3 Damage Analysis and Suggestion of Damage Causes

3.1 Earthquake and Its Property

3.1.1 Characteristic of the Epicenter

The fault rupture of Hyogo-ken Nanbu Earthquake started first offshore at north end of Awaji Island,and the rupture subsequently followed by other two ruptures in south-west and north-east both direction .The north-east fault rupture fracture progressed second fault fracture toward Kobe Station side. Then this fracture caused third fault fracture which damaged severely Kobe city area. That is to say this earthquake progressed chain reaction of three small fault fractures.

Fig.3.1.1.1 shows rupture mechanism of the seismic fault (by Kikuchi, 1995) and Table 3.1.1.1 is the earthquake parameters of Hyogo-ken Nanbu Earthquake .

Table 3.1.1.1 is the earthquake parameters of Hyogo-ken Nanbu Earthquake .

time	5 :46:52 January 17th 1995
epicenter	lat.34° 36'4" long.135° 2'6"
depth	14.3 km
Magnitude	7.2 (Japan Meteorological Agency)
Seismic Moment	2.5×10^{26} dyne・cm
Moment Magnitude	6.9
Fault Area	40 km x 10 km
Total Dislocation	2.1 m
Stress Drop	100~200 bar

As shown the rupture process at upper right in this figure , these fault rupture lasted eleven seconds. During the construction of Shin-Kobe station of Shin-Kansen railway, the station building was designed structurally separated since active fault passed through the station site. It is now under investigation which fault moved during this earthquake. The reason why Intensity 7 belt (JMA) extending east -west is now being discussed from the standpoint whether this is due to unrecognized active fault underneath or due to the amplification of surface soil.

Fig.3.1.1.2 shows existing active faults around Kobe city area (by Shimamoto,1995). Among these faults, followings are believed obviously moved this time; Nojima fault,Suma fault,Egeyama fault and Suwayama fault.

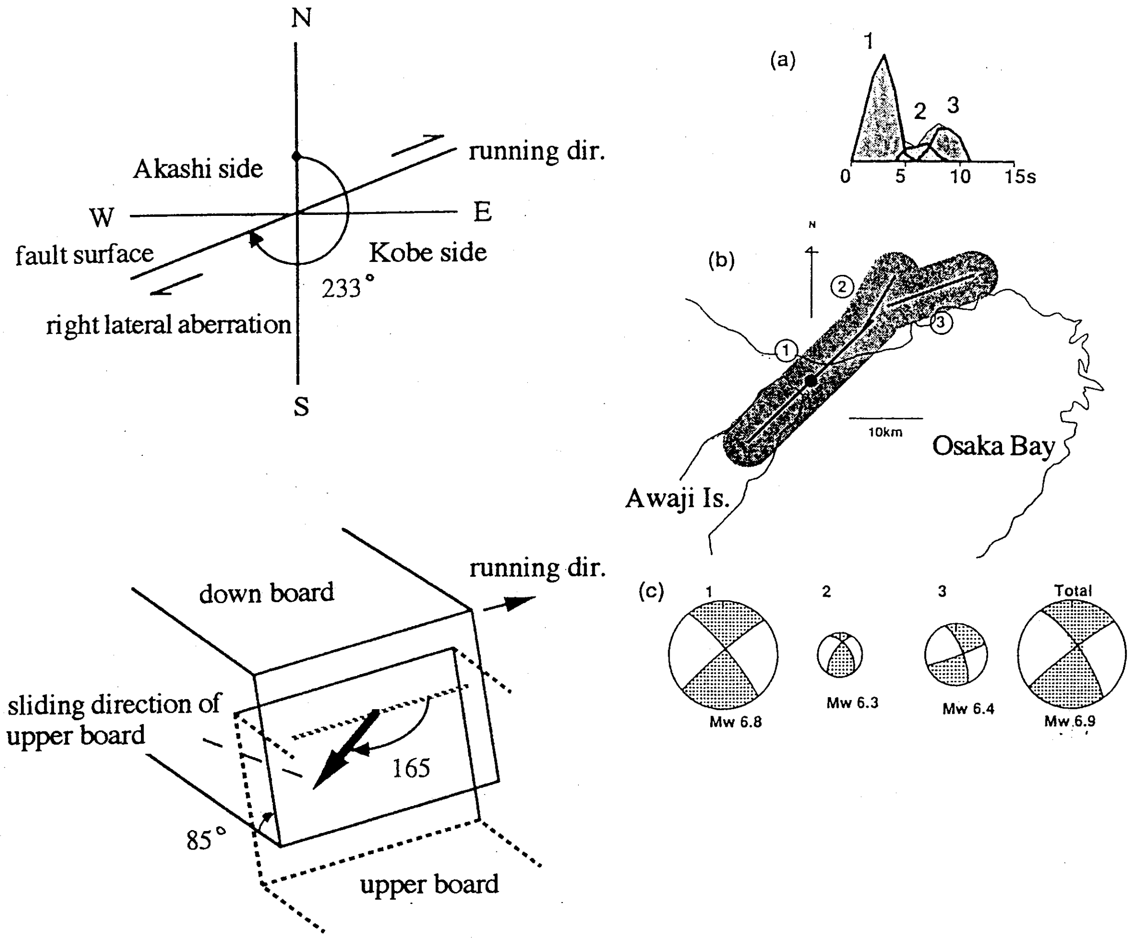


Fig.3.1.1.1 Fault Fracture Mechanism at Epicenter,1995 Hyogoken-Nanbu Earthquake (by Kikuchi)

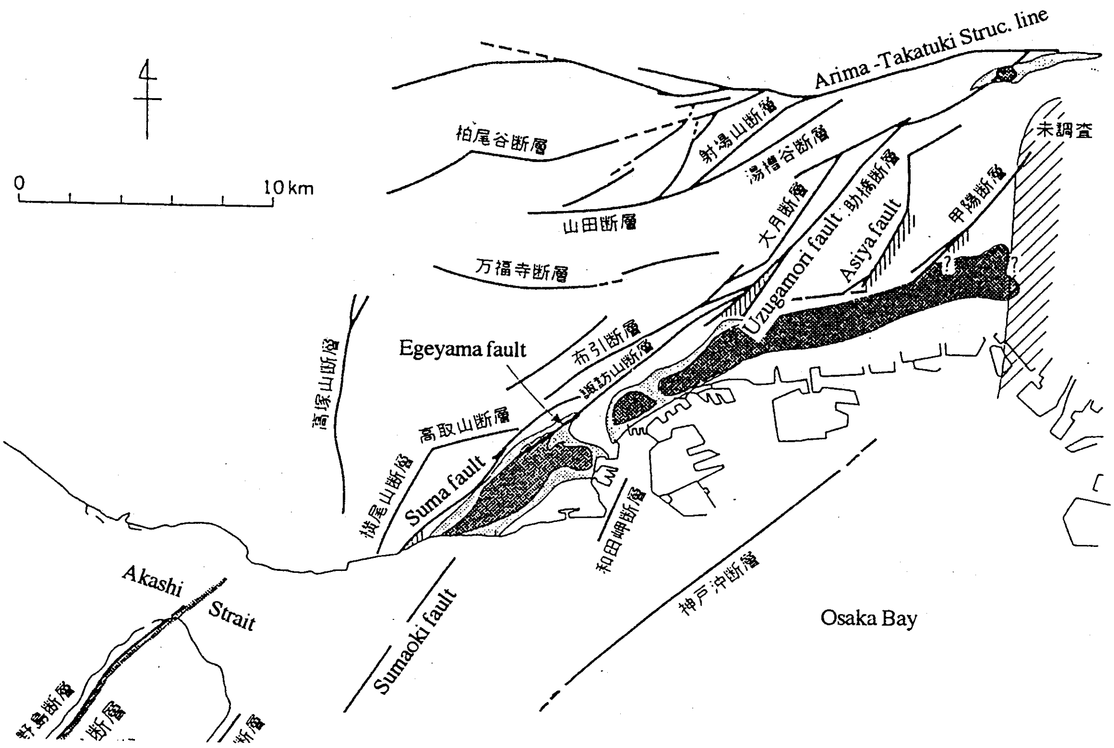


Fig.3.1.1.2 Active Faults at Kobe Area (by Shimamoto,1995)

Considering the above mentioned soil condition and geographical properties, very disastrous damage area to the west side of Kobe station is around JR Takatori and Shin-nagata stations.The severely damaged area of San-nomiya is distributed along Old Ikuta River, i.e. the 'Flower Road'.

Very severely damaged area from Nada station to Asiya station is located along Japan Railways. This damaged area is in the hill-side whereas west damaged area is located in seashore area. In order to identify the reason for the concentration of severely damaged area, it is important to consider the deep subsurface structure and the amplification due to surface soil.

3.1.2 Geological Features and Soil Conditions of Kobe and Its Outskirts

Kobe city is divided into several (three) geological categories , fan form foot of mountain,seaside low land and reclaimed land except mountain area. At the east side of Chuo-ward, several small rivers flow to south and they carry a lot of conglomerate soil from Rokko Mounts . And so their basins are piled up with stones and conglomerates. These areas are fan form foot of mountain. The slope of these areas are steep.Fig. 3.1.2.1 shows the outer layer geological features of central area of Kobe city. Obliquely hatched part shows above mentioned fan form foot of mountain .This figure shows that such areas developed along each river side.

(from Kansai- soil 1992, Japanese Geotechnical Association)

The chain lines in this figure is called as straw-rope pattern epoch sea shore lines and these lines stop at the hatched lines area. These lines mean seashore boundary several thousands year ago as the meaning of the word. The sea side of this line formed seashore low land with thick silt soil layers. (Marine clay layer) Dark green colored parts are artificially reclaimed area.

These straw-rope pattern seashore lines are shown in the north-south soil layer section of Kobe area.Geotechnical data shows that the surface alluvial clay layer is thick at the south of these lines . South-north sections in principal areas are shown in Fig. 3.1.2.2 and Fig.3.1.2.3. From these figures, it is guessed that central area of Kobe city has common geological condition . The geological conditions change from diluvium to alluvium with transition from such fan form foot of mounts to seashore low grounds. In the south-north section passing through San-nomiya station , such boundary locates at JR San-nomiya station and at eastern part, Nada-ward and Higashinada-ward, it is at more southern part; Hanshin Private Railway line and at Hanshin Express Way .

In the section from Kobe station to the Wada cape, this boundary locates at a little northern part than JR line. Severely damaged areas are concentrated at the fan form foots of mounts at eastern parts (Nada-ward and Higashinada-ward) and at seashore low grounds areas (Chuo-ward,Hyogo-ward and Nagata-ward).

It is another feature that Kobe city has many small rivers. Some of them were artificially moved in the modern period. For example, the Flower Road connecting Shinkobe station and Port Island, was the old Ikuta river, and the Ikuta river moved to the east in order to avoid flood in downtown area.

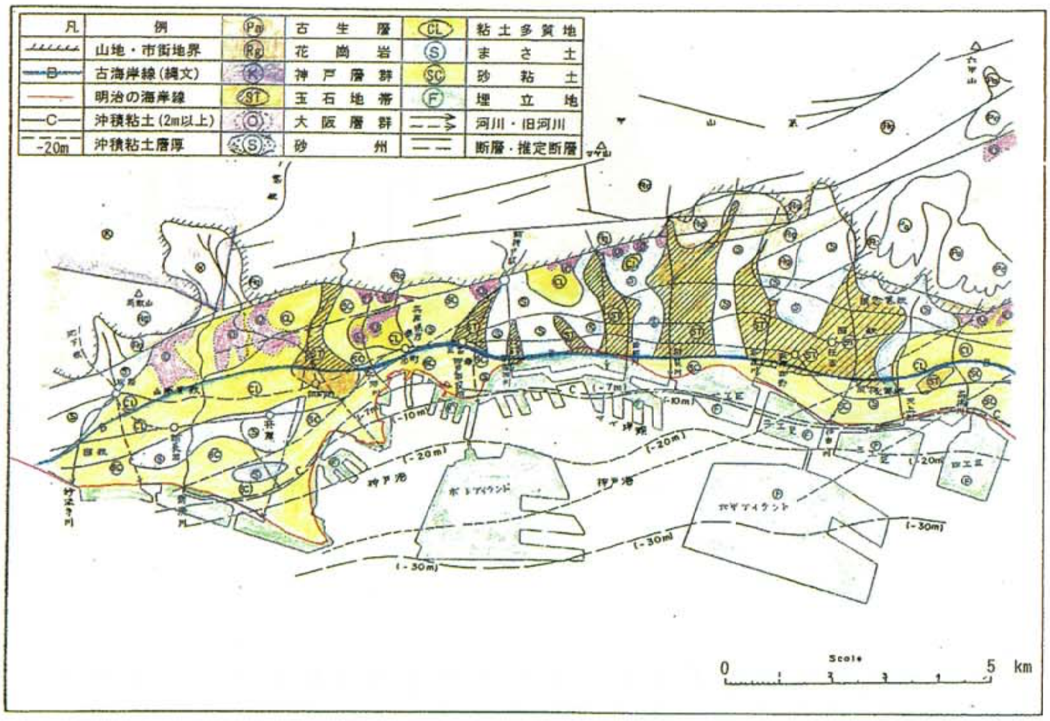


Fig.3.1.2.1　Outlines of Soil Condition at Kobe City Area (Y.Iwami et al ,1982)

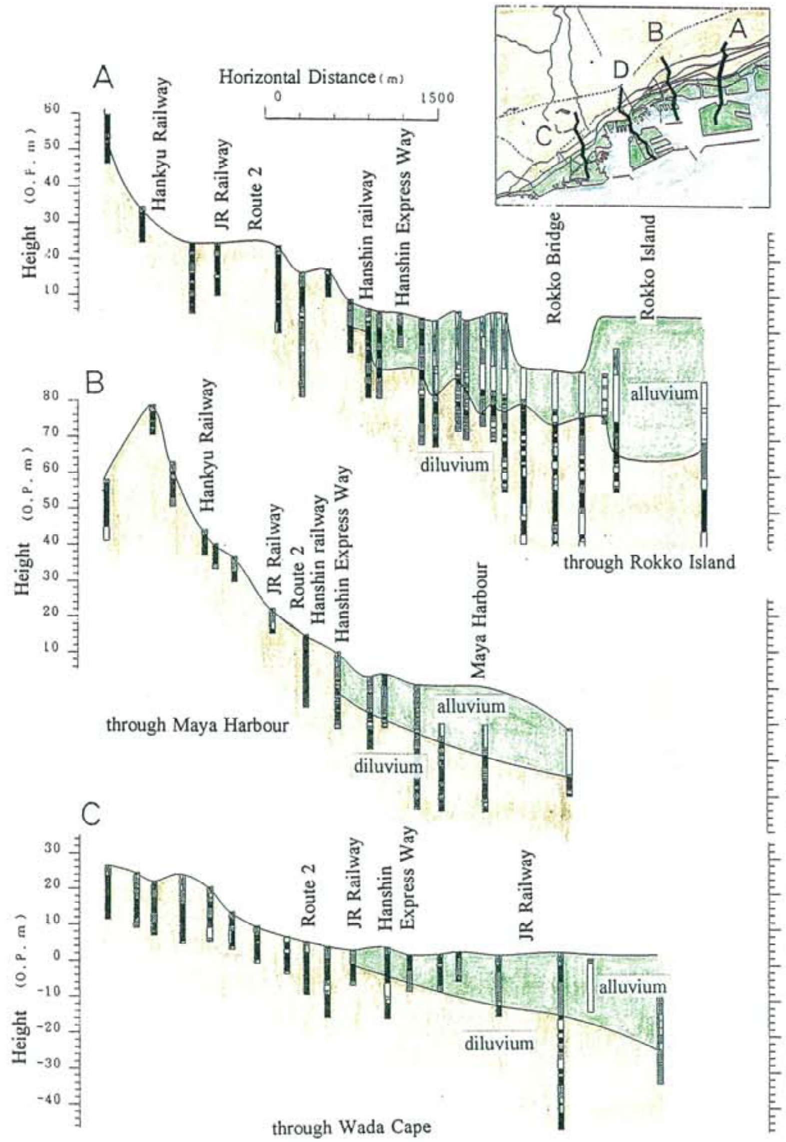


Fig.3.1.2.2 North - South Cross Section of the Damaged Area
("Kansai Jiban, 1992", Geotechnical Engineering Society)

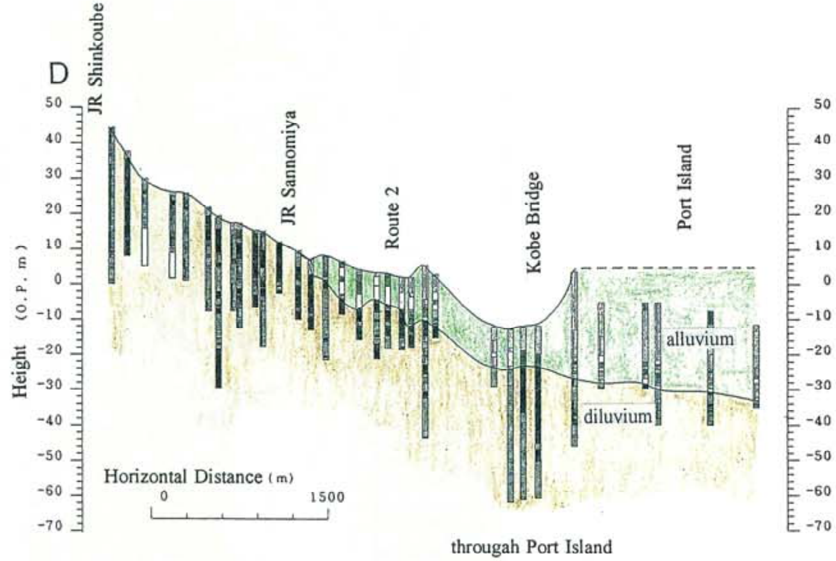


Fig.3.1.2.3 North - South Cross Section through Port Island
("Kansai Jiban, 1992", Japanese Geotechnical Society)

And the Minato river running west of JR Kobe station also moved remarkably. The Old Minato river flowed to sea through Minatogawa Park ,Shinkaichi and Higashikawasaki town. Now Minato river flows to sea from the northern part of Minatogawa Park and Shinnagata station .There were many cases that by such change of river routes, former river areas became to be covered with soft soil and some difference of dynamic property caused promotion of earthquake damage. We have to consider the existence of the Old ikuta river in case of extremely severe damage in San-nomiya area.

3.1.3 Damage Distribution and Soil Condition

The Japan Meteorological Agency announced this earthquake was seismic intensity ; six (the Japanese seven-stage scale; I_{JMA}) in Kobe city and Sumoto city immediately after the earthquake. Afterward it did an additional announce that some parts of the following areas were seven ; Suma-ward, Nagata-ward, Hyogo-ward, Chuo-ward, Nada-ward, higashinada-ward, Ashiya city ,Nishinomiya city,Awajishima Island and Takarazuka city.Seismic intensity 7 which was established after Fukui Earthquake in 1948 was announced for the first time.Fig.3.1.3.1 shows such areas of Seismic intensity seven.These areas are distributed in belt shape at just medium zone between foot of Rokko mounts and present seashore line .These areas were commonly called "disaster belt" of seismic intensity seven. Chuo Kaihatsu Inc.,reported I_{JMA} 7+ for the area with severely damaged RC buildings and the area with collapse ratio of wooden houses larger than 50 %. Such area is shown in Fig. 3.1.3.2. According to this map , among I_{JMA} ; 7+ area, the most severely damaged areas were as follows; Takatori and Nagata station , north side of Hyogo station, around of Motomachi - Sannomiya area Nada, Rokkoumichi, Settsu Motoyama and west side of Ashiya station area.

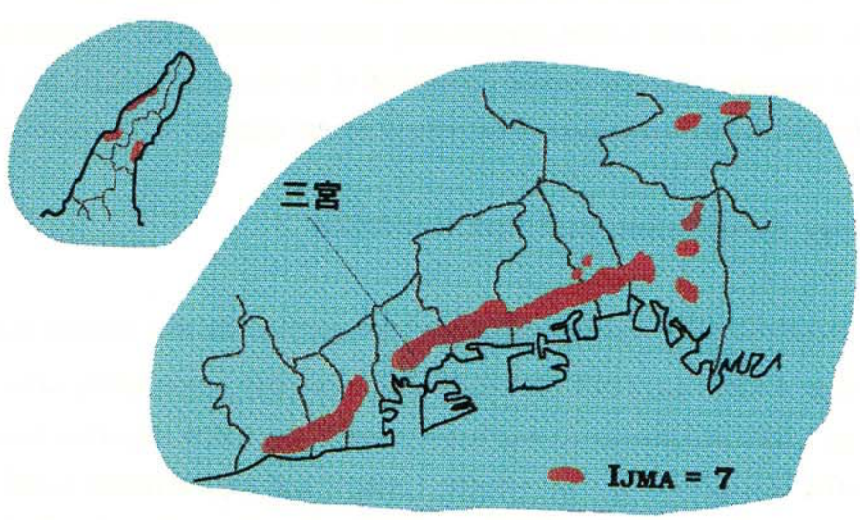


Fig.3.1.3.1 Siesmic Intensity 7 Area by the Meteorological Agency
(Department of Earthquake and Vocanoes,Meteorological Agency)

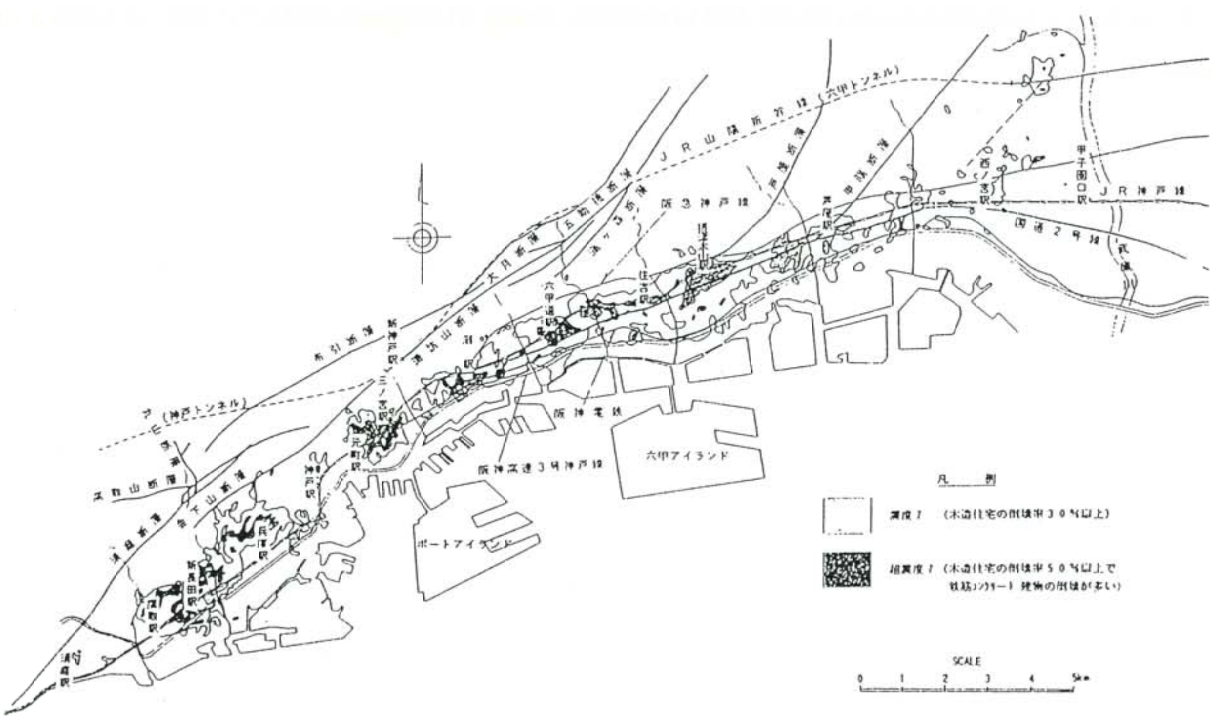


Fig. 3.1.3.2 Damage Classification through Site Detailed Survey
(by Cuou Kaihatu Co.,)

3.1.4 Earthquake Records

It was believed that they had no big earthquakes in Kansai District, so there were so small number of earthquake observation points comparing with Tokyo area. However, several points were set seismometers. In private railway companies,(Hankyu,Hanshin,Kobe kousoku,Sanyou and Subway) simple seismometers were set in their drive control headquarters. However the upper limit of earthquake was too small to be recorded for the earthquake scale of seismographs.

