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A CASE STUDY ON THE COLLAPSE OF A 7-STORY SRC-BUILDING DUE TO THE 1995 HYOGOKEN-NANBU EARTHQUAKE

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ABSTRACT

A 7-story SRC building collapsed at the 4th story due to the 1995 Hyogoken-nanbu earthquake.
A case study was performed using elasto-plastic dynamic response analyses in order to estimate why and how this type of failure has occurred. It can be said that it was the worst earthquake for this building in terms of spectrum shapes, input directions or intensity levels.

KEYWORDS

collapse at the middle story, elasto-plasic dynamic response analysis, SRC column

1. INTRODUCTION

At least 30 reinforced concrete (RC) or steel reinforced concrete (SRC) middle-high rise buildings (with height about 30m) collapsed at the middle stories in the central district of Kobe city when the 1995 Hyogoken-nanbu earthquake occurred.
This type of collapse was rarely observed in Japan.

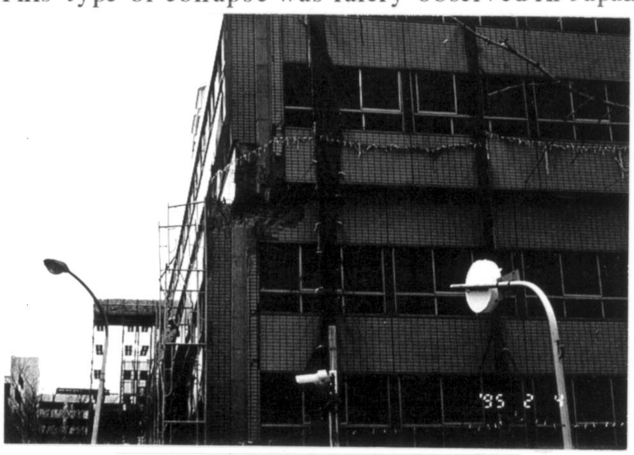


Photo. 1 Overall view of collapse

The 7-story SRC building shown in photo.1 is an example of this type where earthquake damages were concentrated at the 4th story.
A case study was performed using elasto-plastic dynamic response analyses to estimate why and how this type of failure has occurred.

2. THE BUILDING

The building was located inside the intensity level 7 narrow band (1km width and 30km length) defined by the Japan Meteorological Agency (JMA), as shown in fig.1. The maximum acceleration response was estimated to be more than 2000gal. This office building consisted of 7 story steel reinforced concrete (SRC) structure with 2 story basement and 3 story penthouse. The plans under the 6th floor are of a rectangular shape with 27.5m depth (with four 6.875m spans in the E-W direction) and 69.75m width

