	〃	〃	〃	〃	〃	〃	〃	〃	〃	EW1.5
26	〃	〃	〃	〃	〃	〃	〃	〃	〃	NS-1.5	〃
27	〃	〃	〃	〃	〃	〃	〃	〃	EL CENTRO	NS1.0	EW1.0
28	〃	〃	〃	〃	〃	〃	〃	〃	m TAFT	〃	〃



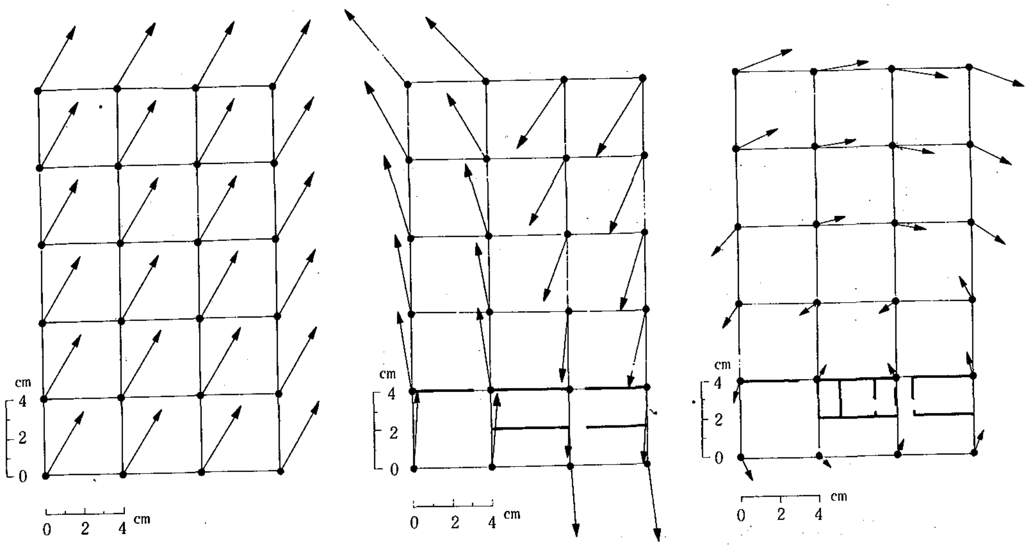
NOTES: * : Component and magnification. HACHI : HACHINOHE record.
m HACHI : Modified HACHINOHE record
m TAFT : Modified TAFT record

Fig. 10 Maximum response displacement

Model A (consists of frames both to N - S and E - W directions)

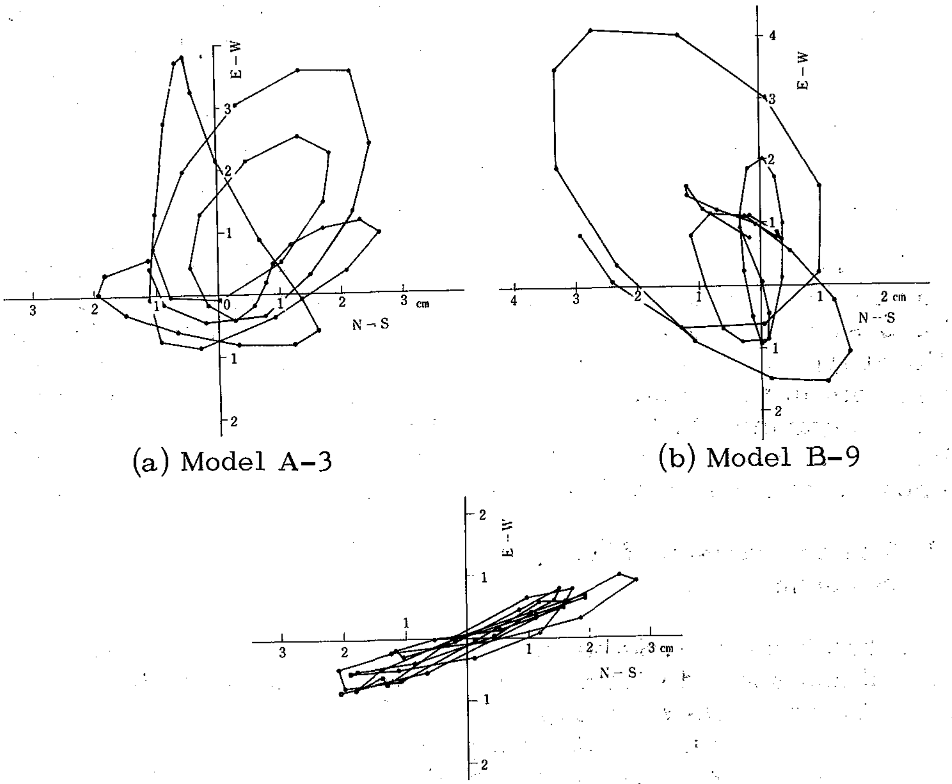
Maximum displacements were varied according to the assumed earthquake records. They were about 3 cm to N - S component of HACHINOHE record and 4 ~ 6 cm to N - S or E - W component of the modified HACHINOHE record of which acceleration was modified so as to be 1.5 times of the original record. Considering the coupling of N - S and E - W components of the earthquake, the maximum displacements in the vector were 4 ~ 6 cm to the HACHINOHE record and to the combination of N - S component of the modified HACHINOHE and E - W component of the HACHINOHE record. Using both components of the modified HACHINOHE record, a displacement of nearly 20 cm was computed. To EL CENTRO and the modified TAFT record, they were 8 ~ 16 and 7 cm, respectively.