Following companies installed strong motion seismographs ;

Kansai Electric Power Co.
Osaka Gas Co.
Kansai Research Conference for Earthquake Observation
Port and Harbor Research Institute, Ministry of Transport
Hanshin Express way Corporation
Kinki District Construction Bureau, Ministry of Construction
NTT (Nippon Telephone and Telegram)
JR(Japan Railway)
Major Construction Companies

Table 3.1.4.1 shows recorded maximum acceleration . Max. velocity and max. displacement are also shown by integrating such data if the digital data are available , Others are listed from published data. And Fig.3.1.4.1 and Fig.3.1.4.2 show such distribution for max. acceleration and velocity respectively.

The special features are described below ;

(1) Kobe Marine Meteorological Observatory

Fig.3.1.4.3 and 3.1.4.4 are the records of accelerations and velocities. Max. acceleration was 818 gal(90 kine) . The biggest one ever recorded is 922 gal which was recorded during Kushiro-oki earthquake in 1993. The Building Research Institute measured microtremor in the observatory after the earthquake. The results showed that there were some difference between sites including the hilltop where 818 gal was observed, so it suggested that the difference might be based on the effect of surface geology. At the surroundings of the observatory , the roof tiles of wooden houses were fell down, however severe damage such as collapse of house was not observed.

Fig.3.1.4.5 shows pseudo velocity response spectrum with 5% damping for 2 horizontal and vertical components. The response of N-S component showed 250 kine at 0.9 second period. E-W component also showed same levels. Vertical component showed 100 kine at T= 1.0~1.5.

Table 3.1.4.1. Part 1　List of Max.Records of Hyogo-ken Nanbu Earthquake

Observation Point (Organization)	Components	Max. Amplitudes			Remarks
		Acc. (gal)	Vel. (cm/s)	Dis. (cm)	
1. Kansai Electric Power Co.					
(1) Shin-Kobe Substation(SSS)					
・close to Kobe Univ. Point	N090E	584.3	76.8	16.26	
・weatherd soil	N000E	510.7	63.7	22.36	
	UD	495.3	25.6	4.94	
(2) Technical Research Institute (KTR)					
・Deep Alluial Soil	N090E	-	-	-	under inspection
	N000E	298.6	36.8	13.75	
	UD	205.0	20.5	5.76	
(3) Amagasaki Thermal Power Plant No.3					
・Reclaimed Ground	N090E	353.6	50.5	18.63	
	N000E	226.6	45.0	21.44	
	UD	373.4	19.8	6.54	
2. Osaka Gas. Co.					
(1) Fukiai Supply Station(FKI)	N120W	686.5	56.9	19.29	
	N030W	802.0	121.3	43.89	
(2) Nishinomiya Supply Station(NSN)	horizontal	792.0			composed max.
3. PHRI					
(1) Kobe Port Office(KBH)	E043N	204.8	33.9	12.36	
	N043W	502.0	100.4	37.91	
	UD	282.8	31.9	11.05	
(2) Amagasaki Harbor (AMH)	N006W	321.2			
	E006N	472.0			
	UD	310.8			
4. Kobe City Office					
(1) Kobe Port Island (POI)	GL-83m_NS	678.8	65.4	24.59	
	GL-83m_EW	302.6	28.1	11.66	
・Four depths (12ch)	GL-83m_UD	186.7	28.0	12.00	
	GL-32m_NS	543.6	63.2	27.05	
	GL-32m_EW	461.7	57.3	19.90	
	GL-32m_UD	200.0	26.8	11.32	
	GL-16m_NS	564.9	74.4	30.72	
	GL-16m_EW	543.2	52.5	22.78	
	GL-16m_UD	789.2	32.3	16.83	
	GL_NS	341.0	85.4	37.38	
	GL_EW	284.1	50.8	27.67	
	GL_UD	555.9	62.0	26.32	
5. JMA					
(1) JMA Kobe (JMA)	EW	616.6	74.2	19.12	
	NS	817.2	90.2	20.20	
	UD	332.8	39.9	10.20	
6. PWRI (Hanshin Expressway Corp.)					
(1) Higashi-Kobe Bridge (EKB)	GL-33m S78W	303.8	65.7	22.28	
・Reclaimed Ground	GL-33m N12W	445.9	72.8	29.47	
	GL S78W	280.7	81.2	43.19	
	GL N12W	327.3	86.6	40.20	
	GL UD	394.8	34.8	13.63	

Table 3.1.4.1. Part 2

Observation Point (Organization)	Components	Max. Amplitudes			Remarks
		Acc. (gal)	Vel. (cm/s)	Dis. (cm)	
(2) Inagawa Roadway Bridge (INA)	GL-2m NS	421.6	39.8	8.36	
	GL-2m EW	417.3	40.0	10.36	
	GL-2m UD	361.3	20.2	5.14	
	GL-30m NS	200.4	24.3	5.59	
	GL-30m EW	185.3	28.6	10.16	
	GL-30m UD	151.9	20.1	5.21	
(3) Amagasaki Viaduct (AMV)	HA	264.6	51.4	13.52	
	UD	324.0	23.2	3.14	
	HB	293.9	50.0	12.59	
7. NTT					
(1) NTT Kobe Office (NTT)	N039E	153.5	25.2	11.37	
・B3F	N309E	330.7	85.9	27.52	
	UD	169.3	19.5	7.47	
8. Tec. Inst., Matsumura-Gumi (Kita ward、MTR)					
・GL-15m	NS	208.4	21.8	4.87	
	EW	213.6	36.2	8.89	
	UD	166.2	10.1	2.35	
・GL-1.5 m	NS	417.1	29.4	5.71	
	EW	526.4	49.4	9.35	
	UD	418.8	13.4	2.31	
9. Takenaka Corporation					
(1) 'A' bldg at Shin-Kobe (SKB2)	N33W	223.2	30.1	15.52	
・B3F	W33S	208.2	24.3	5.49	
	UD	291.9	46.4	11.47	
10. Housing and Urban Development Corp.					
(1) Shin-Nagata Residence(NGT)	N335E	315.2	60.47	20.50	
・B1F	N245E	121.3	21.15	7.69	
	UD	119.1	12.67	4.63	
11. JR (Japan Railway Co.)					
(1) Nishi-Akashi(AKS)	NS	397.0	39.2	7.57	
(Shinkansen Railway Station)	EW	381.0	33.7	7.67	
	UD	319.0	18.0	3.57	
(2) Takatori (TKT)	NS	642.0	138.0	42.00	*under inspection
	EW	666.0	131.0	33.90	*
	UD	290.0	20.0	5.69	*
(3) Kakogawa(KGW)	NS	240.0	22.8	6.04	
	EW	313.0	27.9	8.11	
	UD	168.0	15.5	2.59	
(4) Takarazuka(TKR)	NS	694.0	75.0	24.60	
	EW	587.0	80.0	26.50	
	UD	410.0	35.0	12.50	
(5) Shin-Osaka(SOS)	NS	204.0	45.3	14.80	
	EW	228.0	38.3	10.80	
	UD	188.0	12.9	4.21	
(6) Shin-Osaka Substation(SOSs)	NS	221.0	34.4	9.17	
	EW	229.0	25.2	6.25	
	UD	62.0	6.3	1.93	
・JR Earthquake Information No.23					

Table 3.1.4.1. Part 3

Observation Point (Organization)	Components	Max. Amplitudes			Remarks
		Acc. (gal)	Vel. (cm/s)	Dis. (cm)	
12. Kansai Earthquake Recording Conference (*1)					
(1) Rokko (KBU)	NS	269.8	55.1		
・Faculty of Engineering, Kobe Univ.	EW	305.3	31.0		
	UD	-	-		under inspection
(2) Higashi-Nada (KOB)	NS	421.0	>40		*2
・Motoyama Primary School	EW	774.9	>40		*2
	UD	379.3	>40		*2
(3) Amagasaki (AMT)	NS	271.4	>40		*2
・Takeya Primary School	EW	321.5	>40		*2
	UD	327.9	26.1		
*1 Accelerations for Kansai Conference are computed from velocity records					
*2 records out of scale					
	・Hatched values are computed by the Building Research Institute				
	・High-Pass Filter is applied for Velocity and Displacement				
	(Components with period of longer than 5 second are gradually removed)				
	NS : North-South				
	EW : East-West				
	UD : Vertical				
	GL-83 means data recorded at 83 meters below ground surface				
	GL means ground surface				
	N120W means the direction oriented 120 degrees westward from the north				

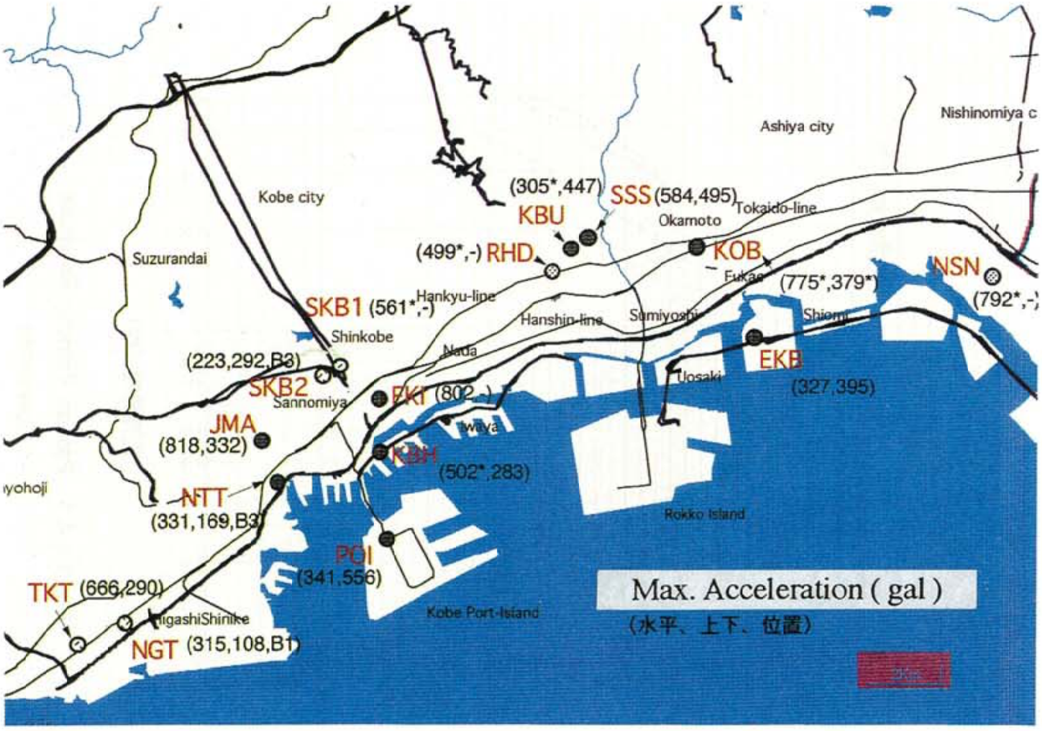


Fig.3.1.4.1 Distribution of Recorded Max. Acceleration (lateral & up and down)

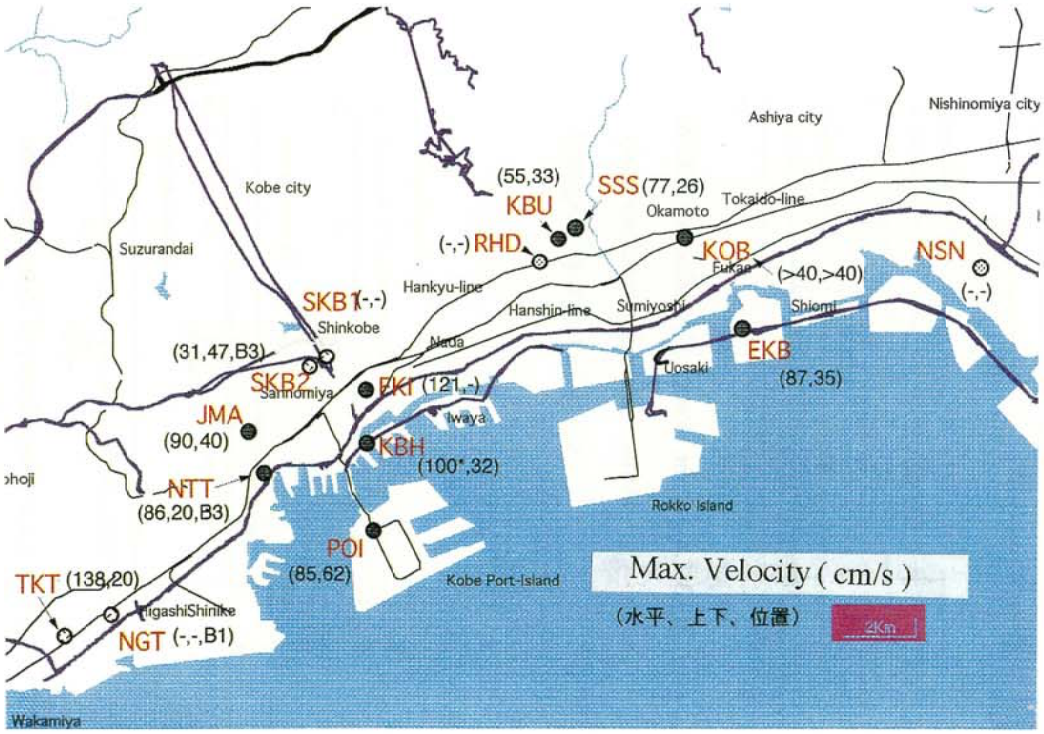


Fig.3.1.4.2 Distribution of Recorded Max. Velocity (lateral & up and down)

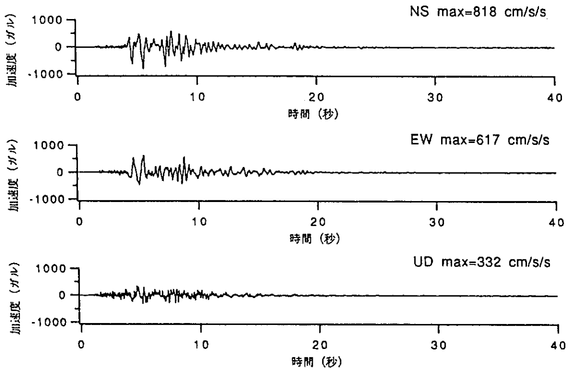


Fig.3.1.4.3 Time History of Acceleration at Kobe Marine Meteorological Observatory
(reported by Meteorological Agency)

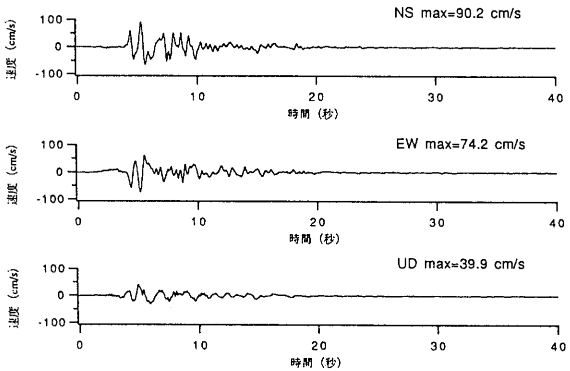


Fig.3.1.4.4 Time History of Velocity at Kobe Marine Meteorological Observatory
(reported by Meteorological Agency)

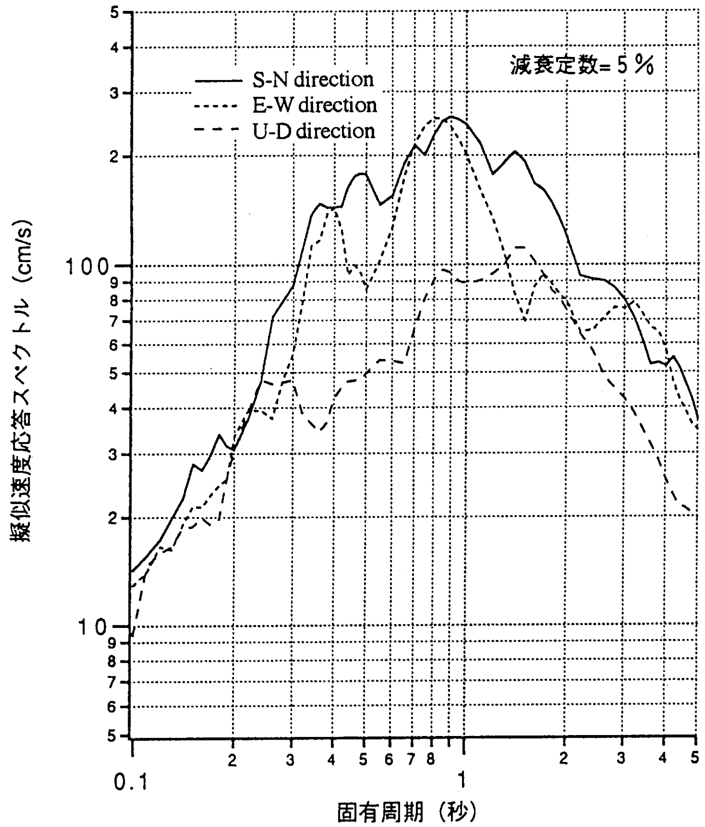


Fig.3.1.4.5 Pseudo Velocity Response Specturum
(reported by Meteorological Agency)

(2) Earthquake Records at Severely Damaged Area

Fukiai Supply Office (Osaka Gas Co.)

Max. Acceleration : 833 gal (2 components composed)

56.9 kine and 121.3 kine (N120W and N30W direction)

Fig .3.1.4.6 and Fig.3.1.4.7 show time histories of acceleration and velocity

The recorded maximum velocity is one of the largest since strong motion observation started in Japan. Fig.3.1.4.8 is the pseudo velocity response spectrum. The velocity 350 kine is seen in N30W at T= 1.1~1.2 second.On the other hand, X-direction showed only 130 kine .Fig. 3.1.4.9 is the comparison between Fukiai and Kobe Marine Meteorological Observatory. In N30W, the spectral value is much larger than that of JMA. The Osaka Gas Company has another station where recorded around 800 gals was recorded in Nishinomiya . The belt of Intensity seven damage grade showed as far as Nishinomiya city . This corresponds to the observed data.

NTT Kobe Station Bldg.

This building is located in the eastern vicinity of Kobe station and is equipped with seismographs at Basement ,3rd and 8th floor. Horizontal acceleration recorded at the basement is smaller compared with the other records such as JMA or Fukiai. Fig.3.1.4.10 and Fig.3.1.4.11 are the acceleration and velocity time histories,respectively . Fig.3.1.4.12 is the corresponding pseudo velocity response spectrum.

JR Takatori Station

Among several recorded motions, Takatori's data was the largest. Acceleration exceeded 600 gals in both directions. Fig.3.1.4.13. shows the time history of acceleration. The maximum velocity showed 130 kine . Vertical acceleration was small.The pseudo velocity response spectrum was as large as 400 kine level at T= 1.3 and 2.0 second . This area might not be in good soil condition.

JR Takarazuka Station

horizontal acceleration : 600~700 gal, 75~80 kine

pseudo velocity response spectrum : horizontal 150 kine (05~2.0 sec)

vertical 70~80 kine

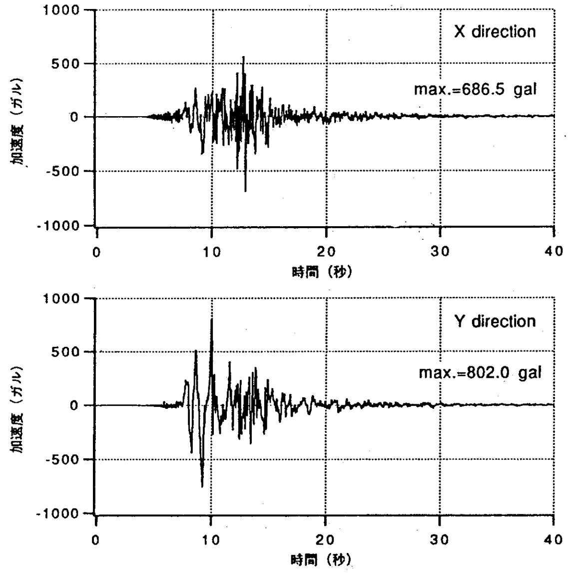


Fig.3.1.4.6 Record of Acceleration (Fukiai)
(Osaka Gas Co.,)

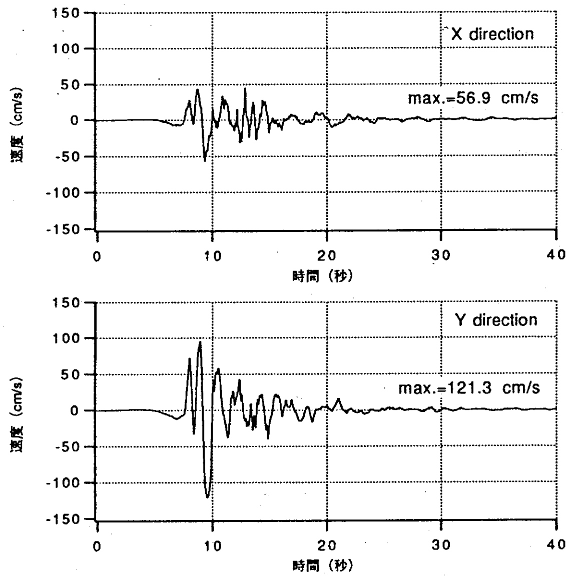


Fig.3.1.4.7 Record of Velocity (Fukiai)
(Osaka Gas Co.,)

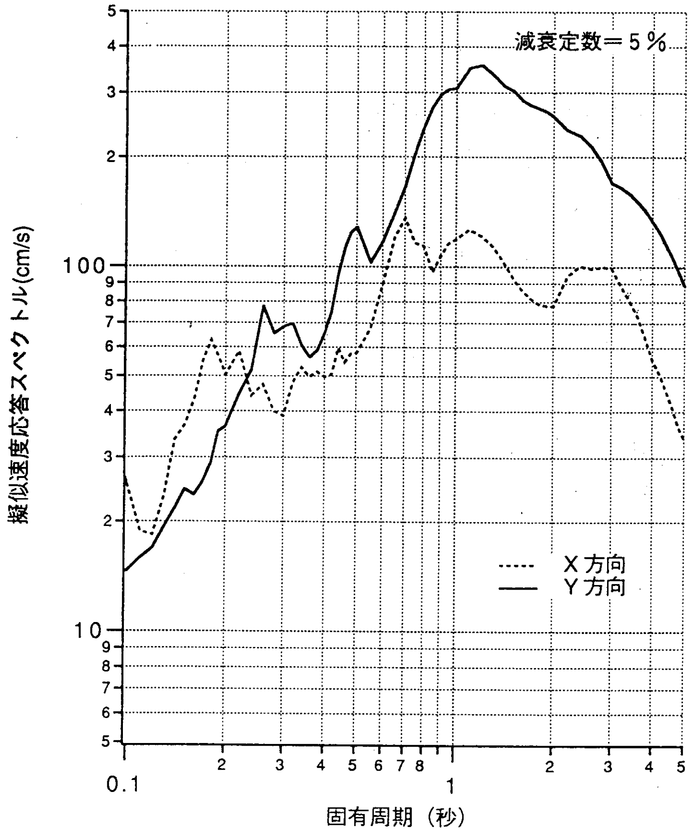


Fig. 3.1.4.8 Pseudo Velocity Response Spectrum (Osaka Gas Co.,)