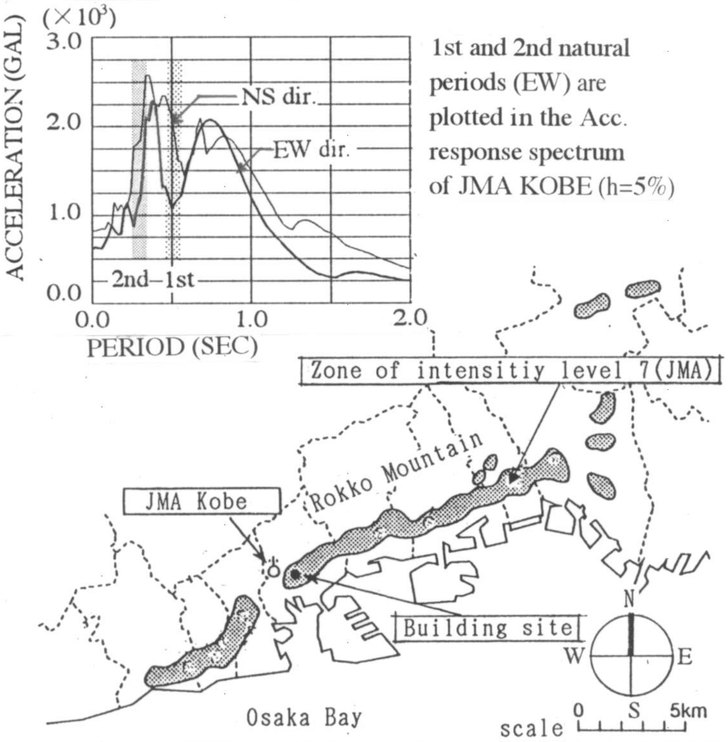


Fig. 1 Location of the building and JMA KOBE

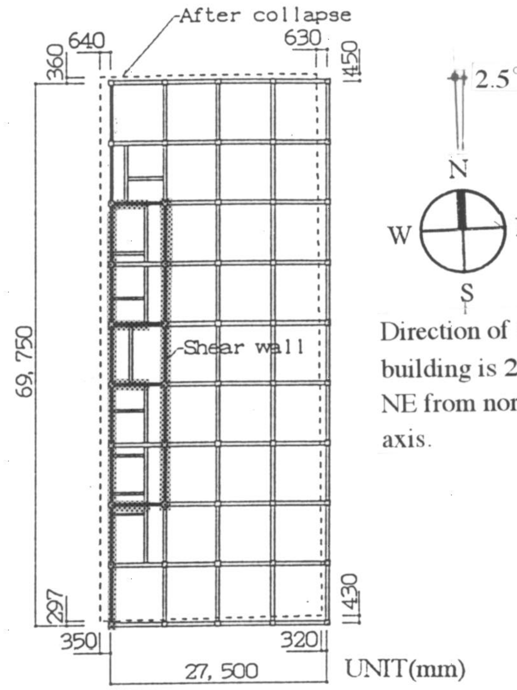


Fig. 2 4th floor plan of the building

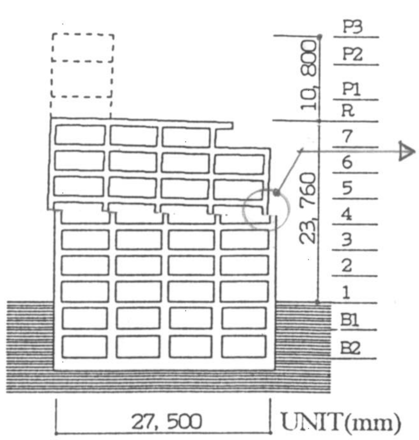
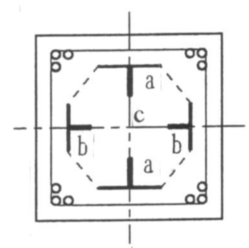


Fig. 3 Elevation of the frame collapse (E-W direction)

Photo. 2 Damage of the column top of the 4th story



B×D=600×600mm (4th story)
a : 2Ls − 100×10mm
b : 2Ls − 75×6mm
c : Tie Plate PL − 9×125@600mm
o : 12×22 φ
hoop : 9 φ − @150mm − □

Fig. 4 Typical cross section of SRC column

(with nine 7.75m bays in the N-S direction). At the 7th and penthouse floors, there are setbacks towards the west. The building core used for the elevator shaft or stair-case, where in-situ reinforced concrete walls are arranged, was located at the west side. The typical story height was 3.3m and the maximum building height was 34.56m. There was a large void space for a banking business between the 2nd and 3rd stories, the columns under the 3rd story were strengthened and sized up in comparison to the ones above the 4th story.
The structure was a moment resisting frame with shear walls, which was designed in 1962 according to the old seismic design code of Japan, and was directly supported by an alluvial deposit at GL-9.77m through rigid foundation beams.

3. THE FAILURE

The building failure was studied using field data; measurement of residual displacements, material strength tests, or observations of the collapse of each structural member. The structure above the 5th story rotated slightly and displaced maximum 640mm to the west and 450mm to the north as shown in fig. 2, and fell down from 240mm at the south-west corner to 1670mm at the north-east corner.

It was possible to determine that earthquake damages were concentrated at the top of the 4th story columns and shear walls just under the 5th floor beams, as shown in fig.3 and photo.2. On surfaces of the 5th floor beams and the 5th and 6th story columns, many shear and bending cracks or the falling off of concrete covering were observed, however the concrete failure did not develop within the shear reinforcement zone (the core zone). Other parts of the structure, especially members under the 3rd story, suffered less damages such as falling off of finishing mortars or diagonal crackings on wall surfaces.
Results of material strength tests are shown in table 1, they clear the design values.