Model B (consists of frames in the E - W direction and frames with walls to the N - S direction)



(a) Model A-3 (b) Model B-9 (c) Model C-8

Fig. 11 Maximum displacement of columns



(a) Model A-3 (b) Model B-9

(c) Model C-8

Fig.12 Displacement of the south-west corner column

In the cases of N - S component and both components of the HACHINOHE record, the maximum displacements were similar to those of Model A. Considering both components of the modified HACHINOHE record, the maximum displacement was 8 cm which was less than that of Model A.

Model C (consists of frames and walls in both directions)
In each case of one component of the HACHINOHE record, the displacements were less than 3 cm. Considering both components, the computed displacements were 2 ~ 3 cm in the case of the HACHINOHE record, and 3 ~ 4 cm in the case of the modified HACHINOHE record.

As mentioned above, the maximum displacement had a tendency to decrease Model A, Model B and Model C in that order.

ii) Displacement of each column
Figs. 11 (a), (b) and (c) are the examples of the maximum displacement of each column to both components of the HACHINOHE record. Comparing these figures, it is remarkable that the maximum displacement of Model C is less than those of the other two, while the torsional vibration arose.

Figs. 12 (a), (b) and (c) show the displacements of the southwest corner column. The loci are irregular in the cases of Model A and Model B, while they has an almost regular locus having an angle of nearly 20 degrees in respect to the N - S axis.

4. A comparison of the results of the analysis with the damage of the building

4.1 Dynamic characteristics of the structure

It was supposed that this building would be apt to be affected by an earthquake having the frequency characteristic of near 10 c.p.s. in elasticity. The fundamental mode would be the rotational type of which the center of rotation was located near the walls both in the elastic and plastic zones. As the results of analysis coincide with the damage of the columns and the measured micro-tremor on the roof, the characteristics after yielding would be somewhat similar to that of Model 8 or Model 9 in Table 4.

4.2 The comparison of the computed displacements with the damage of columns

It has been remarkable that the concrete of columns were crushed and spalled within a wide range along the column's length. The reinforcements were exposed and buckled in several columns. It has been recognized that a comparatively slender column supporting a small axial load would be ductile for horizontal force, while it would be brittle supporting a large axial load or having a short shear span.

The computed displacement at the crushing stage of the columns was about 9 cm which was 4.5 times of the yield displacement. However, the response displacement of Model 8 or 9, which was considered a comparatively reasonable model after yielding, was 3 ~ 4 cm to the HACHINOHE record and to the modified HACHINOHE record. These differences may be for the following reasons:

a) The actual earthquake ground motion has a larger acceleration than the assumed value and/or it had a frequency characteristic which more seriously affected this building. The conditions of the ground soil and of the site suggest these ideas, however, more detailed research remains for future study.

b) The frame had not enough ductility because the depth of covered concrete was too large in respect to the whole depth of the column cross section and the concrete contained some poor coarse aggregates. Cyclic loading with a large displacement would decrease the ductility of the frames.

4.3 The effect of the eccentrically located walls

It is recognized that the eccentrical location of the shear wall would be undesirable for aseismic design. In the case of this building it would be evident that the torsional vibration arose during the earthquake. This building was designed as a frame structure by the seismic coefficient method, when the stiffness and strength of the walls were not considered. However, it would not show any unfavorable result against the intension of design procedure.

5. Conclusion

Judging from the examination of damage and the results of analysis, the following conclusions on the behavior of this building during '68 Tokachi-oki earthquake were obtained:

a) The damage of this building was proceeded by the torsional vibration due to the eccentrically located shear walls. Examination of damage,permanent displacements and micro-tremor and the results of the analysis supported this.

b) The eccentrically located walls would not unfavorably affect this building against the intension of the design. However, it is supposed that damage would be reduced if they were located symmetrically.

c) The degree of damage of the columns had a tendency to exceed that supposed from analysis. It would be caused if the ductility of the frames and the earthquake ground motion were more unfavorable than the assumed conditions in the analysis. However, the detailed research on this point is left for future study.

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