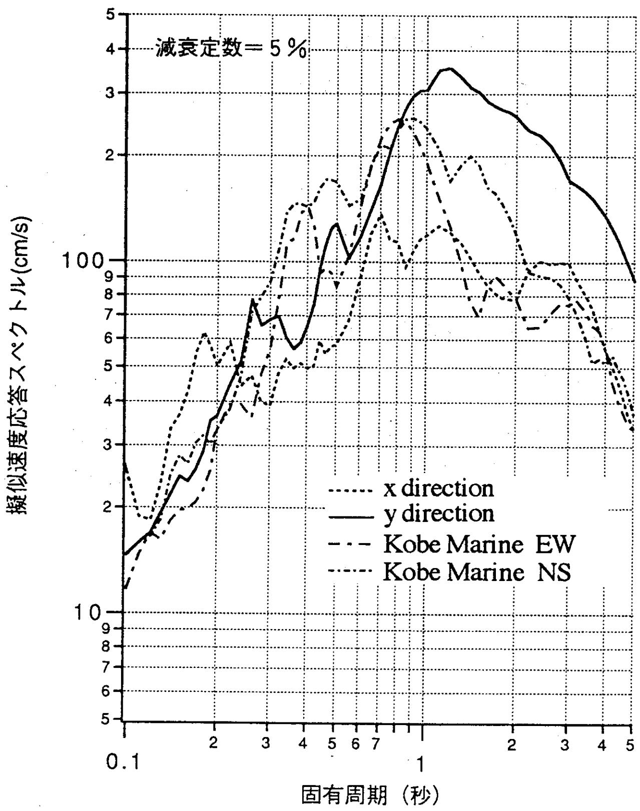


Fig. 3.1.4.9 Comparison of Pseudo Velocity Response Spectrum betw. Fukiai and Kobe Marine Meteorological Observatory

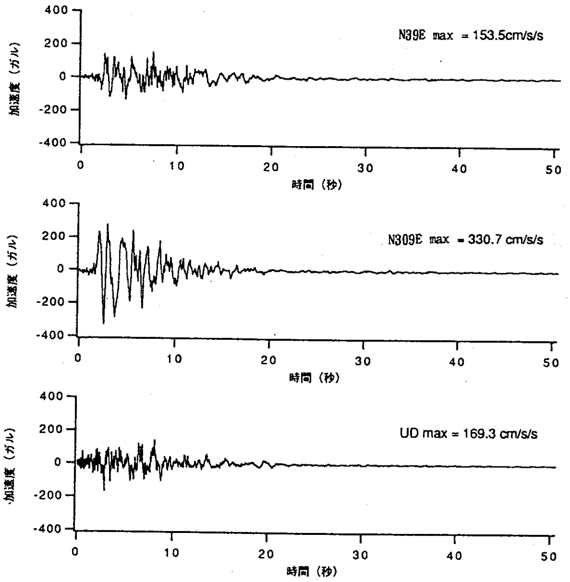


Fig. 3.1.4.10 NTT Acceleration Record
(by NTT Facilities Co.,)

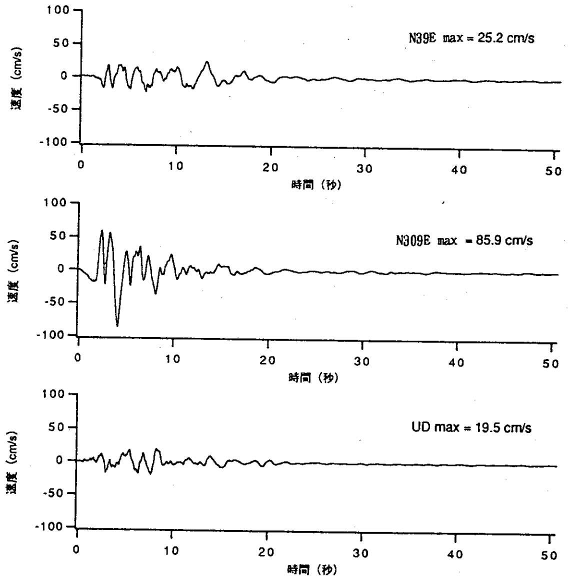


Fig. 3.1.4.11 NTT Velocity Record
(by NTT Facilities Co.,)

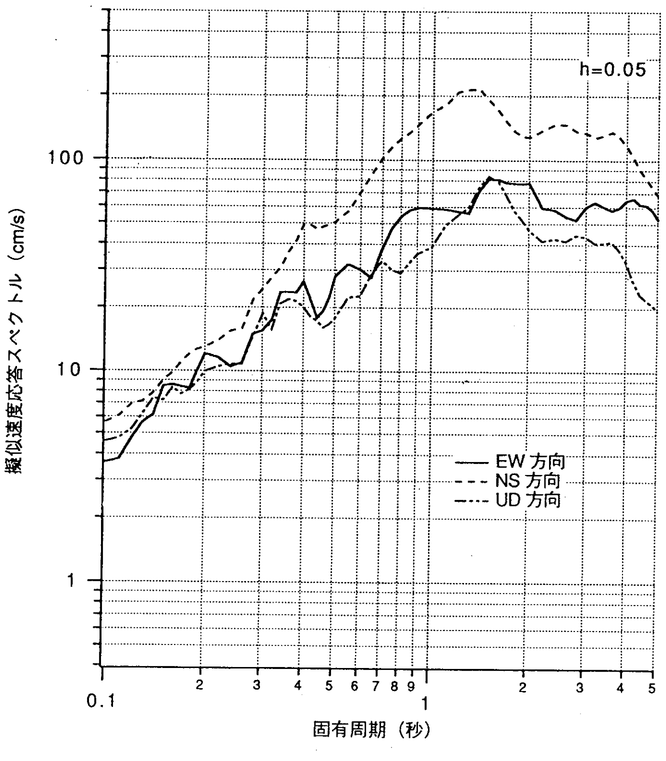


Fig. 3.1.4.12 NTT Pseudo Velocity Response
Spectrum (by NTT Facilities Co.,)

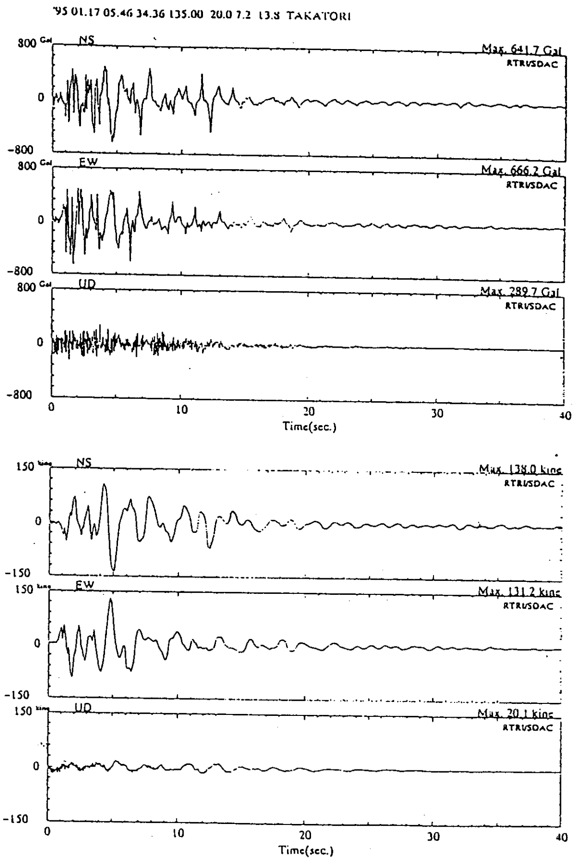


Fig.3.1.4.13 Observed Record at Takatori Sta.
(above ; acc., lower; velocity)
(by Jishin Joho No 23)

(3) Earthquake Records at Hill Side

A Building , Shinkobe

This is a hotel building in which a seismograph is installed at underground 3rd floor.The vertical component is larger than the horizontal component. However the recorded value was smaller than the ones observed on the ground in severely damaged area. The maximum.velocity of the vertical component was 47 kine.Fig.3.1.4.15 is the pseudo velocity response spectrum. (Offered by Takenaka Corporation)

Shinkobe Transformer Substation (Kansai Electricity Power Co.,)

This station stands at steep slope area near the Kobe University campus.Recorded accelerations are as large as 500 ~ 600 gals in three components.Horizontal velocity is 64 ~ 77 kine and the vertical one is not so large.

Faculty of Engineering , Kobe University

Velocity type seismograph were installed ; horizontal 55 kine ,vertical 33 kine.
Fig.3.1.4.16 shows the time history of velocity.

Rokko Substation (Hankyu Rail ways)

At Rokko station , a seismograph was set for train control .This is simple one which records only maximum values . This site is close to Kobe University.

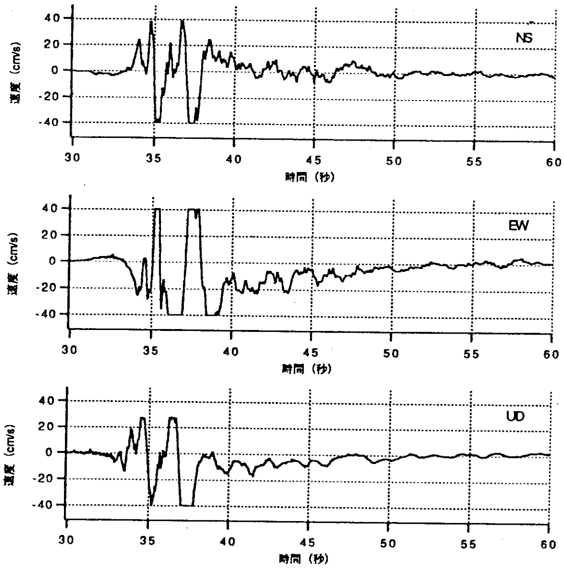


Fig.3.1.4.14 Record of Velocity (Kobe Motoyama)
(KOB,by Council of Kansai Eathquake Observation)

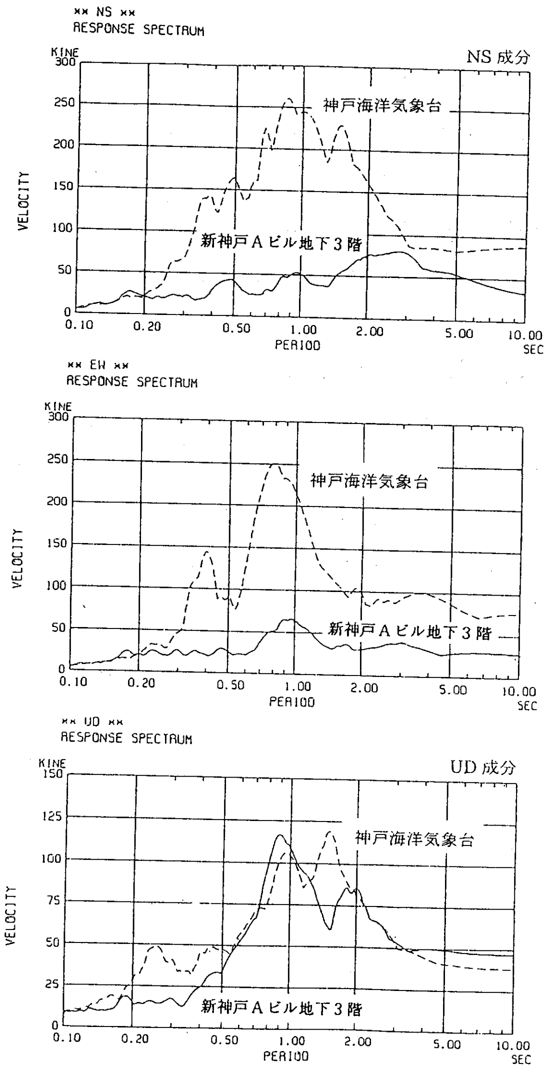


Fig.3.1.4.15 Spectral Ratio of Record of A-Bldg (Kobe) and JMA Record
("Survey Report of Great Hanshin Earthquake,Takenaka Corp.)

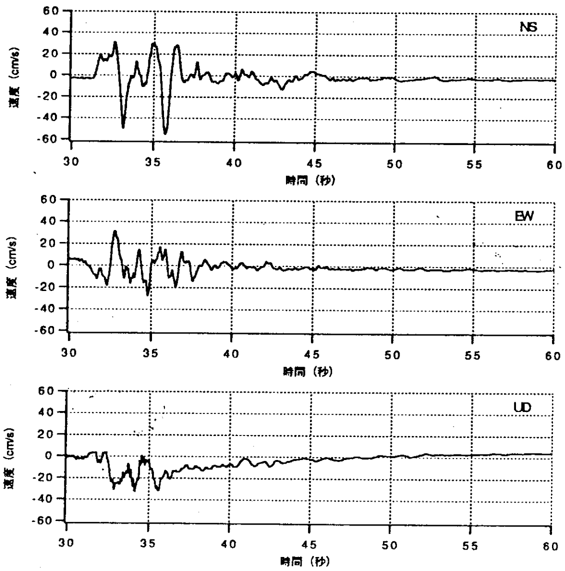


Fig.3.1.4.16 Record of Velocity ,Kobe Univ.
(by Council of Kansai Eathquake Observation)

(4) Earthquake Records at Seashore Reclamation Area

Kobe Harbor Office

This station is maintained by Port and harbor Res. Institute ,Ministry of Transport. N-S 500 gal E-W :300 gal the difference between both direction is remarkable. Velocity amplitude : 100 kine and 34 kine.

Port Island

Development Bureau of Kobe City Office managed the observation which set seismographs at four depths in the soil . As shown in Table 3.1.4.1, the acceleration amplitude decreases as the seismograph approach to the surface of ground.Such phenomena is caused by liquefaction and rapid deterioration of ground stiffness. Fig.3.1.4.17 shows the pseudo velocity response spectrum.

East Kobe Bridge

This is located at 1.5 kilometers south point where the Hanshin Express Way collapsed for several hundred meters.The horizontal max. acceleration was 300 gal and the vertical acceleration was 390 gal at the ground surface.max. Horizontal velocity was 80 kine and vertical one was about a half. Fig.3.1.4.18 is the false velocity response spectrum showing 300 kine at T = around 2.5 second.

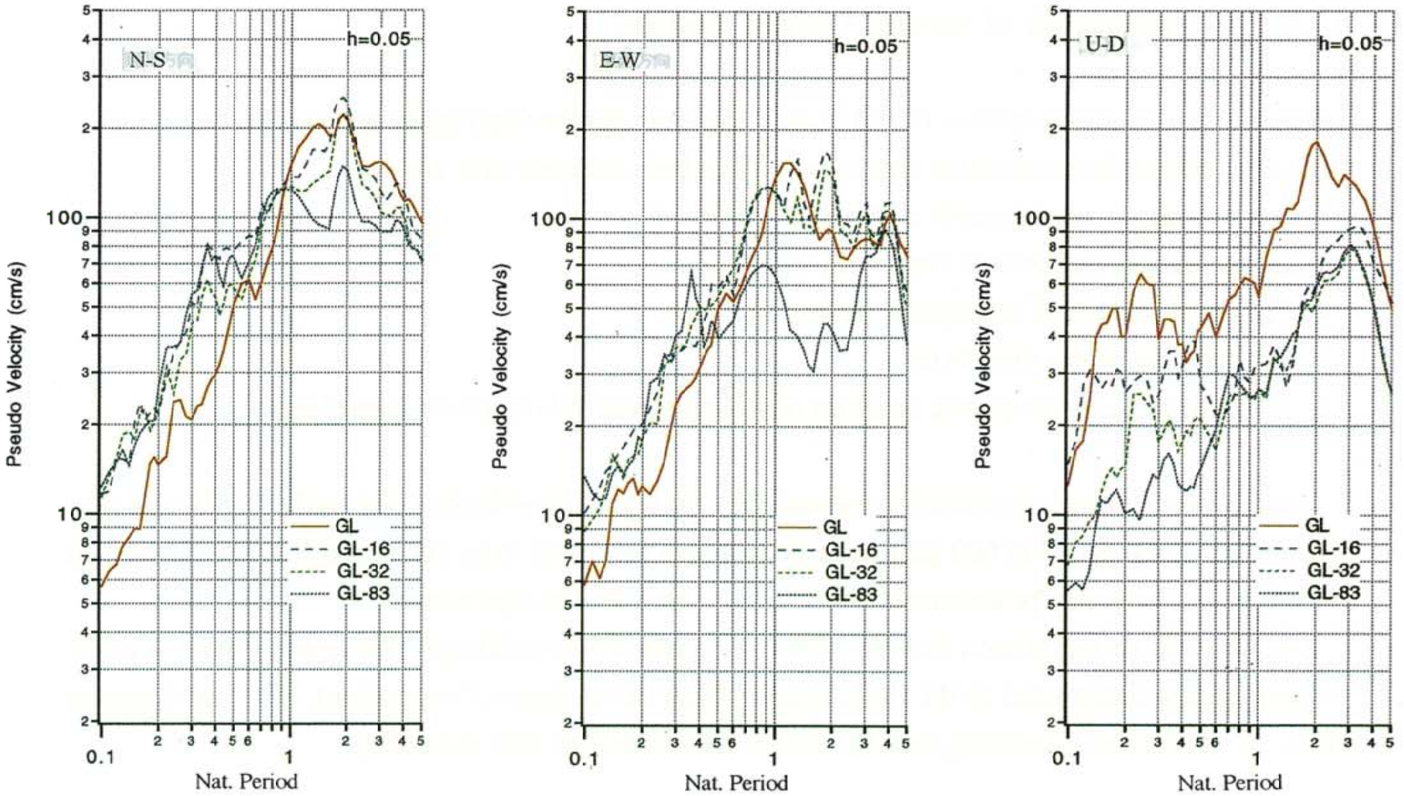


Fig. 3.1.4.17 Pseudo Velocity Response Specrum Recorded at Port Island
(by Bureau of Development, Kobe city and Council of Kansai Eathquake Observation)

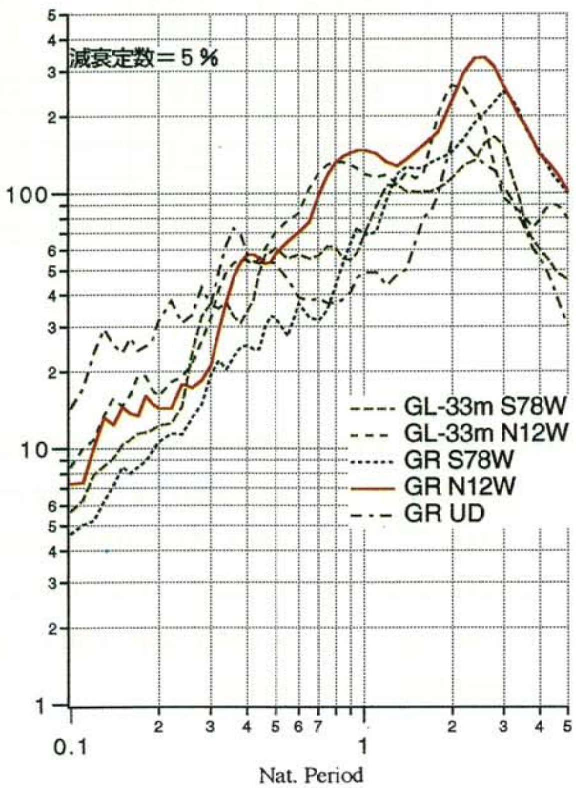


Fig.3.1.4.18 Pseudo Velocity Response Spectrum(East Kobe Bridge)
(by Hanshin Highway Agency and PWRI,Ministry of Construction)

3.1.5 Characteristics of Strong Ground Motion

Characteristics of strong motion record of the Hyogoken-Nanbu Earthquake are as follows:

1. Large amplitude acceleration and velocity have been obtained over wide area.
2. Spectral component exceeds at period 0.8~2.0 sec.
3. Large vertical motion was observed.
4. The duration time of earthquake motion was short.
5. Ground motion has directivity.
6. A record showing the plastic behavior of soil was obtained on soft reclaimed land.

Though the maximum acceleration estimated of the Hyogoken-Nanbu Earthquake varies with sites between 300~800 gal, over 600 gal can be estimated in wide area from Takatori to Takarazuka and the maximum velocity may be assumed over 60kine for area around San'nomiya.

Among these strong motion records some were obtained in buildings. These are at near of Shin-Nagata station (constructed by the Housing and Urban Development Corporation), NTT building near JR Kobe station and A building near JR Shin-Kobe station (by Takenaka Co.).

Table 3.1.5.1 Strong Motion Records Observed in Building

Observation Point	Component	Maximum Amplitude		
		acc.(gal)	vel.(cm/sec)	disp.(cm)
near of Shin-Nagata sta. (B1F)	N335E	315.2	60.5	20.50
	N245E	121.3	21.2	7.69
	UD	119.1	12.7	4.63
NTT building (B3F)	N039E	153.5	25.2	11.37
	N309E	330.7	85.9	27.52
	UD	169.3	19.5	7.47
A building (B3F)	N033W	223.2	30.1	15.52
	W033S	208.2	24.3	5.49
	UD	291.9	46.4	11.47

Every record was recorded at basement floor and the maximum acceleration is less than the other records on the ground. The acceleration and velocity response spectra are shown in Fig.3.1.5.1 and Fig.3.1.5.2 respectively. In these Figs. RT-1,2,3 means the design spectrum for each soil condition category used in the present design code for building. The record of A-building located at mountainside was lower level than the other. On the other hand, amplitude level of Shin-Nagata and NTT are similar to each other for less than 0.5 sec period.