4. DAMAGES AT 4TH STORY

4.1 Damage of the 4th story column

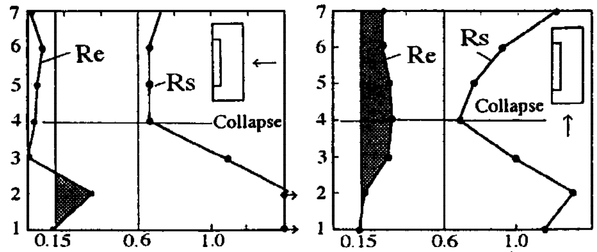
A severe shear failure was observed at the top of all columns. Concrete failure was observed not only in the covering zone but in the core zone. Concrete was broken into small pieces (about 100mm in diameter), which indicated the effect of cyclic dynamic loadings. Some of the encased steels in the column were brittlely broken by tensile stress at the point of the rivet holes, due to that the reduction rate from a gross to an effective sectional area was large in such a small size steel. Longitudinal reinforcements buckled, and shear reinforcements were broken due to tensile stress. Although there were many shear and bending cracks at the surfaces of the bottom of the 4th story columns, cracks did not penetrate into the core zone.
The collapse mechanism was a less ductile weak-column and strong-beam type.
Figure 4 shows a typical 4th story column, 600mm x 600mm in size with a Vierendeel type steel which was fabricated with small size angle sections. The column shear strength was assumed as the sum of the shear strength of RC and steel, however this type of steel has less strength in comparison to the currently used full-web type steel.
The shear reinforcing ratio (Pw=0.11%, 9 ϕ -@ 150mm) was not enough comparing to the currently recommended value (minimum 9 ϕ -@ 100mm), and the allowable shear stress of concrete fs=12kg/cm ^{2} for temporary loading used in those days was overestimated comparing to the currently used design value (fs=9kg/cm ^{2}). Analysis according to the current SRC design method*[1)] recommended by Architectural Institute of Japan (AIJ) predicts that a shear failure occurs prior to a bending yield.

4.2 Damages of the 4th story walls

The westside exterior walls were 150mm thick with a single arranged reinforcement (9 ϕ -@ 200mm, Ps=0.23%), which was weak against an out-of-plane bending. Although it was designed for an in-plane horizontal load, the reinforcing rate was not sufficient to expect an in-plane ductile behavior. Besides, the anchorage into the 5th floor beam was not enough to expect the wall to be resistent against a tensile stress due to overturning action.

5. SEISMIC DESIGN OF THE BUILDING

The seismic design code in Japan was changed in 1981 from a strength dependent method (the old design code) to a ductility and strength dependent method (the new design code)*[2)].
From the current design code points of view :
(1) As shown in fig.10, the equivalent static loading distribution recommended in the old code was underestimated around middle to upper stories. There was relatively less story-shear strength at middle stories.
(2) Figures 5(a) and (b) show the distribution of the stiffness ratio Rs*[2)] and the eccentricity ratio Re*[2)] in the E-W and N-S directions, respectively.
where,
$Rs = rs / \underline{rs}$: stiffness ratio
$Re = e / re$: eccentricity ratio
rs : reciprocal of relative story displacement angle of the story
$\underline{rs}$: arithmetic mean of all rs's
e : distance of eccentricity from the center of stiffness to the center of vertical load
re : elastic radius of gyration defined as the square root of torsional stiffness divided by lateral stiffness
$Rs \geqq 0.6$, and $Re \leqq 0.15$ are recommended for each story to be qualified as a regular structure.
1) The distribution of Rs was not smooth, and was minimum at the 4th story. This is because the column sizes were abruptly decreased above the 4th story comparing to the 1st to 3rd stories.



(a) E-W direction (b) N-S direction

Fig. 5 Distribution of stiffness ratio Rs and eccentricity ratio Re

The 4th to 6th stories have Rs close to the minimum recommended value (0.6) in the E-W direction. The earthquake response tends to concentrate to the middle stories, especially by the E-W direction loading.

2) Re in the E-W direction (except for the 2nd story because of an eccentrical installation of rigid walls for a safety box of a banking business) satisfied the recommended value (0.15), however, Re in the N-S direction are over the recommended value due to the eccentrical arrangement of shear walls. Therefore, torsional movements will occur when the building suffers the N-S direction loading.

6. ELASTO-PLASTIC BEHAVIOR

Elasto-plastic incremental load analyses are performed to obtain the story-shear force and inter-story displacement (drift) relationship.
The assumptions and methods used are :

(1) The horizontal load distribution

It is determined according to the current seismic design code*[2]. The design load level is defined as load factor (λ)=1.

(2) The moment to rotation relationship of SRC members(beams and columns)

It is assumed as tri-linear, in which the stiffness decreases 1st by cracking and 2nd by yielding, then loses rigidity. Cracking moment Mc or yield moment My are according to the current AIJ recommendations*[1], and an interaction between moment and axial-force is considered for columns.

(3) The restoring shear force characteristics of shear walls

It is also assumed as tri-linear, in which the stiffness is degraded with cracking and yielding. Cracking shear strength Qc or yield shear strength Qy are according to the current AIJ recommendations*[1]. The initial shear stiffness of the shear wall is decreased to 30% of the virgin stiffness to consider the behaviour above the design load level.