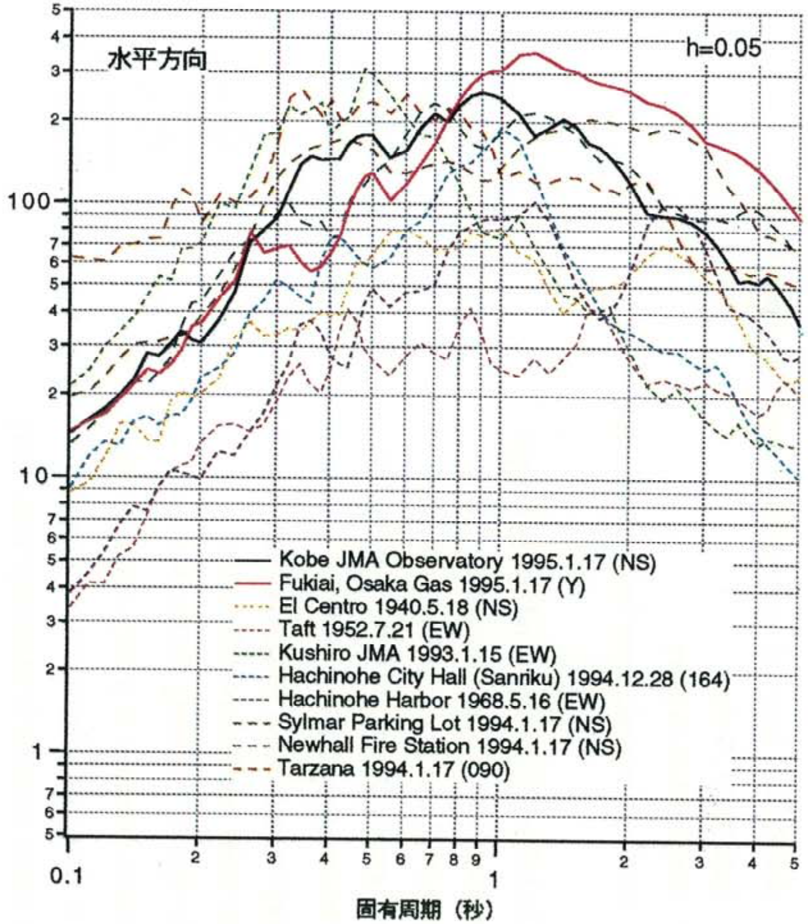


Fig.3.1..5.1 Comparison of Acceleration Response Spectrum for Records in Building

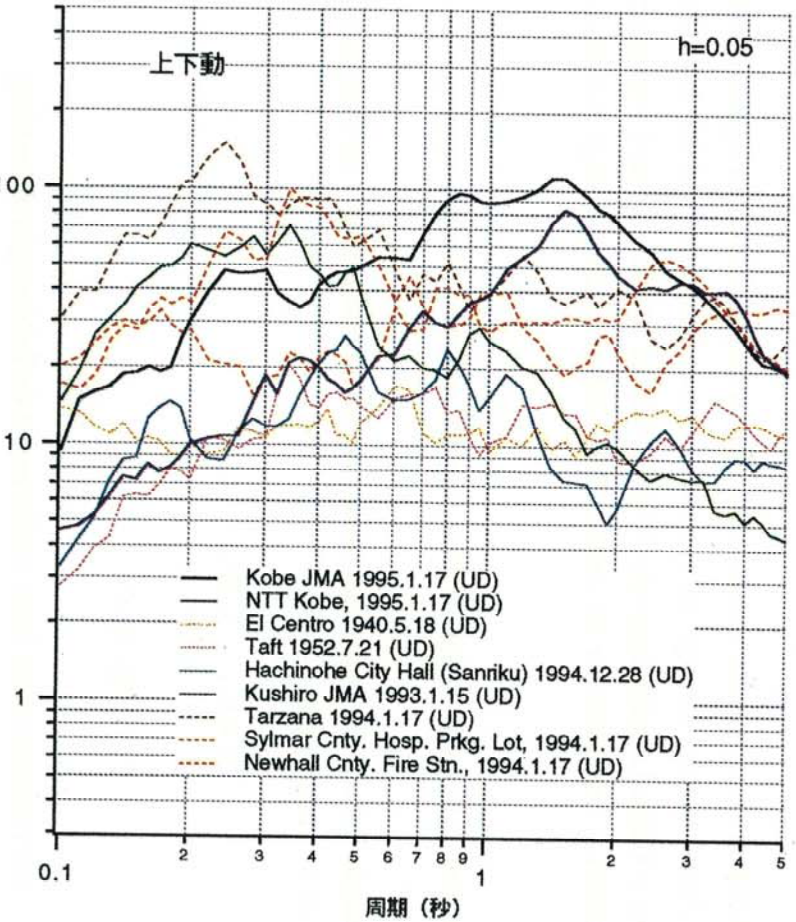


Fig.3.1.5.2 Comparison of Velocity Response Spectrum for Records in Building

3.1.6 Evaluation of the Current Design Seismic Force for the Building Based on the Damage during the Hyogoken-Nanbu Earthquake

The earthquake has left strong motion records with largest amplitudes in strong motion observation history in Japan. From various damage investigation, it has been reported that buildings did not severely damaged or collapsed except those which had structural irregularity such as piloti buildings

In this report evaluation was made for the seismic force subjected to buildings in Kobe city during the earthquake.

(1) The earthquake force specified in the present code is almost appropriate

In the Building Standard Law, Enforcement order, the seismic shear force for design of building is to be calculated from following expressions.

$$Q_i = C_i \sum_{m=i}^{N} W_m$$

$$C_i = Z\, R_T\, A_i\, C_0$$

In this equation,

C_i	:	story seismic shear force coefficient at definite height of building
Z	:	seismic zoning factor
R_T	:	coefficient of vibration characteristics
A_i	:	distribution factor of story seismic shear force coefficient
N	:	number of stories
W_m	:	weight of m-th story
C_0	:	standard seismic shear force coefficient

In case of Kobe, Z=1.0 and for the second phase design (Ultimate Strength Design Co=1.0 as a minimum value. For soil for the category 2 and 3, while for structural design the 1st natural period T is up to 0.8 sec $R_T \approx 1.0$ (because at soil for the category 2 $R_T \sim 0.98$ for $T = 0.8$ sec) then story seismic shear force coefficient at 1 st story, i.e. base shear coefficient defined as $C_1 \approx C_0 = 1.0$ for design. In verifying the earthquake force used in the present standard, it is very important that whether C1, index of the force affected on building, exceeds 1.0 or how much exceeds.Considering multi-story building model shown In Fig.3.1.6.1 we examine

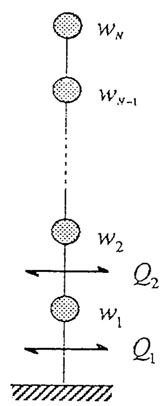


Fig.3.1.6.1.Model of N-story Building

relationship between top acceleration and base shear coefficient. It is assumed that every story has equal mass and distribution of acceleration changes linearly as Fig.3.1.6.2. Define M as the ratio of acceleration at the top of a building to that at ground level , base shear force.
Q1 is as follows:

$$Qi = \frac{a_0}{g} w \{ N + \frac{M-1}{N} \sum_{m=1}^{N} m \}$$

$$= \frac{a_0}{g} w \{ N + \frac{(M-1)(N+1)}{2} \}$$

In above equation g means acceleration of gravity, and,base shear coefficient C_1 is:

$$C_1 = \frac{Q1}{Nw} = \{1 + (M\text{-}1)\frac{N+1}{2N}\} \frac{a_0}{g}$$

The ratio of base shear coefficient to the response acceleration at the top of a building r is:

$$r = \frac{C_1}{Ma_0/g} = \{\frac{1}{M} + \frac{1}{2} \frac{(M-1)(N+10)}{MN}\}$$

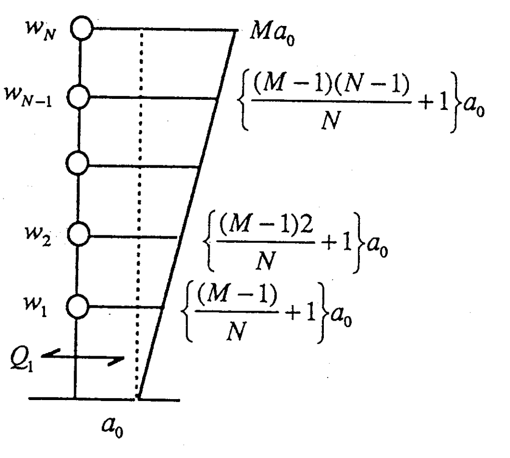


Fig3.1.6.2 Assumed of Linear Distribution of Acceleration

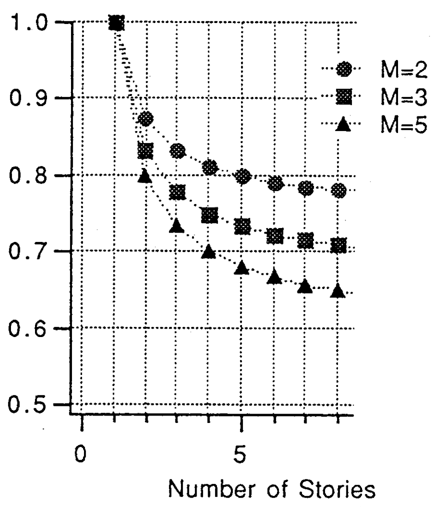


Fig3.1.6.3 Ratio of Base Shear to Top Acceleration

Fig.3.1.6.3 shows plotting of r by number of stories. In this case we assume that the acceleration changes linearly along the height of a building. If it is assumed as top-heavy distribution as well as A_i specified in the standard, the ratio r would be smaller. The magnification factor M for the maximum acceleration at the top of building to at the ground level is empirically 3~4 (or 2.5~3 for single story building) then if base acceleration were 500 gal, it leads to 1.5~2.0 g at the top. In the case base shear coefficient is about 1.1~1.4 at 4th story using r in Fig.3.1.6.3 and M =3~4. As was described previously, with more realistic (top-heavy) distribution, r becomes smaller. Then the seismic force subjected to buildings (i.e. base shear) is at most a little over 1.0 if input acceleration were 500 gal and building were elastic.

In the Hyogoken-Nanbu Earthquake, acceleration exceeding 500 gal was recorded on a free field but as described in section (3), earthquake ground motion is not directly equal to the base acceleration of a building by the dynamical soil-structure interaction. Moreover, the instantaneous maximum acceleration in the record is a maximum value and if it is converted into static external force to building, the effective level of earthquake force should be considered.

As a result, it is assumed that the input motion to the building during Hyogoken-Nanbu Earthquake the Hyogoken-Nanbu Earthquake remained at a slightly larger level specified in the standard even in severely damaged area and the input of earthquake defined in the code is appropriate as a minimum requirement if redundancy descried in section (4) were considered.

(2) The duration time of earthquake was short

In the record of this earthquake, strong part of the motion continues on at most 5~10 sec. It may come from that the magnitude of the earthquake is about 7.2 . The source was not too large as compared with large earthquake occurred under the bottom of the sea and fault rupture completed in short time. In addition, Kobe is within source area then the ground motion consist mainly of Primary wave and then did not last long.

As a result, the repetition of large plastic deformation was few and because of this reduction of structural performance was small and damage had not progress so much.

(3) Strong motion record on the ground differs from the motion recorded in a building

Almost all the large acceleration records during the Hyogoken-Nanbu Earthquake has been obtained on the free field. Though there are few motions recorded in a building, some of them (recorded at B1F, B3F) are about 300~350 gal. See Table 3.1.5.1.

The input motion to a building at the base is strongly affected by dynamic soil-structure interaction. In such cases the amplitude of actual input motion is mostly less than that of the ground motion. The reduction rate depends on the combination of nature of a building and the ground condition (see Fig.3.1.6.4 and Fig.3.1.6.5). Generally, the reduction is estimated about 10~30%.

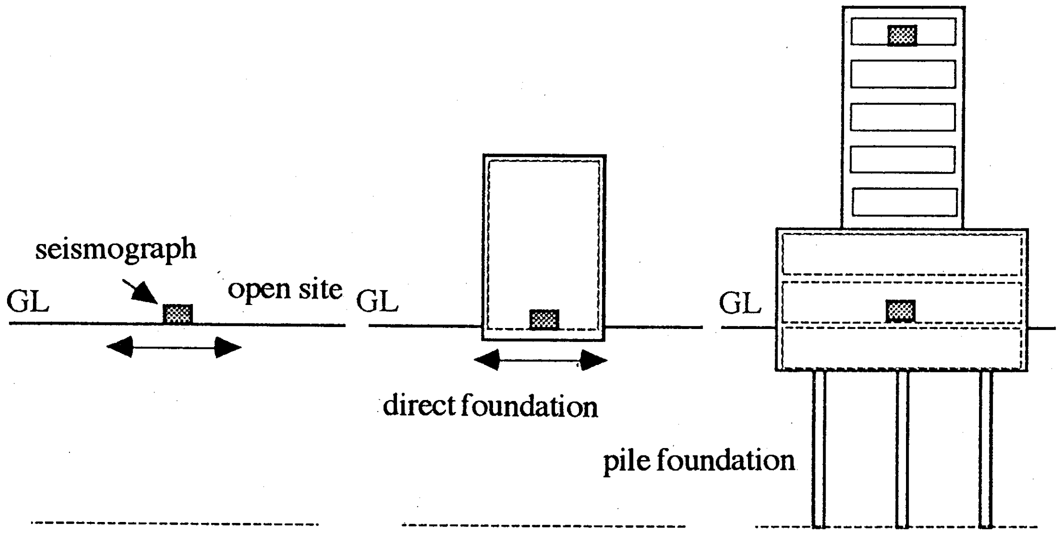


Fig.3.1.6.4. Various Condition of Earthquake Observation

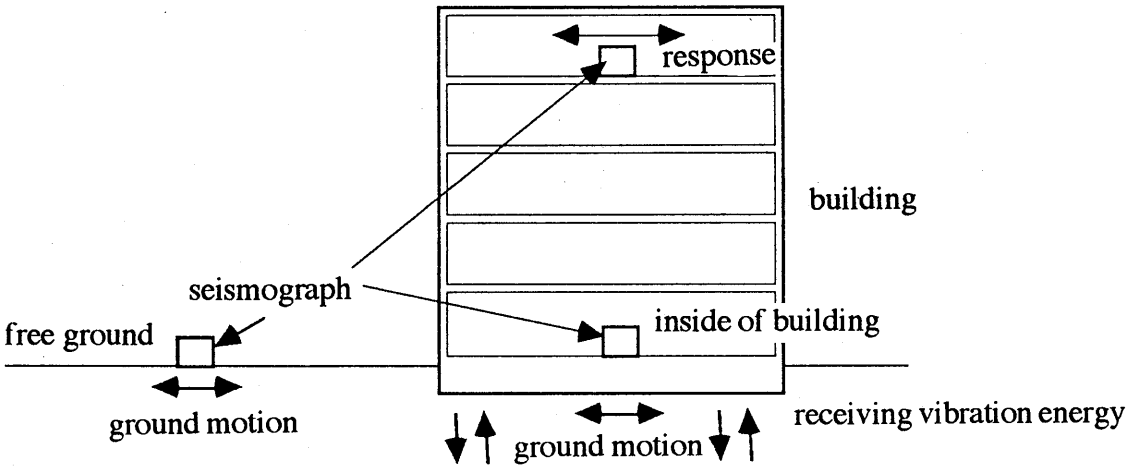


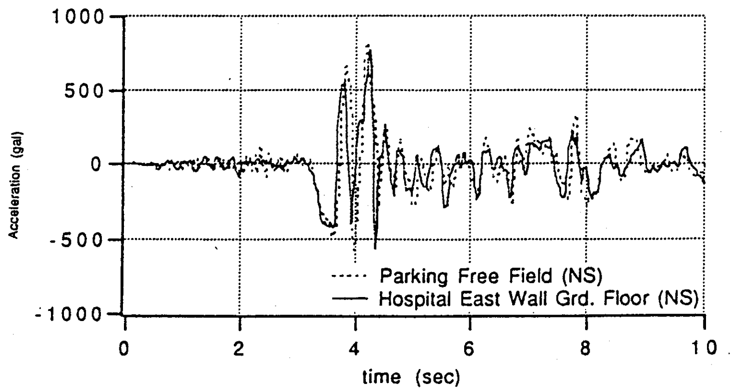
Fig.3.1.6.5 Motion of Free Field and Building

The dynamic soil-structure interaction grows according to the relatively high stiffness of a building comparing with the ground and weakness of soil and then the effective input is reduced. Furthermore, not only the reduction of input at the basement but the increase of damping by effect of energy dissipation to the ground suppress the response of building and the input earthquake force is reduced. The effective input motion has been seen in the actual recorded motions,for example, during the Northridge Earthquake occurred at Jan. 17, 1994, multichannel earthquake recordings at Silmar Country Hospital (olive View Medical Center), California.

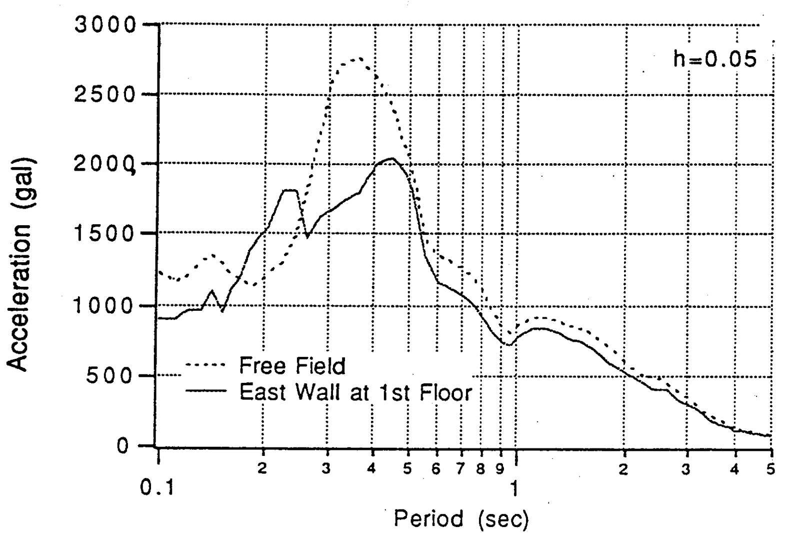
Fig.3.1.6.6.(a) is a comparison of acceleration record of NS component between free field (parking) and inside of building (east wall at 1st floor). The maximum amplitude of acceleration, velocity and displacement are 827 gal, 129 cm/sec and 32.5 cm on the free field and 782 gal, 112 cm/sec and 28.3 cm at inside of the building respectively. Every record at inside of building was smaller than the ones obtained at free field.

From the comparison of acceleration response spectrum(Fig.3.1.6.6.(b)), it is also seen that the spectrum on the free field exceeds to the inside one (this tendency is remarkable at period 0.3~0.4 sec) except short period. As was seen above, even if two points are very close to each other it is sometimes seen that there is a large difference between free field records and the inside ones of the building. In this example, the reduction of acceleration by dynamic soil-structure interaction is not so large. It may come from that this building has no basement floors and built on firm ground.

It can be said that the record on the free field only means the motion at the place where seismometer has set. There is no guarantee the same motion ranges over the whole area of the building. The ground movement is subjected to the effect of the building and therefore the interaction works as a filter which decreases response spectrum at low period.



(a) Acceleration Record



(b) Response Spectrum

Fig.3.1.6.6 Comparison of Records

(4) Unpredictable Marginal Capacity of Building

In a usual structural design, strength of building is shared by main part of structure such as columns, beams, bearing walls and so on, when designing the section the contribution of another submembers is not so precisely considered. The more members expected to have structural strength (though these are not considered), a building has more safety and redundancy. A building is usually redundant (number of indeterminacy is large) and when a part of members begin to get plasticity, the stress is redistributed to the other members and the strength of each one gives full play to its limit state then the restoring force increases gradually. The structural design is done by the strength evaluation (design formula) of every members independently. In these formula a certain safety factor is set respectively and together with the effect described above, the actual safety factor of structural resistant and deformation capacity of structure are actually larger than the calculated. Then if the load level somewhat exceeds the design level, building does not collapse at once. Consider these elements which are usually ignored for seismic design, a building is resistible against the large earthquake which is somewhat larger than that for design.

As a result, the input force to buildings during the Hyogoken-Nanbu Earthquake is considered to be equal or slightly larger than the one assumed in the present standard for structural design at severely damaged area if dynamic soil-structure interaction was considered. In such a case, the base shear coefficient produced in the building might be about 1.0 or slightly larger if building behaved elastic. However, there were various factors such as energy absorption by excess deformation (accompanying damage), effect of indeterminacy, unpredictable marginal capacity etc., then most of the newly designed buildings survived the severe damages as collapse etc. On the other hand, the effect of vertical movement should be clarified by analyses and/or experiments but in some seismic response analysis considering the vertical motion during this earthquake, it has been reported that the vertical motion does not affect so much to the behavior of entire structure. But the influence of vertical motion might be seen at a part of structure such as the center in the floor, resulting fall of furniture or sense of residents to vibration etc.

Evaluating the damage investigation and various analytical studies, it can be said that there is not an urgent necessity of changing the level of seismic force for building design.

3.2 Structures and Materials

3.2.1 Reinforced Concrete Buildings and Steel Reinforced Concrete (SRC) Buildings

(1) Outline of Damages

a) Characteristics of Damages

The followings were observed as the damages of RC structure buildings:

1) The damage of the column situated in the story with low rigidity in which next story rigidity is higher. (ex. a building of which the first floor is used as a piloti or for some stores and the upper floors are used for residential spaces(flats) with many walls.)
2) The damage of the corner column on the first floor caused by the uneven distribution of the wall.
3) The damage of column top on the first floor
4) Shear failure of beam-column joint
5) Inter structure damage (at the border of SRC part and RC part, at the bordering beam between the wall structure part of a stair hall and the other frame structures etc.)

The other cases caused by RC building damages are as follows;

The evacuation was unable, because a door could not be opened due to the shear failure of the non-structural walls . Transportation troubles caused by collapsed buildings.

The similar cases of those ones were already reported from the past earthquake disasters, but what should especially be reported in the earthquake disaster of this time is the story collapse at the special floor of middle and high rise buildings and the joint damages of SRC buildings .

b) Patterns of Damages

The followings show the main damage patterns;

1) The Story Collapse at the First Floor. (Photo 3.2.1.1)

These damages were often seen in the buildings with few walls on the first floor or the ones whose walls were arranged eccentrically. Many of them showed shear fracture of columns. The amount of hoop is insufficient in collapsed buildings .

The shear fracture at the column top was seen in the first floor. Generally, the anchoring edge of the longitudinal reinforcement of column base is set at 1.0 D beneath of column top face (D : Depth of column) . Consequently, it is considered that the damage was concentrated at the column top where the longitudinal reinforcement is less than the other part .

2) Collapse or Severe Damage of *Tiloti* Floors.(Photo 3.2.1.2)

The buildings of which first floors were utilized as parking spaces with wide openings and the upper floors were utilized as apartment houses with stiff walls had the damages especially at those first floors where the stiffness were low. The stiffness distribution towards the height of those buildings were not even. The damages were represented as the story collapse of the piloti floors and the severe damage of columns accompanied with the columns longitudinal shrink(shear fracture).

3) Collapse or Severe Damage of the Specified Floors in the Medium Floor of High and Middle Rise Buildings (Photo 3.2.1.3-3.2.1.5).

The middle and high rise buildings with only story collapse at specified floors. Since the buildings based on the old design standard of law are designed by the evenly distributing design shear coefficient along the height, they are inferior in the strength to those built under the present design standard of law especially at the upper floors. According to the fact that the damage cases in the buildings designed under the present standard have not been reported yet, it can be said that the old standard of law contains at least one of the reasons to occur the collapse of the middle floor of buildings. In Japan, for the buildings of more than seven story, many of those are composed of SRC structure in the lower part and of RC structure in the upper part. The damaged buildings with those structures showed the outstanding decrease of the amount of longitudinal reinforcements of columns, which lead the sudden change of the strength of columns,along the storey at the damaged floor (shifting floor from SRC to RC structures).

The changing to smaller cross sections of columns were also recognized at the floor. Owing to those arrangement systems of reinforcements and the extreme changing of cross sections, the damage was concentrated at such specified floor of lower strength of columns.

4) Damages at the Joint Part of SRC Structure (Photo 3.2.1.6-3.2.1.9)

Some fractures of attached plate at the steel joint part of the SRC column were seen at the SRC structures. The SRC columns shown in the Photo. 3.2.1.6 and 3.2.1.7 are the peripheral columns to moment resisting wall and a large tensile axial stress was loaded at the time of the earthquake. This can be also presumed through the crack caused by the tension seen in the adjoining columns. It is recognized that such a tensile strain caused an excessive plastic strain on the joint position of which the tensile stiffness and the strength are less than those of the main part of steel structure, which lead the fracture of the splice plate of the joint part. A slip at the bottom of anchor plate was also seen at the column base of first floor. The lack of tensile strength and pull out strength of anchor bolt is presumed for the reason.

5) Damages of Wall Type Reinforced Concrete Structures

The damage of wall type reinforced concrete structures is commonly said small and the level of the damage is in the rage of "non-damage" or "quite small damage" in almost all of those structures.

This can be also said for the area attacked by the earthquake of the intensity of 7 (JMA). For example, while there are many other buildings evaluated as "very severe damaged" or "collapsed" in terms of damage rate evaluation, the wall type reinforced concrete structure buildings of next door are not seen any damage. Those cases were often seen.

The wall type reinforced concrete structures with the severest damage out of those investigated are going to be surveyed. Especially, as wall type buildings with severe damage have not been reported yet.

Some of damaged wall type RC buildings are introduced below;
The five story two buildings on the slow slope surrounded by retaining wall of 1-2 meters height were built in 1973(Photo. 3.2.1.10).
The formation is supported by pile base, but the ground fracture occurred under the buildings and the followings are identified in and around them such as sliding of the retaining wall (about 15 cm), subsidence of ground and movement of the buildings themselves (Photo. 3.2.1.11). On the other hand, the damage of the buildings themselves are not so severe. Only shear cracks (about 5 mm width) at corner parts of windows can be concentratively seen on the first and the second floor of the northern side of the buildings. Any shear cracks can rarely be seen on the other structural wall in bay direction (Photo. 3.2.1.12 & 3.2.1.13).
In the vicinity, there are two buildings of seven story and moment resisting frame with structural wall, and since each building has a severe structural damage, no resident has been lived in the buildings since the earthquake (Photo. 3.2.1.14 & 3.2.1.15)
Though the damage rate of wall type reinforced concrete structure buildings is not precisely defined because the investigation was done only for the outlook, the rate seems standing in the level of "small damage". On the other hand, the damage rate of the frame structure buildings situated in almost the same location is in the level of " very severe damage". Those facts show that there is an evident difference in the damage rate following to the structural types.

Photo. 3.2.1.1 4 story building
Collapse by the shear fracture of the corner column of the first floor
The story collapse of the piloti of the first floor

Photo. 3.2.1.2 5 story apartment house building
Collapse of the first piloti floor

Photo. 3.2.1.3 8 story office building
The 6th floor was suffered form the story collapse.(the building behind is non damaged)

Photo. 3.2.1.4 3 story SRC structure governmental building
The shear fracture of the yield strength wall (resistant wall) of the 7th floor

Photo. 3.2.1.5 7 story RC structured apartment house
The story collapse of the second floor The number of the wall of the first floor is more than that on the upper floor.The shear fracture is seen on the column and the wall of the first floor.

Photo. 3.2.1.6 11 story SRC structured governmental building
Damage of the steel joint part of the peripheral column.(Fracture of the doubling plate of the joint part and the fracture of the main brace)

Photo. 3.2.1.7 11 story SRC structured governmental building
Damage of the steel joint part of the peripheral column

Photo. 3.2.1.8 4 story SRC structured office building
The half of the column at the bottom of the anchor plate of the base of column was slipped to the north.

Photo.3.2.1.9 7 story (PH 1st floor) RC structured company building
The shear fracture of the base of the column and the fracture of the foot of the earthquake resistant wall

Photo.3.2.1.10　Total view of the building

Photo. 3.2.1.13　Cracks of the base

Photo. 3.2.1.11　Transfer of the breast wall (about 15cm)

Photo. 3.2.1.14　Damage condition of the neighboring building (7 story frame structure)

Photo. 3.2.1.12　Shear fracture creeping to the corners of the open part of the building ; windows & doors.

Photo. 3.2.1.15　Damage condition of the neighboring building (7 story frame structure: different one from the one of Photo. 3.2.1.14)

6) Other Damages

The other damages than those stated above are as follows;

a. Overturning of the whole building (Photo. 3.2.1.16~19)

The whole of the building is overturned

b. Collapse caused by the twist response corresponding to the deviated wall

c. Failure between the different structures. (Photo. 3.2.1.20)

The building is constructed with SRC structure in the lower part and RC structure in the upper part. The failure is seen on the floor position where the structure is changed.

d. Fall and Overturning of penthouse (Photo. 3.2.1.21-3.2.1.23)

Some penthouses are fallen or overturned.

e. Story collapse influenced by a set back (Photo. 3.2.1.24-3.2.1.25)

The story collapse in the middle floor was caused by the twist response following to the gravity center of the building and the eccentricity of stiffness which was totally influenced by a set back.

f. Damage occurred by the collision of the buildings of next doors(Photo.3.2.1.26)

A part of the buildings are damaged by the collision of the buildings located in vicinity.

g. Collapse like pan cakes (Photo. 3.2.1.27,3.2.1.28)

All the layers of the building was collapsed, which is shaped like pan cakes.

h. Shear failure of columns (Photo.3.2.1.29-3.2.1.31)

The shear failures are seen not only in the columns which have definitely a small amount of shear reinforcement Photo. 3.2.1.30), but also in the columns with small interval of the shear failure reinforcement (Photo. 3.2.1.31).

i. Twist failure of components (Photo. 3.2.1.32)

A beam was broken by twisting.

j. Failures of the capital and the base of columns (Photo. 3.2.1.33)

The capital and the base of columns were fractured.

k. Shear failure of the joint of columns and beams (Photo. 3.2.1.34,3.2.1.35)

The joint parts of columns and beams of RC structure Photo. 3.2.1.34) and the parts of columns and beams of SRC structure (Photo.3.2.1.35) were suffered from shear failures.

l. Failure of columns by bending pressure (Photo. 3.2.1.36)

The failure caused by bending pressure was occurred on the concrete of the capital of columns.

m. Fracture of the gas pressed joint of the reinforced part (Photo. 3.2.1.37,3.2.1.38)

The gas pressure joint of the main brace of columns were fractured.

n. Adhesive segmentation cracking (Photo.3.2.1.39)

A column has a damage of adhesive segmentation cracking.

o. Adhesive segmentation cracking failure of a beam (Photo. 3.2.1.40)

The beam has a damage of adhesive segmentation cracking.

p. Damage of beams by (Photo.3.2.1.41,3.2.1.42)

The beams were shear fractured. because by the span ratio was small.


Photo.3.2.1.16 Overturning of the whole building


Photo. 3.2.1.17 Overturning of the whole building


Photo. 3.2.1.18 Overturning of the whole building


Photo. 3.2.1.19 Story collapse caused by the twist response


Photo.3.2.1.20 Fracture between the different structural types (SRC and RC)


Photo.3.2.1.21 Fall of a penthouse

Photo.3.2.1.22 Overturning of a penthouse

Photo. 3.2.1.23 Overturning of a penthouse

Photo. 3.2.1.24 Story collapse influenced by a set back

Photo. 3.2.1.25 Story collapse influenced by a set back

Photo. 3.2.1.26 Collapse caused by the collision of the buildings

Photo. 3.2.1.27 Collapse in the form of a pan cake

Photo. 3.2.1.28 Fracture in the form of a pan cake

Photo. 3.2.1.29 Shear fracture of a column

Photo. 3.2.1.30 Shear fractures of short columns

Photo. 3.2.1.31 Shear fracture of a column

Photo. 3.2.1.32 Twist fracture of a beam

Photo. 3.2.1.33 Fracture of the top and base of columns

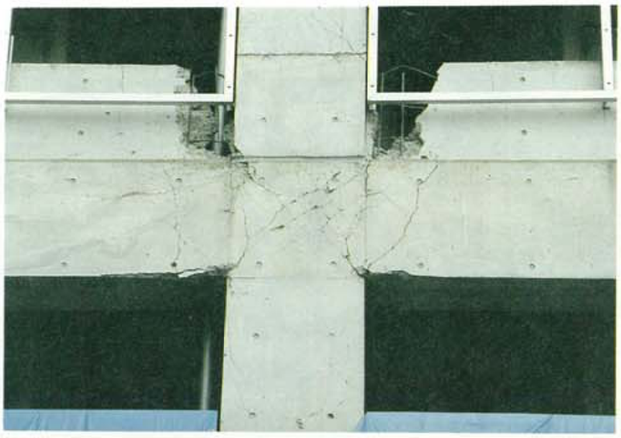


Photo. 3.2.1.34 Fracture at the beam-column joint part(RC)

Photo. 3.2.1.35 Shear fracture Beam-column joints(SRC)

Photo. 3.2.1.36 Flexural compressive failure of columns

Photo. 3.2.1.37 Fracture of the gas pressure welding of the reinforcements

Photo.3.2.1.38 Fracture of the gas pressure welding of the reinforcements

Photo. 3.2.1.39 Bond splitting failure of a column

Photo.3.2.1.40 Bond splitting failure of a beam

Photo. 3.2.1.41 Damage of beams mullion non-structural wall

Photo. 3.2.1.42 Damage of beams mullion non-structural wall

c) The Damage and the Characteristics from the Viewpoint of Design Standards

The design standard of the reinforced concrete structures of Japan was revised in 1971 and 1981.

If the reinforced concrete structures are divided into three groups such as the ones constructed before 1971, the ones between 1971 and 1981 and the others after 1981 according to the standard, it can be clarified by the macro analysis of this report that each damage ratio among the three groups is outstandingly different.

The result is as follows:

1. The damage of the buildings constructed before 1971 is quite severe.
2. The damage of the buildings constructed after 1971 is small. Especially the ones made after 1981 hardly show any big damage except the buildings like piloti buildings which have special characteristics. (Please refer to (3) "The Damage Investigation of the Buildings According to the Present Standard.")

As seen from the result, since there is quite a good correlations between the damage ratio of the buildings and the design standard, the design shear force of the standard was studied here.

In the old standard established before 1981, the horizontal seismic coefficient (please refer to the formula (1).) was defined and the allowable stress design was required for working stress.

In the existing standard, the external force defined in the formula (2) is provided and the allowable stress design is required for so called the middle level earthquake motion.

For the large scale seismic motion, the confirmation of ultimate shear capacity equivalent to factor in the formula (3) is required.

The horizontal seismic coefficient under the old standard;

$$K = 0.2 \qquad (1)$$

However, for the floors in the height of more than 16 m,0.01 should be added for each 4 m.

The design story seismic shear coefficient under the present standard (for the middle level earthquake);

$$C = Z R_t A_i C_o = A_i C_o = 0.2 A_i \qquad (2)$$

Here,

C_i : story seismic shear coefficient

Z : seismic zone factor (1.0)

R_t : dynamic property factor (1.0)

A_i : distribution factor of story seismic shear force coefficient

C_o : standard seismic shear coefficient (0.2)

The required story shear coefficient under the present standard (for large level earthquake motion) ;

$$C_i = F_{es} D_s Z R_t A_i C_o = 0.3 Ai C_o = 0.3A_i \qquad (3)$$

In here, the following marks are symbolized as :

F_{es} : configuration factor (1.0 - 2.25, 1.0 is provided here)

D_s : structural property factor (0.3 - 0.55, 0.3 is provided here)

C_0 : standard seismic shear coefficient (1.0)

The horizontal seismic coefficient of the formula (1) is equivalent to the story shear coefficient in the buildings of less than 16 m in height, but is different in the buildings of more than 16 m in height. However, the difference is very small.

In the following discussion ,It is assumed that the weight is the same in each floor and the height is three meters in each floor.

Figure 3.2.1.1 shows the story shear coefficient used by the formula (1), (2) and (3) ,and the story shear coefficient which is 1.5 times of the rates of the formula (1).

Comparing the formula (1) and (2) from the view point that the same allowable stress design is applied,the shear force applied to the buildings by the old standard are quite small especially on the upper floors, compared with the one applied to buildings by the present code.

Moreover, as shown in the Figure, if the buildings based on the old code only have the yield strength of 1.5 times of the design shear coefficient at the time of the damage,the higher than floors are, the less the shear capacity of them are compared with the one required in the present code.

However, the upper floors have a possibility to exceed the requirement of the present standard according to the structural minimum requirements which is shown in the chapter 3.

Even in the old standard, the regulation for shear design is different between the one before 1971 and the one after the year.

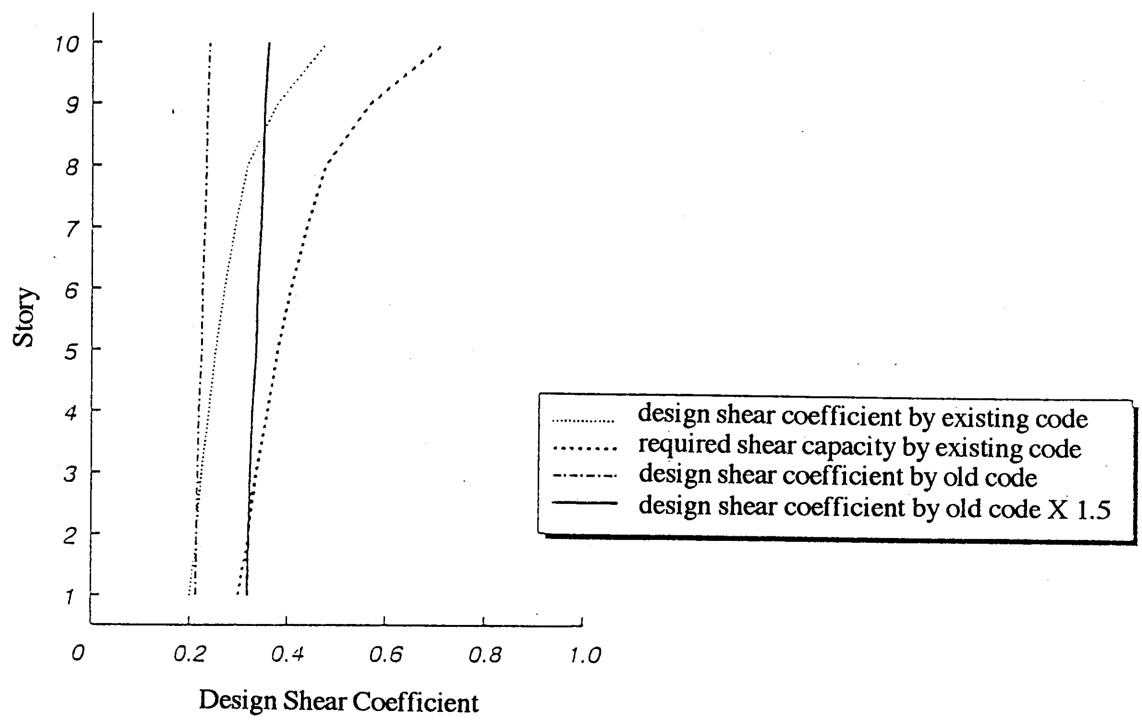


Fig. 3.2.1.1 Design Story Shear Coefficient in the Old and the Present Standards

Namely, the followings are seen in the revised standard established in 1971 and the revised RC standard established by Architectural Institute of Japan :

1)To avoid shear fracture, a regulation based on the concept of the flexural yield preceding was incorporated. In case the flexural yielding mechanism is not guaranteed, the design force for shear reinforcement should be more than 1.5 times of the rate regulated in the standard before 1971.

2) The minimum space of lateral reinforcement was decided as about one third of that stated in the standard before. The effect of concrete shear resistance was also decreased to about three forth of the one stated in the standard before 1971.

It is clarified that the buildings constructed under the standard established by Architectural Institute of Japan (AIJ) after 1971 have the maximum capacity against shear force of 1.5 times more than that of the buildings before 1971, or they are required the structural performance equivalent to the maximum shearing force.

These drastic change of shear design inevitably cause to the expansion of the cross section of members, which results in enhancing the bending capacity. The change to beam yielding mechanism also realized to reduce the damage of the buildings constructed after 1971.

(2) Detailed Case Study of Damaged Buildings

a) Objective

Aiming for recording and keeping the objective references of the damage condition, the detailed analysis of damaged buildings was carried out mainly on the public buildings and especially on RC buildings was done.

The investigation of this time was done as a part of the investigation of the Survey Committee of Earthquake Damaged Buildings of the Ministry of Construction (Chairman: Mr koichi Kishitani) by the collaboration of the Steering Committee of Reinforced Concrete Structures of the AIJ and the Building Research Institute of the Ministry of Construction. The investigation was carried out also with the cooperation of the Japan Structural Consultants Association.

Along with the detailed investigation, the statistical investigation of the collapsed buildings was also done in the Chuo-ku (Central District) of Kobe City.

The investigation was done during from the 16th to the 19th of February, 1995.

b) List of Buildings for Detailed Investigation

The objectives of the detailed investigation are 20 buildings shown in the Table 3.2.1.1 and Fig. 3.2.1.2, numbering from 1 to 20.

Some cases include more than two buildings in one investigation subject.

Seen from the locating as shown in Fig.3.2.1.2, those buildings are classified like 13 cases in Kobe City, 4 cases in Ashiya City and 3 cases in Nishinomiya City. Each year of the completion is in the period of 1957 and 1993; 13 cases were completed before '71, 3 cases were done after '71 but before '81, 3 cases after '81 and 1 case of unidentified year of completion.

Followed by the structure, there are 9 cases of reinforced concrete structure and 11 cases of steel framed reinforced concrete structure.

According to the damage degree, there are 9 cases with some collapsed buildings.They are 5 story collapsed buildings on the middle floor out of them, 3 cases with the story collapse on the first floor and 1 case of the fall of PC roof board. There are 6 cases with the buildings of heavy damage, 4 cases with them of medium level damage, 5 cases of them of minor damage or slight damage and 1 case of which damage degree is unidentified.Some building with a structural piloti to be utilized as a parking space or retail shops on the first floor is also included in the investigation. Five cases of that kind building are reported here.

Table 3.2.1.1 List of the Detailed Investigation Buildings

No	place	completion	struct. type	story	damage grade	others
1	Kobe,Chuo-ku,Kanouchou	1957	SRC	8	collapse	
2	Kobe ,Chuo-ku,ichiban-chou	1969 (1977 ext.)	RC	8	collapse	
3	Koube,Chuo-k u,Kitahonmachi Ave.	1973	SRC	12	collapse	piloti
4	Kobe,Chuo-ku, Nakayamate Ave.	1969	SRC	10	unknown	piloti
5	Kobe,Chuo-ku,Nishimachi	1965	SRC	8	collapse	
6	Kobe,Chuo-ku,Kyoumachi	1978	SRC	10	medium	
7	Nishinomiya,Rokutanji	1971	SRC	8	severe	
8	Nishinomiya,Rokutanji	1967	RC	6	medium	
9	Nishinomiya,Hazezuka	1966	RC+SRC	5	collapse	piloti
10	Kobe,Higashinad a,motoyamaminami	1971	RC(No1) (No2,3)	11	collapse medium	
11	Kobe,Higashinad a,motoyamaminami	1971	SRC	14	medium	
12	Kobe,Higashinad a,motoyamaminami	1971	SRC	14	severe	
13	Kobe,Higashinad a,motoyamaminami	1971	SRC	11	severe	
14	Kobe,Chuo-ku,Dainichi Ave.	1964	SRC	6	severe medium slight	
15	Asiya,Shoudou	1960 1991 (repair)	RC	4	slight	
16	Asiya,Narihira	unknown	RC	4	partially severe	
17	Asiya,kamimiyagawa	after 1981	RC	8	slight	piloti
18	Asiya,Kawanishi	1972	RC	3	part collapse	
19	Kobe,Higashinad a,motoyamaminami	1986	RC	8	collapse	piloti
20	Kobe,Higashinada,Mikage	1993	RC	5	collapse	piloti

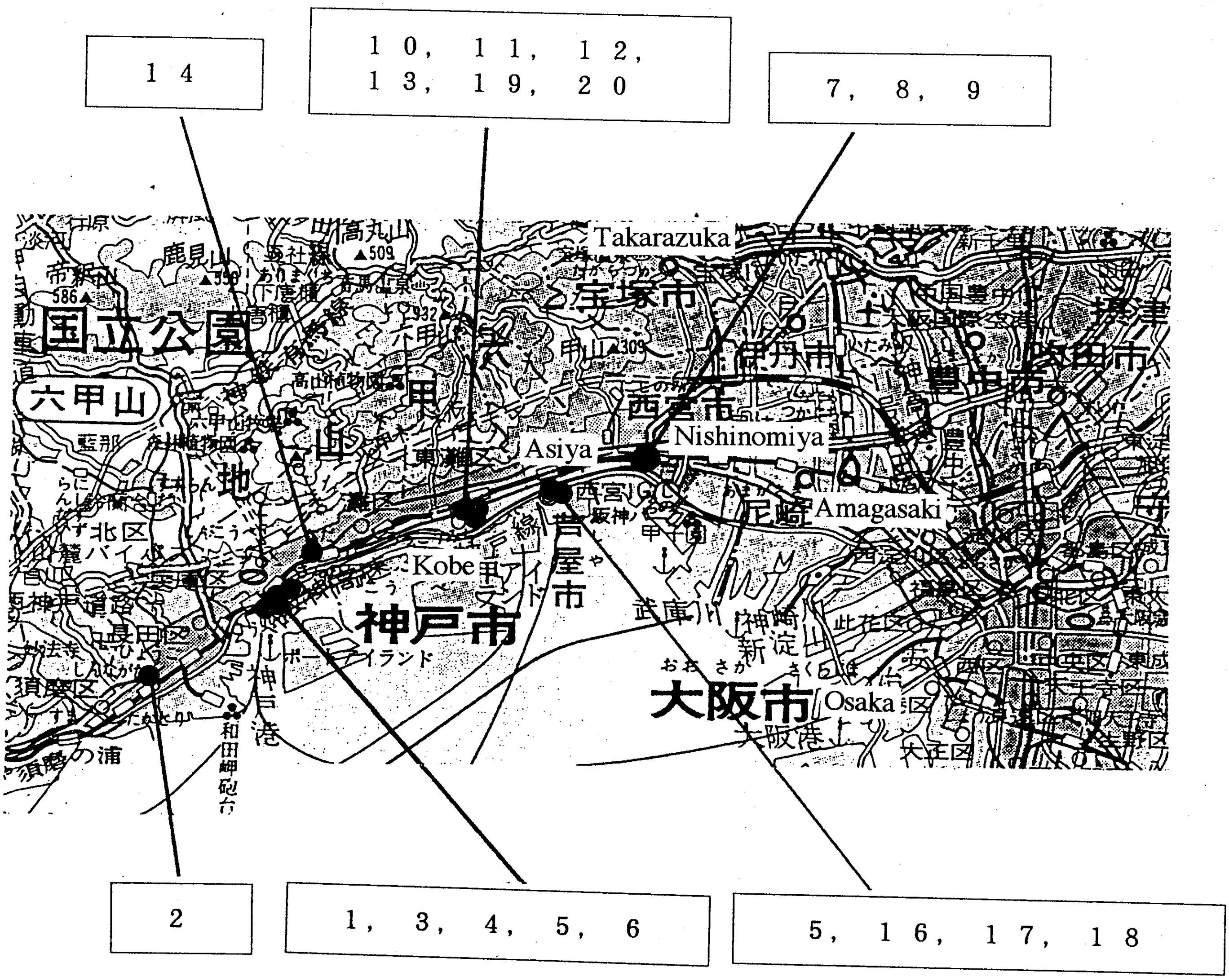


Fig. 3.2.1.2 Locations of the Buildings for Detailed Investigation

c) A Case Study of the Investigation

The outlines of the investigation is published as "The Detailed Damage Investigation of the Reinforced Concrete Structures by the Southern Hyogo prefecture Earthquake (Promt News,by the Earthquake Disaster Investigation Committee). Among them, a eight story office building is reported here in detail. (Refer to the building with the No 1 in the Table 3.2.1.1).