(4) The shear strength of SRC members (beams and columns)

The shear stiffness is determined according to the current AIJ recommendations*[1], however for the shear strength , a large enough quantity is given to assume that no shear failure occurs prior to the bending yield.

(5) Design values of materials

Yield strength of reinforcements and steels are increased 10% from the nominal design value due to the material strength tests. Concrete strength used is a nominal design value.
Young's modulus and Poisson's ratio used are from the current design recommendations*[1].

Results are :

(1) Relations between the story-shear force to inter-story displacement (drift)

These are shown in fig.6(a) and (b) for the E-W and N-S directions, respectively. The ultimate strength can be retained 2.86 times (it was determined at the 4th to 6th stories, therefore, more can be retained under the 3rd stories) of the design seismic load level in the E-W direction and 6.09 times in the N-S Direction. The stiffness and strength under the 3rd stories are larger comparing to the 4th to 7th stories, in both directions. The stiffness reduction rate from the 3rd to 4th story are so large as 54% (E-W) and 63% (N-S).
Strength and stiffness are larger in the N-S direction, because of the larger shear force capacity of walls. Figure 7 shows the story-shear force to drift relationship of the 4th story (N-S direction) separated in five plane frames (A,B and C are open frames, D and E are frames with shear walls), 64% of the horizontal load is sustained by walls.

(2) Plastic hinge pattern

Figure 8 shows a plastic hinge pattern of a typical E-W direction frame. Many plastic hinges are observed at the beam ends (not at the top of the 4th story columns) above the 4th story, and exterior tension-side columns. These results predict that a collapse mechanism is a weak-beam type, however the real damage was a weak-column type.
One of the reasons is the assumption made in the analysis of SRC members that no shearing yield occurs prior to bending yield. The maximum load

Table 1 Material strength test results and design values

Materials	Test Results	Design Values
Concrete (8 test pieces)	Fc=22~30MPa (density=2.3)	Fc=18Mpa
Reinforcement**1 (6 test pieces)	σ y = 290~330Mpa σ B = 410~510Mpa	σ y = 235MPa
Steel**2 (3 test pieces)	σ y = 290Mpa σ B = 480Mpa	σ y = 235MPa

Fc : concrete strength

σ_y, σ_B : yield and tensile strength

**1 : Round bars (9 φ~25 φ) are used for reinforcement

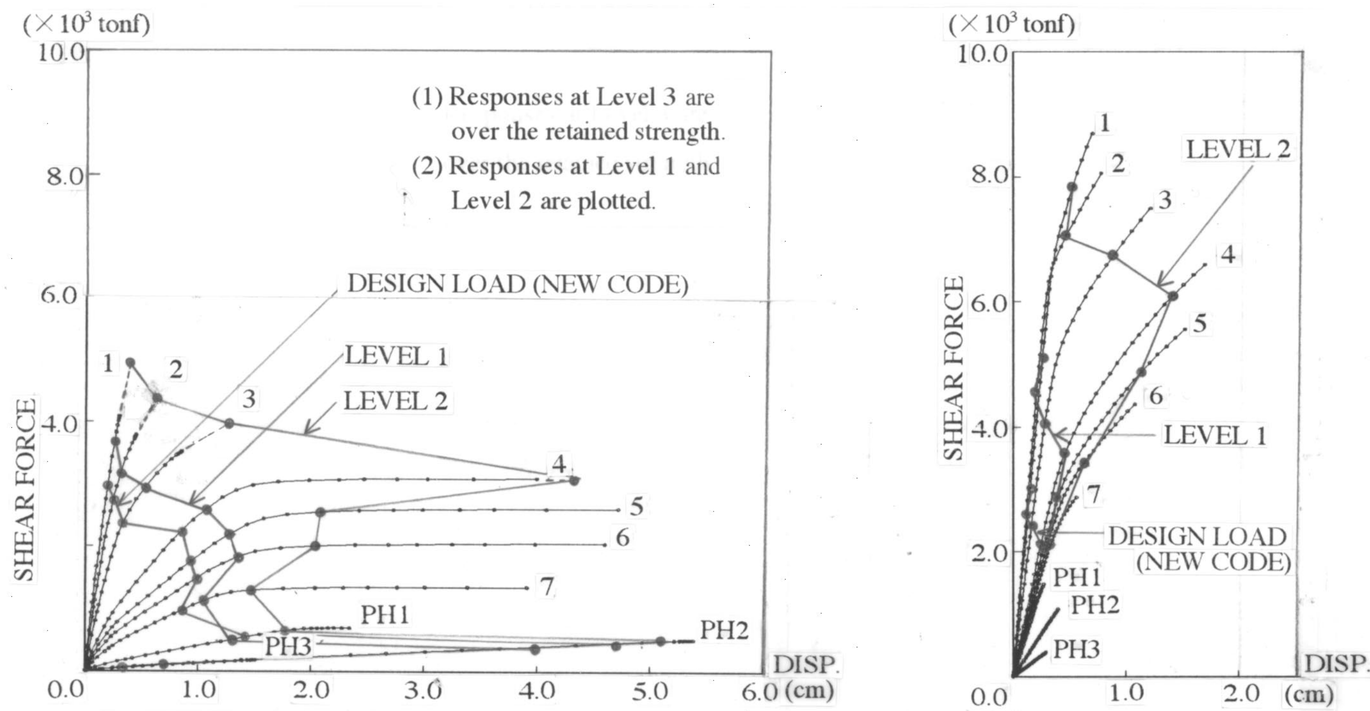
**2 : Rivets are use for joints

Table 2 Natural period (sec)

β	E-W dir		N-S dir	
	T1	T2	T1	T2
1.0	0.47	0.28	0.24	0.14
0.3	0.50	0.29	0.32	0.16
0.1	0.52	0.30	0.43	0.19

T1, T2 : natural period of 1st and 2nd modes

β : stiffness reduction factor of shear walls



(a) E-W (span) direction

(b) N-S (bay) direction

Fig. 6 Relationship of story-shear force to inter-story displacement

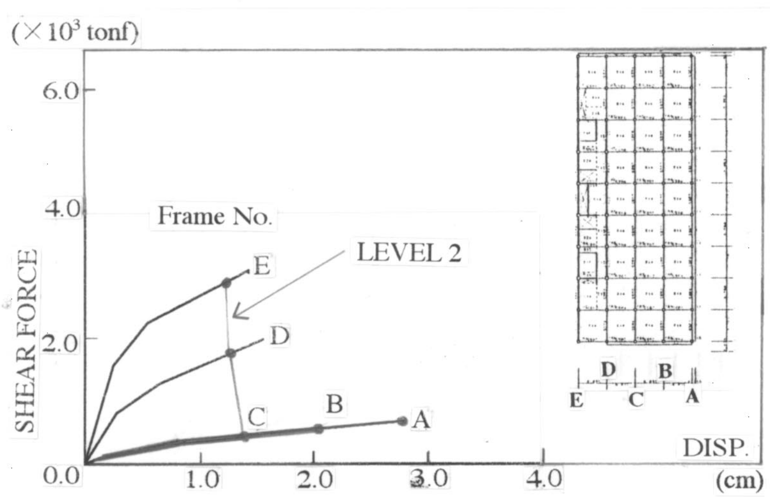


Fig. 7 Relationship of story-shear force to inter-story displacement separated in 5 plane frames

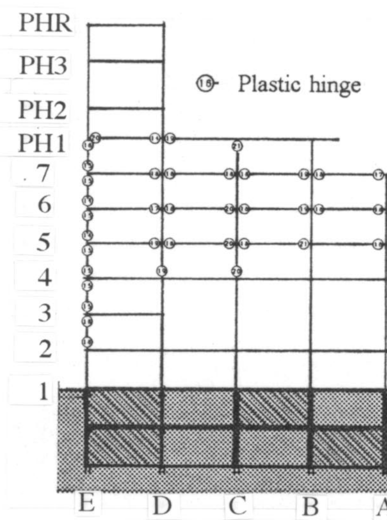


Fig. 8 Plastic hinge pattern of a typical frame in the E-W direction

factor decreased to 2.65 in the E-W direction, when considering the shear yielding of the column according to AIJ recommendations*[1)], however the collapse mechanism does not change drastically. Therefore, another phenomena might have occurred: an effect of UD (up-down) component in the earthquake or a relative strength decreasing of column due to a 2-direction bending which was produced by the diagonal direction loading.

7. DYNAMIC RESPONSE ANALYSIS

Non-linear dynamic response analyses are performed considering 2-directions (N-S and E-W) for the earthquake input data, which were recorded near the building site (JMA KOBE 1995.01.17, 820gal in NS, 619gal in EW and 333gal in UD component).