1) Outline of the Building

plan configuration :	a clear rectangle of 94.6m x 26.0m		
elevated configuration :	8 story on the ground,1 story under the ground 4 story penthouse		
height of eaves :	30.4m		
maximum height :	46.65m		
building area :	2.674m2		
total floor area :	23.848m2		
type of the structure:	frame structure with bearing wall (Refer to Fig.3.2.1.3)		
	SRC structure (basement to the 5th floor)		
	RC structure(6th floor to the roof floor)		
basement :	1150 x 1150mm	1st - 2nd floor :	1000 x 1000mm
3rd - 4th floor :	850 x 850mm	5th floor :	850 x 850mm
6th - 7th floor :	650 x 650mm	8th floor :	600 x 600mm

reinforcement: plain bars both for the longitudinal and shear reinforcements,space of each column hoop is 300mm. gas pressure welding was used for the joint of the longitudinal reinforcement.

steel material : unidentified design: 1954 Year of completion : April, 1957

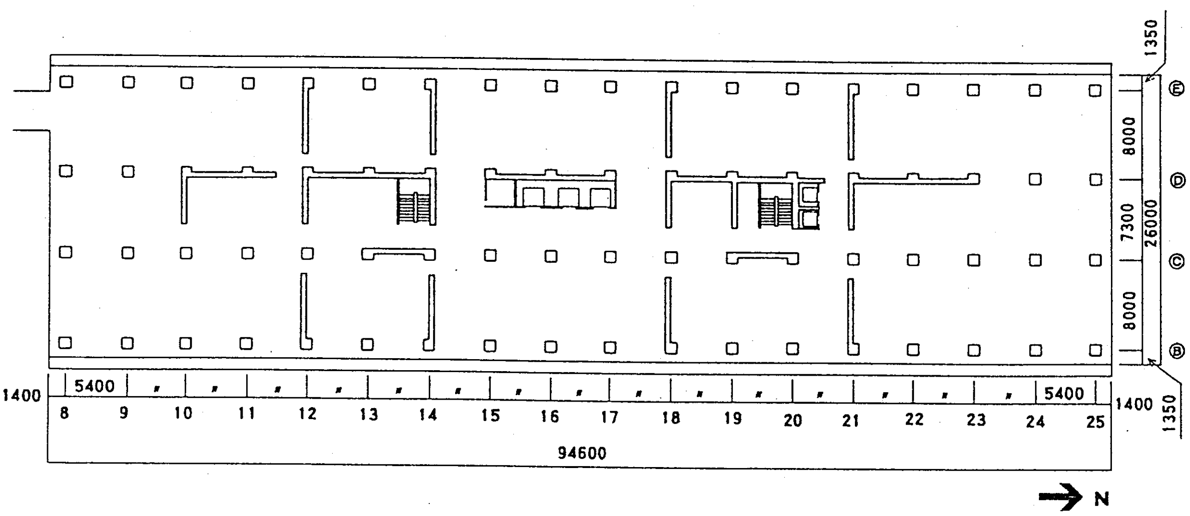


Fig. 3.2.1.3 Plan of the 1 st Floor

2) Summary of the Damage

The general characteristic of the damage is story collapse of the 6th floor. The upper floors from the 7th floor are fallen mainly to the north, but they are also slightly pushed out to the east. (Photo. 3.2.1.43)
The damage condition of the columns ,the shear walls of each floor and the result of the damage grade are stated below. The names of the planes follow the ones shown in the Fig. 3.2.1.44.
In the basement, only one column connection to the shear wall side of D-plane of structure caused the buckling of the main reinforcement at column base and covering concrete was spitted out . A part of the steel is exposed but the damage of the joint is not observed. (Photo. 3.2.1.44). The damages of the other columns and shear walls are comparatively small.

- On the 1st floor,columns and shear wall as damage grade III - IV are partially observed in the bay direction .
- On the 2nd floor, large shear cracks are seen on the shear wall and the side columns of the middle plane of structure. On one of the side columns, a horizontal fracture is seen in the height around 140 - 150 cm from the slab surface, and the compression failure of concrete and the buckling of main reinforcement are occurred. There, steel joint plate inside the column is exposed. (Photo. 3.2.1.45)
- On the 3rd and the 4th, there can be seen some of columns of the damage degree III, but the damage degree of the columns outside is I - II.
- On the 5th floor, though the columns of the degree V and a column of the degree VI caused by shear failure are observed, the degree of the shear wall remains in III. (Photo. 3.2.1.46)
- The 6th floor totally collapsed and beams attached to the floor slab. There is only the space of the height of a beam remained.
- On the 7th floor, there are some places where the collapsed columns and the shear wall of the 6th floor are exposed and broken through from the slab of the 7th floor. The damage degree for a side column of the bearing wall is evaluated as IV, the other columns are comparatively small. (Photo. 3.2.1.48)
- The 8th floor has also a comparatively small damage on the columns as well as the 7th floor.

3) Diagnosis and Views

As the structural characteristics of the story collapsed 6th floor, it is recognized by the human eye observation that the 6th and the 7th floor have less main column reinforcement of 65 cm square and 8 bars of diameter 22 mm and the structure of them is changed into RC structure without steel frame, while the lower half of the columns of the 5th floor is SRC structure of 75 cm square and with 20 bars of diameter 25 mm of the main reinforcement. The ratio of the main reinforcement of the columns of the 6th floor is 0.8%, which is equivalent to the ratio of the minimum reinforcement specified in the old standard and it is about one forth value of the main reinforcement of the columns of the 5th floor. Since the strength ratio to the supporting weight of the 6th floor is presumed relatively smaller compared with those of the upper and lower floors,it is considered that the damage was concentrated in the weaker 6th floor by the large seismic motion.

The damage of the column of the 2nd floor (Photo. 3.2.1.45) is caused by the connection of the joint, because the damaged position is corresponding to the steel joint part. The damage process is presumed as follows:

The columns was received a tensile force as the attachment column of the shear wall.By the tensile force, the joint part was damaged. After the longitudinal reinforcement of the column was subjected to plastic deformation, they had buckling to occur the drop out of the covering concrete.

The column of the basement (Photo. 3.2.1.44) is also in the same condition:

It had a tensile force as the attachment column of the shear wall. The anchor of steel frame at the base of column was damaged and the longitudinal reinforcement of the column had a tensile yield, which then lead the buckling of the longitudinal column reinforcement by the subsequent compressive force.

It can be supposed that because of those damages, the continuous shear walls as one of the important earthquake resistant elements lose the strength, and consequently, the ultimate shear capacity of the 6th floor which is originally small, decreased much more. It is also assumed that the damage degree IV & V on the columns of the 5th floor was caused by the collapse of the 6th floor.

Photo.3.2.1.43 Total View ; The story collapse of the 6th floor

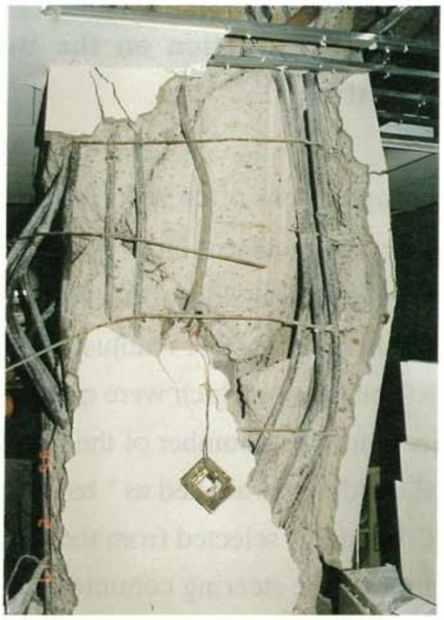


Photo. 3.2.1.46 Column of the 5th floor

Photo.3.2.1.44 Basement ;Side column of the bearing wall column reinforcement

Photo.3.2.1.47 Floor slab of the 7th floor Column

Photo.3.2.1.45 Bearing wall of the 2nd floor and the side column shear cracks ,buckling of the main column reinforcement

Photo. 3.2.1.48 Damage condition of the 7th roof floor of the 6th floor is pushed out to the 7th floor

(3) Damage Investigation on the Buildings Based on the Present Code

a) Objective

The damages were often seen in the buildings designed by the old code, but the damage in the buildings by new code was also reported.
This investigation was done to leave the references of the damage condition of the buildings by new code and to help the further readjustment of the present design code. The subject of the investigation was the RC buildings, which were quite damaged,designed based on the present code .
The subjects are 43 in number of the following buildings:(Table 3.2.1.2)
- The RC buildings evaluated as " ban of use" by Kobe City
- The RC buildings selected from the data as of the 20th of April, 1995 based on the result of the joint investigation of the steering committee of RC structures of AIJ and the Building Research Institute of the Ministry of Construction, which was conducted from the 16th to the 19th of February, 1995.

b) Investigated Buildings

The investigated buildings are shown in the Table 3.2.1.2, numbered from 1 to 43.
The buildings except No. 5 and No.6 were diagnosed as clearly dangerous buildings at the first stage of the emergency risk assessment work done from the 18th to the 22 nd of January, 1995 and treated as the ones of the ban of use. After that, a follow-up investigation for those 43 buildings was carried out by the Building Research Institute, the Ministry of Construction. Through the investigation, the following damages were found: 16 buildings of more than severe damage (Two out of them were damaged by collision) and 21 buildings of less than the medium damage(One out of them was reported the foundation damage). There are 6 buildings constructed before the establishment of the present design code.
The building locations of the 35 buildings of except the buildings of collision damage and the buildings designed by old code are as follows:
8 in Higashinada-ward (4 buildings of severe damage are reported), 13 in Nada-ward (3 of severe damage), 6 in Hyogo-ward (3 of severe damage), 2 in Nagata-ward and 2 in Ashiya city. Among them ,the number of reinforced concrete buildings is 23 (11 of severe damage), that of steel reinforced concrete buildings is 8 (3 of severe damage) and 4 unidentified structures are reported.
Among 14 buildings of more than severe damage, 9 buildings are with piloti. This kind of structures are all reinforced concrete structures and their number of story is from 4 story to 10 story. The number of span of the damage direction is basically 1 span. Out of the severe damaged ones, collapsed 5 buildings are all with piloties. In other 5 buildings, 3 are damaged at the anchor of the base of column or the structure changing floor from SRC to RC , one building is damaged by the inadequacy of the steel joint part and one is damaged by torsion because the building is located in the corner and the structural walls are unevenly distributed.

Table 3.2.1.2 Third Supplementary Investigation (Severe Damaged Buildings before 1981)

Part 1 Severe Damage and Collapse

No.	dam.grade	address	year	str./story	span num.	dam. mode	note
1	collapse	Higashinada	'86	RC-8	3x5	shear fail,subside	piloti
2	collapse	Nada,shinzaike	'86	RC-7	2x3	col. crash,bar buckle,slope1/20	piloti
3	collapse	Higashinada	'90	RC-5	1x1	col. bend crush	piloti
4	collapse	Chuou,wakana	'88	RC-10	1x1	col shear crush	piloti
5	collapse	Nada,Yamato	'93	RC-4	1x2	Col com. crush	piloti
6	collapse	Hyougo	'93	RC-14	1x5	3Fl wall ,joint	Bad joint
7	sev.dam.	Higashinada	'88	RC-5		Col shear ,torsion	Ec. wall
8	sev.dam.	Chuou,Tutui	'89	SRC-7		Wall,col bend crush	SRC-RC
9	sev.dam.	Hyougo	'86	RC-3-7	1x6	Col. crush	piloti
10	sev.dam.	Higashinada	'85	RC-6	1x8	Col crush	piloti
11	sev.dam.	Hyogo,Ekimae	'88	RC-10	1x6	Bar buckle	piloti
12	sev.dam.	Chuou,Edo	'84	SRC-9		Anchor ,bar buckle	SRC anc.
13	sev.dam.	Nada,Ohishi	'85	RC-6	1x3	bend crush	piloti
14	sev.dam.	Chuou,Yamate	'92	SRC-9		base plate slide	SRC anc.

Part 2 Severe Damage by Collapse of Adjoining Building

15	collapse	Chuo,Kitanagasa	'86	RC-6	1x3	1F col. crush	pencil bldg
16	Part seve.	Higashinada	'82	RC-4		1F col crush	pencil bldg

Part 3 Medium and Small Damage

17	medium	Higashinada	'94	RC-5	2x11	col. comp.crush	piloti
18	medium	Higashinada	'85	RC-9,B1	1x4	col.comp.crush	piloti 1f
19	medium	Hyogo	'83	RC-7	1x5	unknown	piloti 1-3f
20	medium	Nada,Sakuraguci	'91	RC-10,B1	2x3	col.bond split.	1F piloti
21	medium	Nagata,Matuno	'84	RC-7	1x5	col.bar buckle	1F piloti
22	medium	Higashinada	'87	RC-7-10		corn.col. crush	
23	medium	Nada,Yumiki	'85	RC-8	1x6	beam yld ,jnt crack	duct frm
24	medium	Nada,Kusugaoka	'91	HFW-10	1x7	beam yield	Wall frm
25	medium	Nagata,Matuno	u.c.	RC-9	2x3	beam yld,col.crack	1-3 piloti
26	medium	Nada,Nagate	'88	SRC-12		3Fl wall crack	set back
27	medium	Nada,Sakura	'85	RC-5		1 col. Bend rupt.	1f piloti
28	medium	Nada,Yamato	'87	RC-11		b-c joint beam crk	
29	medium	Nada,Biwa	'82	RC-8		1 col bar buckle	piloti
30	medium	Asiya,Ohhara	'86	SRC-6		1 col bar buckle	1f shop
31	small	Hyogo,Irie	'85	RC-9	1x5	1 col bar buckle	piloti
32	small	Hyogo	'93	SRC-12,B1	6x7	B1 col bar buckle	B1 park
33	small	Higashinada	-	RC-8	2x2	found dam.	Prt piloti
34	small	Nada,Kishichi	'92	SRC-7		unknown	
35	small	Nada,Iwaokita	'90	SRC-14		ext.wall shear fail	
36	slight	Asiya,Shiomi	-			soil rupt. Bldg.incl.	
37	none	Nada,Funadera	'91	RC-4		unknown	

Part 4 Buildings Before Existing Design Code

38	fall down	Higashinada	'73	RC-6	L shp	1 col. Shear crush	piloti
39	fall down	Higashinada	*	RC-6	1x	1 col. Shear crush	piloti
40	fall down	Higashinada	'80	RC-4,B1	1x2	B1 col. Shear crush	b1 piloti
41	medium	Nada,Tomoda	'79	RC-8	1x2	1F col. Bond splt	piloti
42	medium	Chuo,Yamate	*	RC-3	1x	hp plain, col bend rpt	1f shop
43	severe	Asiya	*	RC-7		Hp plain col shr rpt	piloti

Note: 1) * means that the year of completion (before or after '81) is assumed by the type of reinforcement, that of hoop and its space.

2) Definition of the terms in the column of "Structural characteristics"

piloti : an open space mainly used for a parking space with walls at the upper floors

piloti shp: open frame such as for shops and upper rigid frame

c) A Case Study of the Investigation

The damage report of a 8 story apartment house (piloti: No.1 of Table 3.2.1.2) is stated below.

1) Outline of the Building

plan configuration:
3 block of 5.6 (6.1)m x 11.1 m, which plan is irregularly shape (as shown in Table 3.2.1.3) with each interval of 3.7 m from sough to north, a balcony and a corridor of cantilever slab, a connecting corridor attached to the building of next door, and a stair well set on a part of the 2nd floor.(note Table 3.2.1.3)
elevated configuration:
8 story on the ground, no basement, no penthouse, a set back with a span on the 8th floor, and a pillti of the 1st floor.(a shear wall from south to north)

height of eaves :	22.35 m
maximum height :	22.95 m
building area :	about 200 m2
total floor area :	about 1300 m2 type of structure :

size of columns:
1st floor 800 x 800 mm, 2nd floor 700x700(600)mm
5th floor 600 x 600 mm, 8th floor 550 x 550(600)mm

reinforcement:	
main reinforcement;	D19, D22, D25 (SD345) gas pressed joint
hoop :	D10, D13 (SD295A) flashbat weld (square shaped spiral)
concrete :	Fc240(1.2F), Fc225(3,4,5F), Fc210(6,7,8F), usual concrete
year of designing :	February, 1985
year of completion :	March, 1986

2) Summary of the Damage

The collapse was occurred in the west side of the 1st story to make the whole building inclined 4 degrees to the west. (Photo.3.2.1.49). The damage is concentrated in the first story and is very small at the higher story, which is like shear cracks, are only seen on the wall. (Photo 3.2.1.50) Connection corridors (pin connected at the main building side and the pin-roller connected at the next building side) of all stories are fallen down. (Photo. 3.2.1.51)
Almost all the columns of the first story at the west side suffered shear failure and are easily fractured.

(Photo. 3.2.1.52) The 4-frame of the east side which are not tightly connected to the 2nd story with a stair well and the column of the A-frame of the south side occurred the bending failure at the top and bottom of column without story collapse. (Photo 3.2.1.53) The column occurred bending failure on the 2nd floor too. In south-north direction, a shear wall is arranged from the top story to the 1st story and it is supposed that the such shear wall responsed heavily from east to west, where a complete piloti is composed. In the shear wall, any big shear cracks are not recognized, this wall is bent in the out-of-plane direction to be fractured. (Photo 3.2.1.54) The fracture at the gas pressure welded joint of the main reinforcement is observed (Photo 3.2.1.55), but the number is not so big as imagined. The fracture at the flare welding part of the hoop is also observed, but its number is also small and such fracture is occurred in the other parts than the joint part. (Photo 3.2.1.56) The hoop of the column is rather densely arranged, but the inside tie hoop is not placed, which caused the concrete failure after the shear failure and the difficulty of axial force sustainability.

There are three story RC apartment housing(9-11 story) in the same site, which were designed and constructed at the same time. They had rather much damage only in the non-structural walls. (Photo 3.2.1.57) Only the main building of which 1st story was constructed as piloti had unrecoverable damage.

3) Discussions

Since this building was designed in accordance with the present building design code (so called "New Seismic Design Standard"), it is very important to investigate the reasons of the collapse for evaluating the required seismic resistance performance of the present building code. Therefore, it needs a sufficient discussion afterwards, however only a brief analysis is reported here.

a) Investigated Points

The building showed story collapse of the first story and so, the following points are stated below:

i) analytical estimation of the failure mode of the first story column and the comparison of the actual damage conditions

ii) The approximate evaluation of the shear carrying capacity of the first story.

b) Analytical hypothesis

The analytical hypothesis to investigate the above two points are as follows:

i) The final bending strength of the column and the shear failure strength are based on the ultimate bending strength conventional calculation formula and revised Arakawa formula for shear capacity in the "Structural Regulations of Buildings" published by Japan Building Center, which were widely and mostly used at the time for design of the buildings and are still widely and mostly used in Japan.

For the comparison, the investigation using ACI standard formula was also carried out. The details of the formula should be referred in Table 3.2.1.3.

ii) For the material strength, the closest value to the actual value is assumed, because the result of the concrete material test has not been done yet. The details should be referred to the Table 3.2.1.3.

iii) Here, the axial force of the column is only considered as permanent load and the change of the axial force caused by horizontal and vertical motion of the earthquake is ignored. For the calculation of the vertical load, the assumed value of both general story and penthouse is 1.2tonf/m2 (including the weight column, wall, etc.).
iv) The shear force of the column to reach the flexural yielding is acquired by the following: The sum of the ultimate flexural moment of the column top and column base is divided by the inner height the column.
v) The column shear force at the time of horizontal ultimate stage is taken the smaller one, between the shear force at the time of the ultimate bending strength or the shear failure capacity acquired by the revised Arakawa formula. The calculated shear capacity by ACI standard formula is considered as reference value.
vi) For the calculation of the horizontal strength, the simple sum of the shear force of all the 1st floor columns which was acquired in the previous assumption.(v). It means that the individual crashed damage caused by the lack of ductility and the torsion of structure are ignored.

c) Consideration

The Result is presented in Table 3.2.1.3,Part 2.The followings are found:
i) The failure mode used by the values of the approximate calculation formula of the ultimate bending stress and the revised Arakawa formula for shear capacity is as follows:

- Four inner columns (CB3, CC2, CD3, CE2) have shear failure.
- The other 8 outer columns have bending failure.

This is the same as the case used by the calculated value of ACI standard formula.
ii) The actual failure mode is equivalent to the analytical failure mode except the columns: CC1, CF1, CB2, and CF2. The analytical failure mode of these columns are bending failure, but the actually shear failure. The reason of the difference between the analytical mode and actual mode is presumed on the influence of the column axial force. It means as follows: CC1, CF1 and CB2 are the columns in which the axial force by overturning moment is in compression side when the horizontal force works to the direction of collapse of the buildings.Therefore, the ultimate flexural capacity increases, and shear failure occurred. Though the increase of shear capacity by additional axial force to such columns can be considered, that of the ultimate bending shear capacity was seemed to be more increased.
For the column CF2, those considerations are not available, but stairs are attached to the column CF2, which caused some influence to the total result.
iii) The horizontal shear resistance is 0.80 in base shear coefficient. (The hypothesis used for the analysis to acquire the value should be carefully discussed afterwards).The value of 0.80 is never so big value judging from the severe condition like the collapse of this building. Because the ductility factor of the column can be estimated as small (because of 8 columns' shear failure out of 12 columns and 0.45 which is the maximum value of the frame structure is assumed as D_S value) and since the hinge mechanism of the buildings is as the one which should avoid the concentration of the plastic

deformation on the 1st story (The maximum configuration factor by the stiffness ratio, 1.5 is also assumed), the required shear force carrying capacity factor is 0.675 (0.45 x 1.5) and the horizontal base shear coefficient ; 0.80 is about 1.2 times of that.

For the torsion of the buildings, the twisted deformation after the collapse is not so outstanding. However, the influence of the torsion should be discussed more for the detailed process of collapse.

Photo. 3.2.1.49 South Facade of 8 story Apartment Housing

Photo. 3.2.1.50 North Facade ,Same Bldg.