7.1 Input earhquake record

Some considerations are performed to use the JMA KOBE record :

(1) Spectrum characteristics

The observatory site is the top of a hillside (a hard soil, composed of diluvial deposits), and located out of the zone of intensity level 7. The soil condition of the building site is categorized as an alluvial deposit which consists of multi-layered flood gravels that fell from Rokko mountain covering the base layer of diluvial deposite. Although there are differences in the soil condition, the spectrum shape of input earthquake applied to the building might be similar to the JMA KOBE :

1) The calculated transfer function from base layer to surface showed a peak around 0.33 sec (3.1Hz).
2) There are some records around the zone of intensity level 7, which have peaks around 0.3~0.5sec(2～3Hz) in the spectrum.
3) The natural vibration period of the building is within the periods from 0.1 to 1.0sec.

(2) Direction of external load

Its power was strong in the direction perpendicular to the fault (37° NW from north axis), which is coincident to the real damage observed (52° NW from north axis).

(3) Building to soil interaction

As the building has a rigid basement structure which was deeply inserted into the ground, the input level should be decreased when considering the building to soil interaction.

7.2 Natural modes of vibration

(1) Natural periods

Natural vibration periods of the building are estimated by changing the shear stiffness of walls, as in table 2. Using the initial shear stiffness, 0.47sec (1st mode) and 0.28sec (2nd mode) are obtained in the E-W direction, however they extended to 0.52sec and 0.30sec, respectively after cracking. As shown in fig.1, the acceleration response spectrum of JMA KOBE presents 2 dominant peaks with more than 2000gal around 0.3-0.4sec and 0.7-0.8sec, therefore resonance might have occurred as both (1st and 2nd) periods of the structure get closer to the earthquake dominant peaks.

(2) Natural mode

In fig.9(a), the 1st and 2nd modes of the E-W direction shows that a whipping phenomenon will occur at the penthouse structure, therefore the column axial force in the west side will increase according to the overturning moment response.
A coupled mode of drift and torsion occurs when stimulating to the N-S direction (fig.9(d)), thus the responses will concentrate to the east side open frames.

7.3 Assumptions and methods of analysis

(1) Analytical model

A lumped mass and multi-spring model are used in the analysis. 5 plane frames in the N-S and 10 plane frames in the E-W direction, which have a tri-linear relationship between a story-shear force and drift, were connected by rigid slabs as diaphrams. Each floor has 3 degrees of freedom namely X and Y direction drift and overall rotation.
This model (30 degrees of freedom with 10 mass) can consider a torsional effect due to a movement of the stiffness center caused by frame yielding.

(2) Hysteretic characteristics

It is a degrading tri-linear type derived from elasto-plastic incremental load analysis. The 3rd slopes are extended over the analytical retained strength, in which a restoring force is assumed to maintain at a large displacement region.

(3) Input level of the earthquake record

Three levels are considered. Level 3 is the original

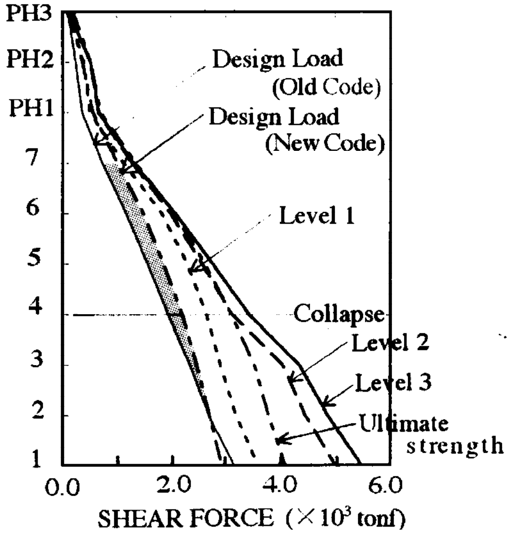
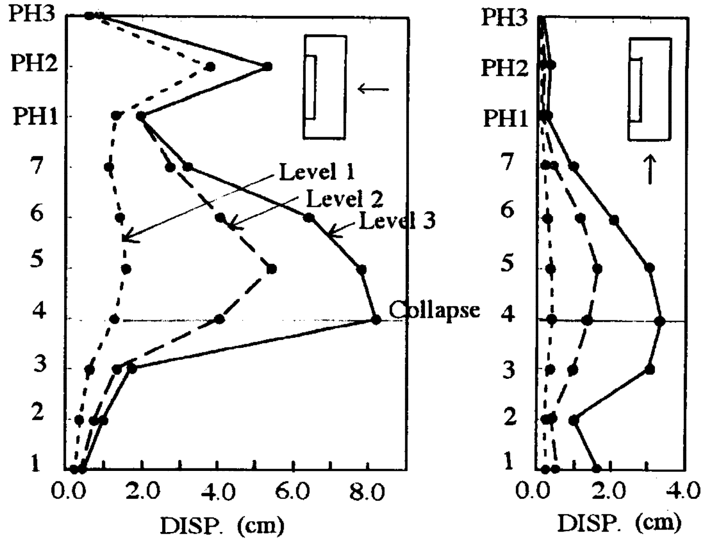
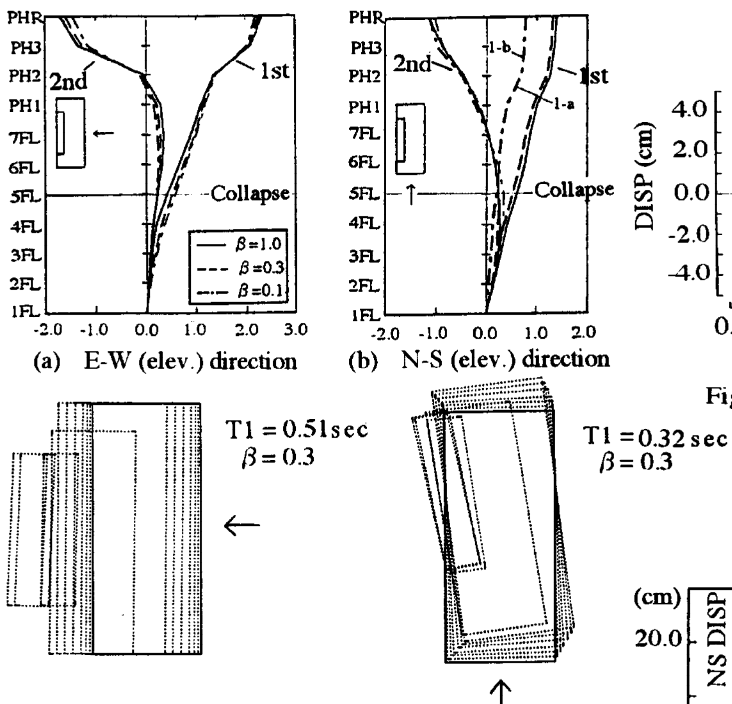


Fig. 10 Maximun story-shear force response (E-W direction) (design loads are plotted)



(a) E-W direction (b) N-S direction

Fig. 11 Maximum inter-story displacement response



(a) E-W (elev.) direction (b) N-S (elev.) direction

(c) E-W (plan) direction (d) N-S (plan) direction

Fig. 9 Natural mode of vibration

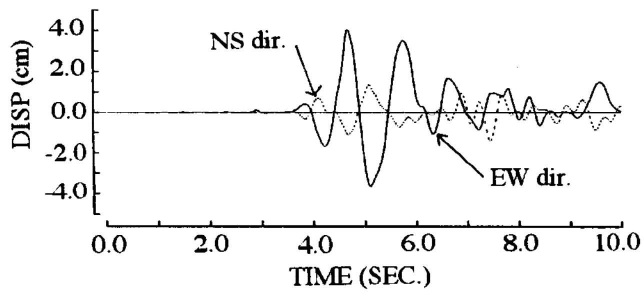
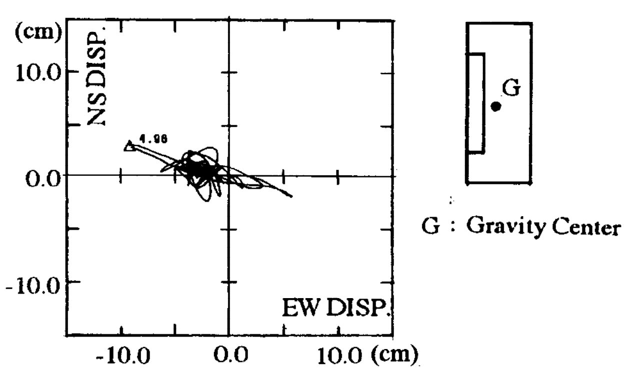
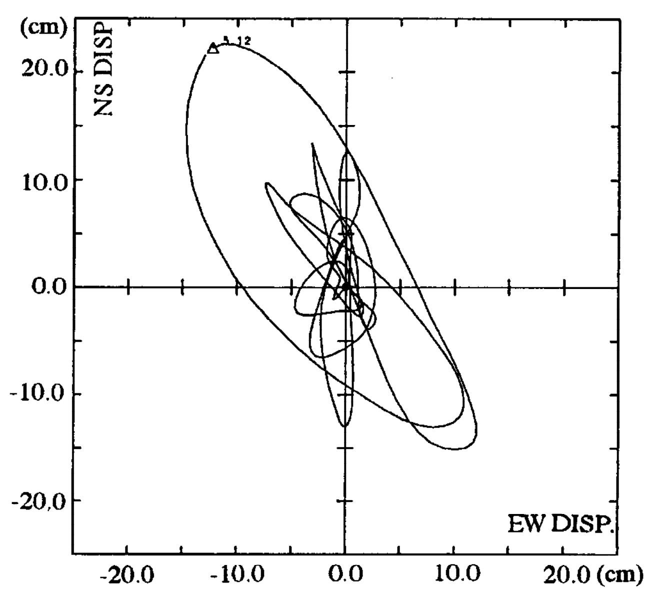