Photo. 3.2.1.51 Roofed Passage

Photo. 3.2.1.52 Shear Fructure of 1 FL Column

Photo. 3.2.1.53 Flexural Fructure (1FL)

Photo. 3.2.1.54 Fracture of Shear Wall

Photo. 3.2.1.55 Shear Fructure of Column(1FL,C_{D3})
Rupture of Gass Pressure Welding of
Longitudinal Bars

Photo. 3.2.1.56 Shear Crush of Column
(1FL)

Photo. 3.2.1.57 South Facade of 5 Story Building

Table 3. 2. 1. 3. Data for the Calculation of Ultimate Force Carrying Capacity of First Floor
Part. 1

Estimation of Ultimate Base Shear Coefficient

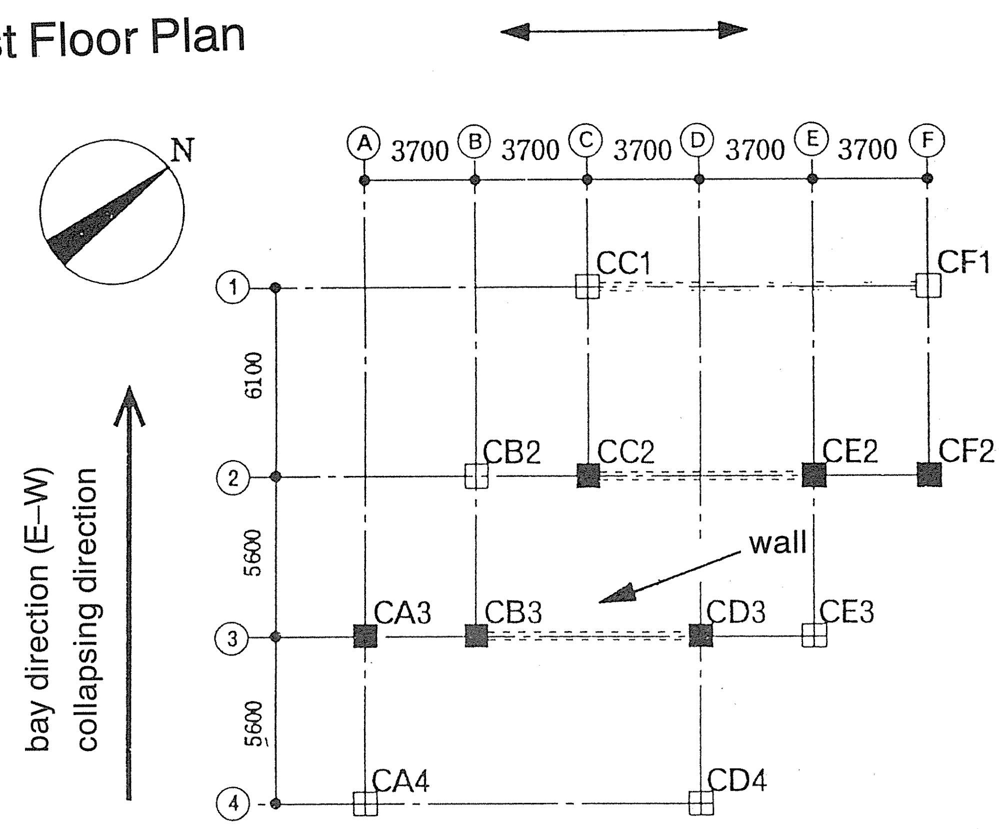
Total weight (1FL) W =1695.8 (tonf)

Total horizontal capacity Q = 1347.4 (tonf)

$$C^B = Q / W$$
$$= 0.80$$

Span direction (N–S)

1st Floor Plan



List of Column Sections

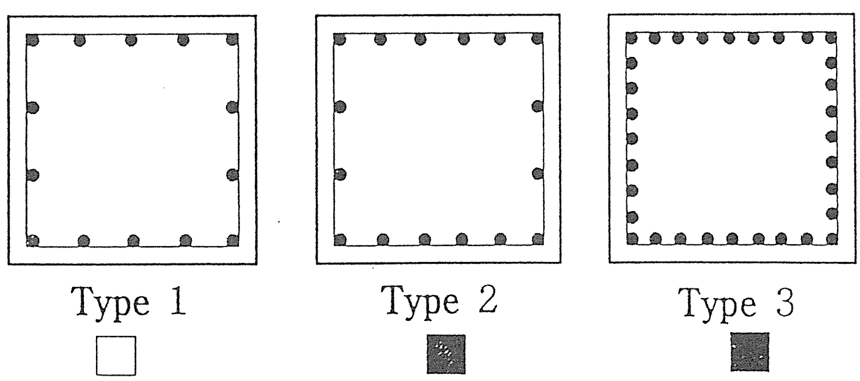


Table 3. 2. 1. 3. Data for the Calculation of Ultimate Force Carrying Capacity of First Floor Part. 2

◎ materials (assumed)

concrete : FC=240kgf/cm^2 main bars : SD35 (fsy=4000kgf/cm^2)

hoop : SD30 (fhy=3500kgf/cm^2)

◎ axial force (w=1.2tf/m^2 assumed)

column	floor area (m^2)	N (tf)	total N (tf)	num. of story
CA3,CD3	5.18	6.22	49.8	8
CA4,CD4	15.54	18.65	149.2	8
CB2,CE3	5.18	6.22	49.8	8
CB3,CD3	23.31	27.97	224.0	8
CC1,CF1	16.93	20.32	142.4	7
CC2,CE2	24.24	29.09	232.7	8

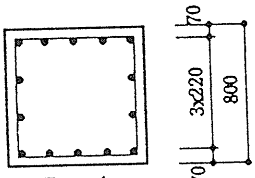
◎ column section

TYPE 1 CA4,CB2,CC1,CD4,CE3,CF1 (h=2600mm)

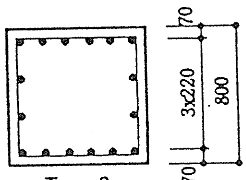
main bar : 14-D25 (p_g =1.11%)

hoop : D13 @ 100

(p_w =0.32%、 p_h =0.73%)



Type 1



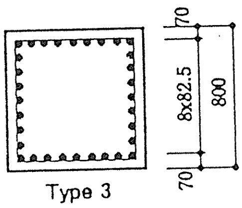
Type 2

TYPE 2 CA3,CF2 (h=2600mm)

main bar : 16-D25 (p_g =1.27%)

hoop : D13 @ 100

(p_w =0.32%、 p_h =0.73%)



Type 3

TYPE 3 CB3,CC2,CD3,CE2 (h=2600mm)

main bar : 32-D25 (p_g =2.54%)

hoop : D13 @ 80

(p_w =0.40%、 p_h =0.91%)

◎ result of calc. (axial force for ultimate flexural moment is only that of permanent boad)

Columns	TYPE	N (tf)	N/fc′	M_Y	Qn
CA3, CF2	2	49.8	0.03	83.4	132.0
CA4, CD4	1	149.2	0.10	99.7	136.6
CB2, CE3	1	49.8	0.03	73.1	128.6
CB3, CD3	3	224.0	0.15	(190.5)	(159.4)
CC1, CF1	1	142.4	0.09	98.0	136.0
CC2, CE2	3	232.7	0.15	(192.4)	(160.1)

1) Calc. Ult. Moment (by AIJ)

$$M_u = \begin{cases} 0.5a_g f_{sy} g_1 D + 0.5N g_1 D, & 0 > N \leq N_{min} \\ 0.5a_g f_{sy} g_1 D + 0.5ND(1 - \dfrac{N}{bDf_c'}), & N_b \geq N \geq 0 \\ \{0.5a_g f_{sy} g_1 D + 0.024(1+g_1)(3.6-g_1)bD^2 f_c'\}(\dfrac{N_{max}-N}{N_{max}-N_b}), & N_{max} \geq N \geq N_b \end{cases}$$

2) Calc. Shear Failure Capacity(revised Arakawa′s equation)

$$Q_u = \left\{k_u k_p \frac{0.115(f_c'+180)}{a/D+0.12} + 2.7\sqrt{\rho_w f_{wy}} + 0.1\frac{N}{BD}\right\} Bj$$

(4) Study on the Damage Factors

The study on the factors of the damage patterns is stated below:

a) Collapse of the Medium Story

1) Estimation of the Ultimate Lateral Strength of Buildings by Old Code

Since the damage is story collapse , the ultimate story lateral strength is discussed as the sum of columns capacity. As columns of the upper and lower stories from the collapsed story only have slight damages, and so it is assumed the shear failure capacity of columns are larger than that of flexural yield capacity.

This assumption is quite appropriate to the columns of upper stories where longitudinal reinforcement ratio and axial force ratio are small. The flexural yield moment of columns are given as the following formula;

$$Mu = 0.8\, a_t\ \sigma_y D + 0.5ND(1\text{-}N/\ b\, D\, \sigma_B) \qquad (4)$$

M_u : flexural yield moment of a column (tfcm)

a_t : tensile reinforcement amount of a column

σ_y : material strength of tensile reinforcement (tf)

N : axial stress of a column (tf/cm2)

σ_B : concrete strength (tf/cm2)

b : width of a column (cm)

D : depth of a column (cm)

As $N/bD\sigma_B$ is about 0 - 0.2 and about 0.1 in the middle story, the formula (4) can be simplified like the formula (5) as follows :

$$Mu = 0.8\, a_t\, \sigma_y\, D + 0.45ND \qquad (5)$$

The shear force of a column at the time of flexural yield is stated as the formula (6):

$$Qm = 2Mu / ho \qquad (6)$$

where here,

ho : clear span height of a column (cm)

The shear capacity of i-th story can be obtained as the sum of shear force of each column obtained from the formula (6) divided by the weight of the upper stories from i-th floor (sum of the axial force of each column).

The followings are the assumptions:

i) The cross section of the columns are all the same.

ii) The amount of tensile reinforcement of all the columns of a story is constant.

iii) The ratio (D/ho) of column height to clear span of a floor is 1/3.

iv) The ratio of the cross sections of columns in the story area is 1/100.

v) The average building weight is 1,2 tf/m_2.

vi) The yield point of main reinforcement is 3.6 tf/cm_2

Under these assumptions, the ultimate shear capacity can be obtained by the formula (7).

$$_mC_i = 160 / (n + 1 - i) * {_iP_t} + 0.3 \qquad (7)$$

$_mC_i$: shear capacity factor of i-th floor

$_iP_t$: tensile reinforcement ratio of columns

n : number of stories

i : the number of stories from the ground

In the formula (7), the first term in the right hand side is the one by the tensile reinforcement and the 2nd one is by the axial force. As recognized by the formula, the axial force of column strongly influences the such capacity. If the axial force decreases to the half of the original caused by the vertical motion (up and down),the 2nd term is to be 0.15 and the ultimate shear capacity greatly decreases.

2) Consideration

The formula (7) is given based on only the bending strength of columns through the observation of the earthquake damage situation. If a column member fails in shear failure before it reaches flexural yielding, or a beam reaches flexural yielding, before the column member reaches flexural yielding,, the value of the story shear capacity becomes smaller than the value obtained by the formula (7).

Therefore, it can be said that the formula (7) is an upper limit of ultimate strength of buildings given by the detailed calculation considering the hinge mechanism of buildings. For shear failure of columns, when the amount of main reinforcement is small and the ratio of axial force to compressive strength of columns is small as the cases stated below, since the shear strength is greater than the bending strength, shear failure hardly occurs except on the columns of the stories in the lower story. Moreover, even though columns would have shear failure. The following shear force coefficients can be expected [1] because of some margin of the maximum ultimate shear force to the allowable shear stress of columns:

- For the buildings based on the standard before 1971 - about 0.3 - 0.4
- For the buildings based on the standard after 1971 - about 0.4 - 0.6

Figure 3.2.1.4 shows the comparison of the result of the formula (7) when the minimum tensile main reinforcement ratio ($_iP_t$) is set up as 0.003 which almost corresponds to equivalent to the minimum reinforcement ratio (0.8%), and the minimum requirement value of ultimate shear capacity under the present standard. In this case, lack of strength is remarkable in the middle story. (the hatched area in the figure)

In actual buildings, the design shear force is bigger in the lower story than that of upper story, so it is common to increase the column cross sections as well as column main reinforcement. The increase of those cross sections and main reinforcement reflects to the increase of the ultimate shear force coefficient in the lower story. The buildings of more than 7 story are commonly constructed by SRC structure until the middle story and RC structure only for upper stories in Japan. In this case, the switching story to RC structure is outstandingly lack of strength, and ,there are some cases

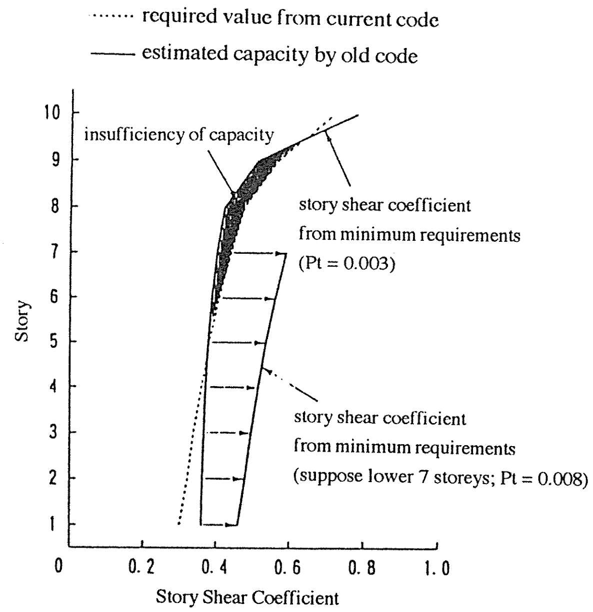


Fig.3.2.1.4 Estimated Values of Ultimate Story Shear Coefficient of Buildings Due to Old Code

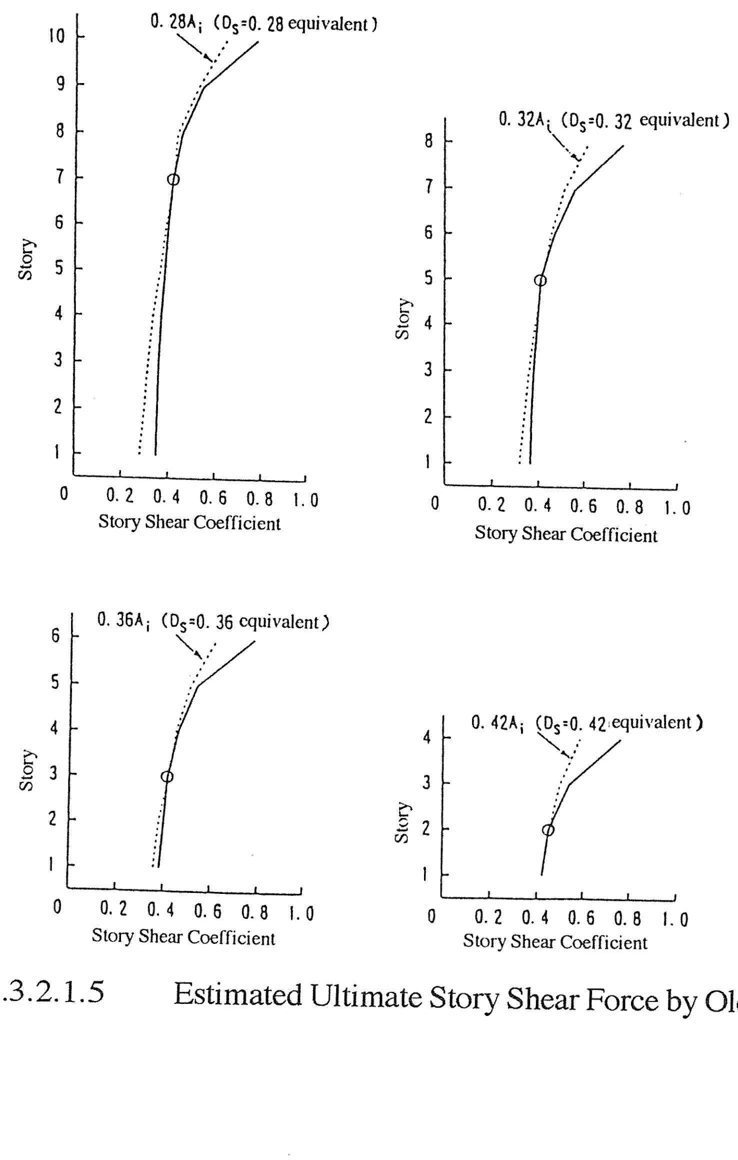


Fig .3.2.1.5 Estimated Ultimate Story Shear Force by Old Code of collapse of the stories as shown in Photo. 3.2.1.43. In this figure, the estimated value of the yield strength is shown, when the ratio of the main reinforcement of columns in the lower 7 stories is set up as 0.008. As recognized from the figure, the lack of surplus strength of the middle story is more emphasized.

Fig. 3.2.1.5 shows the result of the investigation of the10, 8, 6 and 4 story buildings under the same condition of the Fig. 3.2.1.4, explaining about the shear force coefficient of each story when the shear force coefficient at flexural yielding of column bending yield and the shear force coefficient by Ai distribution are overlapped in low-rise buildings. The point of correspondence of those shear force coefficients occur relatively in the lower stories, and the Ds conversion value at the time of correspondence is large as recognized from the figure. Therefore, in low rise buildings, even the buildings built under the old standard has some surplus strength of bending strength and if some failure would happen, it should be considered as the shear failure of the lower stories. It was commonly recognized that except the buildings with piloti story many failures happened in the 1st story of less than 6 story buildings caused due to this fact.

In the study above,This correspondence of the lateral strength coefficient and the set up shear force coefficient, is much influenced by the assumed shear force coefficient distribution, and such correspondence happens in the lowest story in the distribution as the regulation of the old standard. The form of shear force distribution in the earthquake of this time is very close to the form of A_i distribution under the present standard for the buildings discussed in this report.

3) Conclusion

i) The required design shear force coefficient of the old standard and that of the present standard are different in the upper stories. (about 0.2 is set up for all the stories in the old standard, but in the present standard, the bigger value is set up for the upper stories followed by Ai distribution)

In the buildings designed by the old standard and with little strength surplus against the design requirement, the lack of strength originally occurs in the upper stories. However, by the minimum reinforcement requirement, the roof story and the vicinity stories hold rather strong strength. It is considered that the shear force similar to the Ai shear force distribution worked more than the shear force distribution by the old standard in the earthquake of this time. And the specific stories situated in the middle stories ,which are outstandingly lack of strength, were concentratively damaged.

ii) Most of the buildings of more than 7 stories are constructed by SRC structure in the lower parts and RC structure in the upper parts. In those structured buildings large decrease of the column main reinforcement which leads to the sudden strength change of columns is recognized at the damaged stories. (converting story to RC structure) The size down of the column cross section is also recognized in the stories. The damages were concentrated in the specific floors with lower column strength.

iii) The axial force influences much to the column bending strength. If the axial force becomes outstandingly smaller due to the vertical earthquake motion (up and down), it can be considered that the sudden excessive horizontal deformation occurred and collapsed by the decrease of shear capacity.

iv) The buildings designed by the old standard but followed to the RC design standard established after

1971 have only small damage, which leads to the conclusion that it is because of the drastic change of shear design and the buildings designed following to the standard after 1971 are remarkably enhanced the structural performance of whole building including bending strength, compared with the ones designed before then.

1) Hiroshi Kuramoto, Toshimi Kabeyasawa
" The evaluation of safety factor of structural member strength in the existing allowable stress design method", Technical Paper to the Annual Report of Japan Concrete Institute , vol.15, No.2, 1993 (in Japanese)

b) Damages on Reinforced Concrete Buildings with Piloties

1) Introduction

The damage of the reinforced concrete buildings with piloties (shortly :piloti buildings) is one of the most typical damage by the earthquake of this time. The damage was seen in any buildings without regard to the year of designing before or after the present design standard, not concentrated only in the buildings designed before the standard to which damages in the middle stories of medium and high rise buildings, torsion and shear failure of first story were concentrated.

A study on the damage factors of piloti buildings of this time should be done to save piloti buildings ,which are often seen in big cities, from the disastrous earthquake this time. The collapse mechanism of the piloti buildings is based on the story collapse of the 1st story, since the piloti buildings are utilized for parking spaces or shops in the 1st story and apartment housings in upper stories, and thus the walls to separate each flat existing in the upper stories do not exist in the 1st story and only exist independent columns to result small rigidity of those buildings.

2) Problems of Piloti Buildings

i) Shear Force Coefficient Distribution

The present standard shear force coefficient distribution Ai which requires rather bigger shear force at the top , gives a proper story strength distribution to the buildings reflecting average vibration properties, but does not give a proper story strength to the buildings with piloti which hold unbalanced rigidity distribution. It is quite difficult to set up a proper shear force coefficient to correspond to the vibration mode in which deformation concentrates in a specific story of high rise buildings without any investigation used by a dynamic analysis.

ii) Stress of Piloti Story, Especially Axial Force and Shear Force

It is very difficult to set up properly not only the distribution along the height of buildings (as stated in i)) but also the design shear force of columns in a piloti story. Moreover, like one span building, when the axial force changes a lot, the interaction effect due to axial force influences much to the bending strength, shear strength and rigidity of columns. Even though the bending yields of the top and the bottom of the 1st story columns in both sides are assumed before designing, the working axial force to columns needs to be properly evaluated for shear design. Unless the interaction between axial force and moment is properly and fully considered for the analysis,total reasonable design is very difficult.

iii) Rigidity Ratio and Eccentricity Ratio

Piloti structure buildings are quite difficult structural formations to be designed, even though the difficult points are considered for designing as stated in sections i) and ii). However some piloti buildings damaged a lot might be designed without any recognition as piloti structures. The following reasons are considered for that ; For the modelling of analysis, there is no problem on the 1st story, because only independent columns are situated there, but the modelling influences much to the result of analysis and that of design at last , if the structure has the non-structural walls in the stories above the 2nd story.

Firstly the modelling might have been done, ignoring the non-structural walls under assumption of a beam yielding type mechanism. Actually, since the non-structural walls resisted well against the horizontal force, so the story collapse of the 1st story occurred on the contrary.

The story collapse might have been facilitated if the eccentricity of the buildings was added.

The influence of the non-structural walls have been caught attention as the problems of the shear failure of short columns and that of eccentricity since the 1968 Tokachi-oki earthquake, further the re-evaluation should inevitably be done in the near future from the view of securing the rigidity balance (plane & elevation) to influence a whole building structure.

iv) Seismic Resistance of Story Collapsed Buildings

The input energy (overturning moment) at the time of earthquake motion can be constant, if the buildings have the same number of story, same height and the same natural period except the final collapse mode, which should be studied in detail afterwards. It is quite difficult only at the first story of a high rise building to absorb the input energy, not to speak of a low rise building. It is very easy to confirm that higher strength of structural members (bending moment & shear force) should be needed even in a low rise building than the normal case. The relationship between the assumption of constant input energy and the necessary capacity should be studied more.

v) Negative Stiffness of Restoring Force Characteristics and P -Delta Effect

Piloti buildings, is quite different from the buildings of whole beam yield mechanism type building which can be expected the increase of the capacity after the beam yielding caused by the floor slabs. There are no increase of capacity in accordance with the deformation increase after the yielding of columns, since the 1st story is consisted only of independent columns. Therefore, the P - delta effect (increase of overturning moment by dead load followed by the increased deformation) in accordance with the deformation after the first story collapse mechanism is distinguished. It is necessary either to control the column yielding of collapsed story or to regulate deformation strictly.

3) Conclusion

It is necessary for designing piloti buildings to evaluate properly the dynamic behavior, interaction between axial stress and moment, P - delta effect, rigidity of shear walls and the non-structural walls of the upper stories, etc. and to investigate the design method of piloti buildings to be established in the near future.

c) Damage on Steel-encased Reinforced Concrete Buildings

1) Introduction

Steel-encased reinforced concrete (SRC) buildings had not been severely damaged in the past earthquakes. In the 1978 Miyagiken-oki Earthquake, most damage to SRC buildings was cracking of non-structural walls and minor damage to structural members.

In the 1995 Hyogoken-Nanbu Earthquake, a large number of SRC buildings were damaged in Kobe, Ashiya and Nishinomiya. The characteristics of their damages can be roughly classified as follows:

i) Damage to mixed buildings which consist of SRC in the lower stories and RC in the upper stories
ii) Damage to SRC buildings using built-up steel members
iii) Damage to column bases of SRC buildings
iv) Fractures of splice of steel in SRC member
v) Fractures of anchor of reinforcement in shear walls
vi) Damage to non-structural partitions

Examples of damaged structures on the above-mentioned six items are introduced and the cause of the damage are discussed in this section.

2) Examples of Damaged Structures

i) Damage to mixed buildings which consist of SRC in the lower stories and RC in the upper stories

Damage to a 7 stories office building constructed in Chuoh ward of Kobe are shown in Photo.3..2.1.58. This building collapsed in the third story. It is confirmed from Photo.3..2.1.59 which shows dismantling conditions that the building consisted of SRC frames up to the second story. Photo.3..2.1.60 shows damage to a relatively new office building in Chuoh ward of Kobe. This building also consisted of SRC frames up to the second story and severely damaged at the floor revel of the third story in which frames change from SRC to RC. In both buildings, it is expected as one of the cause of damage that the distribution of story shear capacity along building height changed suddenly in the damaged story.

ii) Damage to SRC buildings constructed with built-up steel members

Built-up steel sections which formed the core of SRC members were often fabricated with angles arranged to form lattices or ladders. Such SRC members with the built-up steel section were used for almost all SRC buildings in the 1940s to early of the 1970s.