Fig. 12 Time history of the 4th story drift (at Level 2, time=0~10sec)



(a) In the case that shear stiffness of walls is higher



(b) In the case that shear stiffness of walls decreases to 1/30

Fig. 13 Orbit of the 4th story drift at the gravity center (at Level 3, time=0~15sec)

JMA KOBE, and level 2 and 1 are decreased proportionally to level 3 :

Level 3= 820gal(NS) +619gal(EW)

Level 2= 500gal(NS) +377gal(EW)

Level 1= 250gal(NS) +189gal(EW)

(4) Damping factor(h)

h=5% for 1st natural mode of vibration, and proportional to the stiffness for higher modes.

7.4 Results

(1) Maximum story-shear force response

As shown in fig.6 and fig.10, the story-shear force responses are over the ultimate strength of the structure in level 3. The structure might be able to resist elastically for level 1 in spite of being over the design load. The 4th story will lose the restoring force at level 2 in the E-W direction (fig.6(a)).

(2) Torsional movement

The maximum drift responses at level 2 are separately plotted for five plane frames (fig.7). Responses are concentrated to the east side open frame (A frame) due to the torsion.

(3) Inter-story displacement response

The maximum inter-story displacement responses concentrate at 4th and 5th stories especially in the E-W direction, as shown in fig.11. One and a half cycles of drift with more than 1/100 inter-story displacement angle occur at level 2 in the E-W direction, as shown in fig.12, which will be over the deformation capacity of the SRC column with less ductile composition.

(4) Orbit of the horizontal displacement

1) The building vibrated stronger towards the E-W direction (at level 3, the gravity center displaced max. 3.34cm NS and 8.18cm EW at 4.98sec), as shown in fig.13(a), which is consistent with the damaged direction of the building.

2) If the shear stiffness of walls decreases to 1/30 in the analyses, the building vibrates stronger towards the N-S direction (at level 3, a gravity center displaced max.22.6cm NS and 14.7cm EW at 5.12sec), as shown in fig.13(b), which is consistent with the strongest direction of JMA KOBE.

8. CONCLUSIONS

Conclusions are summarized :

(1) Input level of the earthquake load was estimated to be more than 500gal.

(2) Input direction of the earthquake load can be considered the same as the JMA KOBE, which produced 2-direction bending in columns. The observed residual displacements are consistent with this external load direction.

(3) The 1st natural period (T1(EW)≒0.5sec) of the building is between 2 dominant peaks (around 0.4sec and 0.8sec) of the earthquake response spectrum, therefore responses increased according to the extension of natural period caused by the frame yielding. Responses around middle stories increased because the 2nd natural mode (T2 (EW)≒0.3sec which was close to the spectrum peak) was stimulated.

(4) Responses concentrated to the middle or higher stories especially for the E-W direction, due to that the story-shear stiffness and strength above the 4th story were so small comparing to the ones below the 3rd story.

(5) Responses concentrated to the east side open frames as the overall torsion, according to the eccentricity in N-S direction, was stimulated.

(6) SRC columns at the 4th story consisted of fabricated Vierendeel type steels and poor shear reinforced concrete. This composition was less ductile against the shear force. Therefore, concentrated stresses to these columns could not be redistributed to other structural parts.

The seismic design of this building has some dis-advantage, such as non-smooth stiffness and strength distributions in height, a greater possibility of brittle failure because of the SRC columns composition, or the eccentrical arrangements of walls. It can be said that it was the worst earthquake for this building in terms of spectrum shapes, input directions or intensity levels. Therefore, it is possible to see the role importance of ductility and, of stiffness and strength distribution with the dynamic effect, which have not been taken into account by the old code.

REFERENCES

1) Architectural Institute of Japan, " Standard for structural calculation of Steel Reinforced Concrete structures", 1987

2) The Ministry of Construction, " The building standard law of Japan", 1990