Photo.3.2.1.61 shows the failure of a SRC column fabricated with built-up steel in the fourth floor of a 11 stories apartment building located in Chuoh ward of Kobe. This building had SRC with built-up steel frames in the upper floors supported by SRC with full-web steel frames on the lower three stories. Brittle failure of columns was observed in the fourth story in which frames changed from SRC with full-web steel to SRC with built-up steel. The cause of this failure may be that the shear capacity and sustaining capacity for axial load of SRC columns with built-up steel were less than those of SRC columns with full-web steel. This performance has often been pointed out in earlier experimental studies.

Photo.3..2.1.62 shows the bond splitting failure of a SRC column with built-up steel in a 8 stories office building in Nishinomiya. This building consists of wall frame system with a center core. In the seventh story, severe damage were observed and the bond splitting failure as shown Photo.3..2.1.62 occurred in a large number of outer columns.

iii) Damage to column bases of SRC buildings

There were many buildings in which the steel base plate was anchored at the ground floor level using a detail known as a non-embedment type steel base. Photo.3..2.1.63 shows the damage of a column base using this base plate detail in a 8 stories office building in Chuoh ward of Kobe. The failure had occurred at the bottom of the boundary column of a multi-story shear wall. Anchor bolts at the column base had been pulled out due to tension in the boundary column, which was transferred by overturning moment in the shear wall. And then buckling and rupture of longitudinal reinforcing bars in the column had occurred due to compression and tension from the shear wall. This kind of damage was often observed in this earthquake.

iv) Fractures of splice of steel in SRC member

Photo.3..2.1.64 shows the damage of a SRC boundary column with a gable wall in the fifth story of a 8 stories office building in Nishinomiya. In this damage, the fractures of splice steels in the SRC column occurred due to tension which was transferred by overturning moment in the shear wall. And then buckling of longitudinal reinforcing bars in the column occurred due to compression from the shear wall as well as the above-mentioned fracture in column bases.

v) Fractures of anchor of reinforcement in shear walls

Photo.3..2.1.65 shows the damage of a 9 stories office building in Chuoh ward of Kobe, which consists of SRC with full-web members. Although the damage to the main frames was not so severe, the fractures of anchor of transverse reinforcement in shear walls to boundary columns occurred. It was difficult to anchor reinforcing bars in shear walls into the core of columns because there were steels in the columns. As a result, anchor of the reinforcing bars were arranged in the outside of the core. Although rational detailing and placing of reinforcement for SRC structures are recommended in the recent AIJ guidelines, this building may have been designed by the old codes.

vi) Damage to non-structural partitions

During the 1978 Miyagiken-oki Earthquake, the non-structural partitions of many medium-and high-rise SRC buildings were severely cracked. Doors were stuck and egress was inhibited similar damage frequently occurred in this earthquake. Photo.3..2.1.66 shows an example of damage to a 11 stories apartment building in observed in almost all houses. A lot of repairs may be required for this building although the structural members have not subjected to damage so much. Therefore the extent of permission for this kind of damage should be reevaluated.

3) Conclusions

Through the survey of damage to SRC buildings in this earthquake, the following items which should be further examined can be drawn;

1) Evaluation of the proper distribution of story shear capacity along the building height in mixed buildings which consist of SRC frames in the lower stories and RC frames in the upper stories.
2) Reevaluation of the performance of column base and splice of steel in SRC columns.
3) Detailing for the anchor of reinforcing bars in shear walls into SRC members.
4) Evaluation of the stiffness and design criteria of non-structural walls.

Photo . 3.2.1.58 Damge of 7 story Office Building

Photo . 3.2.1.59 Dismantling Condition of Building

Photo . 3.2.1.60 Damage of relatively New Office Building

Photo . 3.2.1.62 Bond Splitting Failure of SRC Column

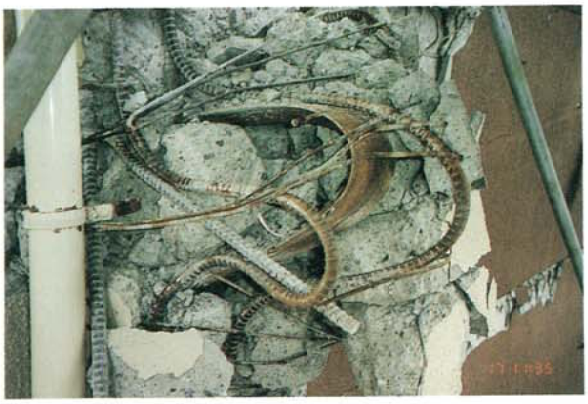


Photo . 3.2.1.61 Failure of SRC Column with Built-up Steel

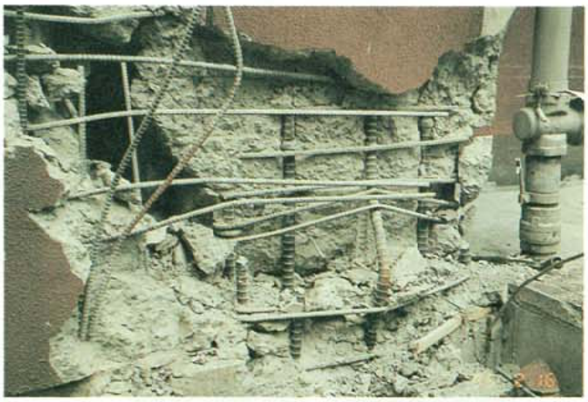


Photo . 3.2.1.63 Damages of Column Base

Photo . 3.2.1.64 Fracture of Splices of Steel

Photo . 3.2.1.65 Fracture of Anchor of Reinforcement

Photo . 3.2.1.66 Damage of Non-structural Partitions

d) Material and Construction

1) Introduction

The earthquake of this time caused much severe damage on thousands of reinforced concrete structures as collapse, fall of floors, large scale failure, etc. The detection of the causes and the countermeasures for the future should of course be necessary from various aspects, and moreover, the study on the material and construction of the reinforced concrete is also be inevitable. The causal factors of deadly damaged or collapsed reinforced concrete buildings are out as inappropriate treatment of reinforcement end portion like a hoop, gas pressure welding of reinforcement, and also cracks by reinforcement corrosion by salt damage, lack of concrete strength, etc. Those weight may influence the structural capacity of those buildings. This report shows the result of the investigation on the damage by salt, alkali aggregate reaction and reinforcement condition as well as compressive strength of concrete and Young's modulus to know the relation between the damage condition of reinforced concrete buildings, the materials and the construction methods, by taking core specimens from the damaged buildings.

2) Investigation Method

The investigation was carried out four times in Kobe City and Ashiya City to know the point of problem of the damaged reinforced concrete buildings on the materials and construction. The 1st investigation was done on about 140 buildings from the 25th to the 28th of January, '95 to grasp the summary of the damage of reinforced concrete buildings. Firstly the eye observation of external appearance was done, then the analysis of the Cl- content in the concrete and the kind of aggregates was carried out by collecting 30 concrete pieces. The 2nd investigation was also a visual investigation of the damage of inside and outside of 5 apartment houses but it was carried out in detail from the 30th, January to

Table 3.2.1.4 Buildings for Core Boring Test and Damage Condition

	use	FL	compl. year (kgf/cm^2)	fc'	structural type	damage grade	damage condition
A	office bldg.	8	'57	210	SRC: 1–5FL RC: 5–8FL	collapse	fall story collapse at six floor
B	office bldg.	4	'60	180	RC	small	shear cracks of columns, 1FL
C	office bldg.	8	'71	180	SRC	severe	shear crush of columns and wall
D	connecting part betw. hall and main bldg.	2	'71	210	RC(moment resisting frame)	severe	four columns(1FL) Tops:shear crush
E	gymnasium	3	'72	180	column, wall: RC. girnder:Steel roof:prestressed concrete(PC)	collapse	P. C. roof fell down, 2FL column flexural rapture

the 4th, February, 1995, for investigation of damage condition. The 3rd one was for the investigation of the compressed strength and young's modulus, which was done by collecting concrete cores from the damaged 5 reinforced concrete buildings to test those items from the 2nd to the 4th of March, '95.

The 4th one was for the study of the influence of alkali aggregate reaction in the damage. 16 buildings which were doubted to have alkali aggregate reaction by the emergency risk assessment were taken as the objects and collection of concrete pieces from the 8 buildings of the 16 ones was carried out from the 17th to the 19th, April, '95. The investigation of the concrete compressive strength and Young's modulus was started by collecting several pieces of 10 cm diameter core specimen from the 5 damaged buildings shown in Table 3.2.1.4 and the specimens were tested due to JIS A 1107 "Method of obtaining and testing drilled cores and sawed beams of concrete".
The collection spots of core specimens are basically the floors of much damage in the center and the surrounding upper and lower floors. The number of specimens are 1 - 4 from one spot (Same floor and same members condition as well as the ones with shear cracks were chosen from the heavily damaged floors. When the core was taken from the members with cracks, it was taken from the part where the cracks are not seen and seems comparatively in good condition. The collected cores were cut at both ends and the surface of both ends were finished by grinding. At the time of compressive strength test, the specific gravity was obtained by measuring the size of the specimens and the mass and Young's modulus was also obtained by measuring stress-strain curve used by a compressor meter. Young's modulus was decided as 1/3 secant modulus of compressed strength.

3) Result of Investigation and Consideration

i) Compressive Strength of Concrete and Young's Modulus

The test result is shown in the Table 3.2.1.5 and the distribution of the compressive strength of concrete in each building is shown in the Fig. 3.2.1.6. Those test results are considered as follows:
The specified design strength of concrete of building A was 210 kgf/cm2, but every compressive strength of core specimens obtained from the building was less than the specified design strength, and showed a strength of which mean value was 130 kgf/cm2. The damage condition of the building is the story collapse of the 6th floor. Through the analysis of causal factor of the damage is assumed that since the building structure consists of SRC from the 1st floor to the middle part of the 5th floor column and of RC for the upper floors, the sudden change of the strength and rigidity of the building was occurred on the 6th floor. Looking through the compressive strength of concrete, the mean

value of compressive strength of each floor is 122kgf/cm2 for the 5th floor, 131kgf/cm2 for the 6th, and 135kgf/cm2 for the 7th floor, which means the compressive strength of the story collapsed 6th floor is not especially low. However, the mean value of the compressive strength of concrete is lower than the specified design strength and since the value is very low, the required strength and the rigidity of a structure seems to become shortage on the 6th floor. Moreover, looking through the concrete condition of the 6th floor of which whole layer is collapsed and fallen, it can be said that the compressive strength of concrete was so low that it lead whole story collapse which was begun in several places. The mean value of the compressive strength of concrete in the core specimens taken from the basement wall of the building B is 135kgf/cm^2 and the compressive strength of the retaining wall of an external dry area (basement) is 264kgf/cm^2. There is much difference between them. It is difficult to consider that there is much difference in mix proportion of concrete, but can be considerable that the difference comes from the difference of humidity conditions between the basement wall, which does not contact with the ground and that of the retaining wall, which directly contacts with the ground. Because the humidity condition influences the curing of concrete. The damage condition of the building is cracks of the 1st floor column but the definite cause of the damage can not be said, for the compressive strength of the concrete of the 1st floor column has not been investigated yet. The mean value of the compressive strength of the concrete in the specimens from the building C is 139kgf/cm2, which is quite low compared with the value of the specified

Table 3.2.1.5 Test Result of Concrete by Core Boring Spec.

	core boring site	Max load (t)	height diameter ratio	fc' kgf / cm^2	Young's modulus $\times 10^5$ kgf/cm^2	specific gravity
A	5FL column	10.77	1.61	134	1.31	2.31
	5FL wall	8.72	1.66	109	1.54	2.35
	6FL column(1)	8.22	1.74	103	1.27	2.30
	6FL column(2)	15.05	1.60	187	–	2.37
	6FL wall	8.93	1.10	104	–	2.30
	7FL column (1)	7.36	1.78	93	–	2.32
	7FL column (2)	12.53	2.00	161	1.52	2.31
	7FL wall	12.75	1.25	152	1.88	2.36
B	B1FL wall (1)	10.83	1.41	132	1.53	2.34
	B1FL wall (2)	11.04	1.22	131	–	2.37
	B1FL wall (3)	12.45	1.04	143	–	2.38
	B1 retaining wall	22.05	1.28	264	–	2.36
C	6FL column	14.08	1.06	162	–	2.29
	6FL wall	9.39	1.01	108	–	2.24
	7FL column (1)	14.41	1.54	178	–	2.33
	7FL column (2)	10.68	1.07	123	–	2.28
	7FL wall (1)	11.90	0.99	135	–	2.26
	7FL wall (2)	11.89	1.36	144	–	2.35
	7FL wall (3)	11.40	1.12	133	–	2.26
	7FL wall (4)	9.57	1.18	113	1.37	2.26
	8FL column	12.38	1.18	146	–	2.29
	8FL wall	11.81	1.38	143	–	2.30
D	1FL long column	20.21	1.99	259	1.58	2.32
	2FL long column	20.39	2.00	261	2.08	2.31
	1FL column (1)	21.67	1.38	263	–	2.32
	1FL column (2)	18.19	1.27	217	–	2.30
E	3FL wall (1)	21.09	1.45	258	–	2.31
	3FL wall (2)	14.33	1.48	176	2.02	2.14
	3FL wall (3)	20.17	1.38	244	–	2.29
	3FL wall (4)	15.42	1.30	185	–	2.34

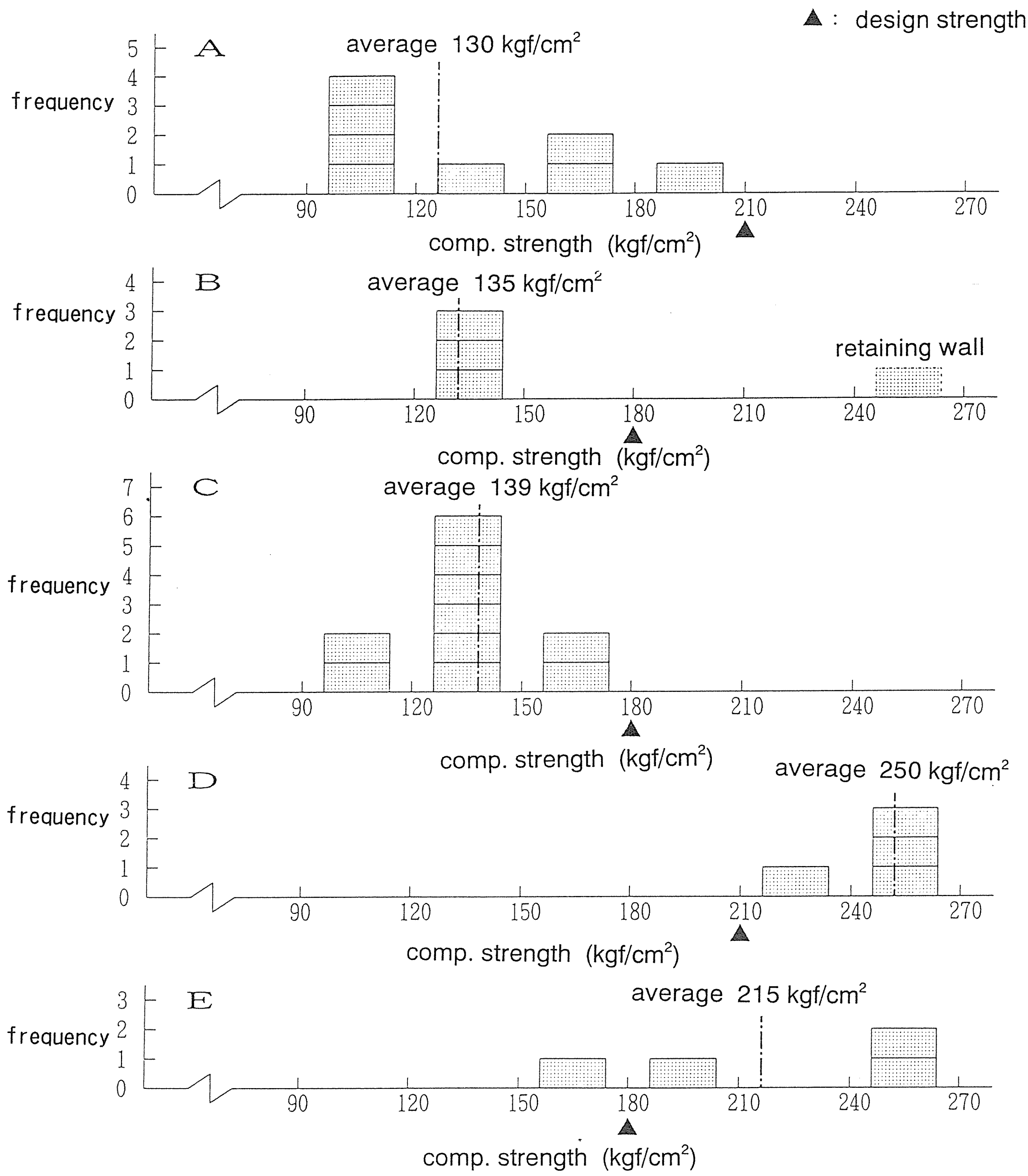


Fig. 3.2.1.6 Distribution of Concrete Compressive Strength for Each Surveyed Building

design strength, 180kgf/cm2. The damage condition of the building C is the shear failure of the 7th floor column and the seismic resistant wall and the mean value of the compressive strength of the 7th floor concrete is 138kgf/cm2, but the value for the 6th floor is 135kgf/cm2 as well as 145kgf/cm2 for the 8th floor, which means the value for the 7th floor is not especially low. However, as same as the building A, the concrete compressive strength of the building C so low that the required strength and rigidity for a structure is shortage on the 7th floor for the damage concentrated on the floor. The mean value of the compressive strength of the concrete specimens from the building D is 250kgf/cm2 and the concrete construction condition is rather good. The damage of the building is the shear failure of the top of the columns which support the 2nd floor, but the damage is not seen on the long columns to hold the 3rd floor. As the columns of the 1st floor to hold the structural body upper from the 2nd floor had a strong input force, the shear failure was occurred at the top of the columns.

The mean value of the compressive strength of the concrete among the core specimens from the building E is 215kgf/cm2. Even though some core specimen showed less than 180kgf/cm2 (specified design strength) as the compressive strength, the value of the compressive strength totally more than that of the design strength. The core specimens of the building were taken from the same wall in different height, but any constant tendency of concrete compressive strength could not been seen. The characteristic of the damage condition of the building is the fall of PC roof constructed between the steel beam and the concrete wall. Since the rigidity of the wall was shortage, some space between the beam and the wall made to get the roof to be taken. It is very difficult to secure the rigidity of concrete by the compressive strength, and it is quite presumable that the compressive strength of concrete of the building did not influence the damage.

Though the definite tendency of the relationship between the compressed strength of concrete and the damage condition of buildings can not be clarified, for almost none of the core specimen from non damaged buildings were collected, it can be considered that the reason to make the large size damage on buildings was on the fact the compressed strength of concrete was lower than the specified design strength except the problems on structural design such as the shear failure of columns of the 1st floor piloti, the fall of PC roof, etc.

Young's modulus of concrete is slightly smaller than the formula in the reinforced concrete structure calculation standard established by the Architectural Institute of Japan, but compared with the actual investigation results of the structural concrete compressive strength and Young's modulus, it is not especially small and reasonable value.

ii) Chloride Ion Content in the Concrete and the Corrosion Condition of Reinforcement.

Fig. 3.2.1.7 shows the frequency distribution of the chloride ion content which was measured by using the concrete pieces collected from the damaged members of 16 damaged buildings. The minimum of the measured values of the chloride ion content is 0.09kgf/m^3, the maximum is 2.50kg/m^3, and the mean value is 1.05kg/m^3. Those values are almost corresponding to the result of the investigation done in Kobe-Osaka area before. It is assumed that those buildings were built before 1986 when "The Regulation Standard of Total Chloride Content in the Concrete" by the Ministry of Construction was enforced.

For that reason, the chloride content of those buildings shows high value. The buildings showing the chloride ion measurement value as 0.09kg/m3 were built during 1947 and river sand was used to those buildings. On the other hand, there are some buildings even built before the 2nd World War, but showing a rather large value of the content such as 1.04kg/m3.

Through the observation of the corrosion condition of the exposed reinforcements in the members of damaged buildings, much rust in the main reinforcements and many members with hoops and stirrups with sectional loss caused by corrosion were recognized.

iii) Alkali Aggregate Reaction

On the first investigation, cracks of unidentified reason on the wall of one building out of 140 buildings were recognized but no damages caused by the earthquake were seen on the wall. Out of collected 30 concrete pieces, 3 pieces were suspicious of reactive aggregate, but any cracks which might be caused by expansion were not recognized on the pieces.

On the 4th investigation, the cracks supposed to be caused by the alkali aggregate reaction along the axial direction of the member were obviously recognized on 3 buildings of the 15 buildings of which location were recognized. The foundation beam of one of the buildings was dug out and through the observation, since the reaction rim around the coarse aggregate of concrete of the part was recognized, the alkali aggregate reaction was definitely been occurred. There is also one building where some suspicious cracks are observed. Through the eye observation of the concrete pieces it is clarified that the alkali aggregate reaction was occurred, as the reaction rim was seen around the rough aggregate. The cracks caused by the alkali aggregate reaction on the external appearance were hardly seen, but there were 2 buildings which supposed to use the aggregate which is suspicious of reaction through the eye observation of the concrete pieces. Generally the members with much alkali aggregate reaction are concerned with the decrease of capacity, but a definite relationship between the cracks caused by the alkali aggregate reaction and the damages caused by the earthquake was not recognized in this investigation.

iv) Concrete Filling Condition

The concrete filling condition by eye observation was almost fine. Some old buildings showed partially on their columns and beams. Those buildings often use river sand as the coarse aggregates and the distribution of reinforcement was generally not dense. Because of those, the condition of concrete filling in those buildings depends more on concrete condition than on the reinforcement distribution condition.

v) Treatment of the Reinforcement Ends

It is desirable to place 135 degree hooks and to provide enough extra spaces at the ends of the shear reinforcements such as hoops of columns, stirrups of beams, etc. However in the investigation of this time, the ends with 135 degree hook in both ends of the shear reinforcement could hardly be recognized. There were many cases like one end was sustained by 135 degree hook and the another end was sustained by 90 degree hook. There were also many cases like both ends were sustained by 90 degree hook. There was no case of opening the end of the shear reinforcement when the end was bent in 135 hook. However, the reinforcement was cut down at the bent part. The reinforcement distribution method for the end should be further studied on the following points:

Every reinforcement end like shear reinforcement should be 135 degree hook or not.

The reinforcement of welding close type should be used for that or not.

For the joint of the reinforcement, while many damaged gas pressure welded parts were seen, many undamaged gas pressure welded parts were seen at the same time. Some other places were damaged in this case.

vi) Conclusion

It was recognized through the investigation of this time that the compressive strength of concrete was low in some reinforced concrete building . Moreover, many problems are discovered by investigation of many items on the buildings such as the relationship between chloride ion content in concrete and corrosion of reinforcement, alkali aggregate reaction, concrete filling condition, bending degree of the reinforcement ends, etc. The interaction of those of materials and construction and those by the earthquake has not been clarified yet. Those should be studied further in the future.

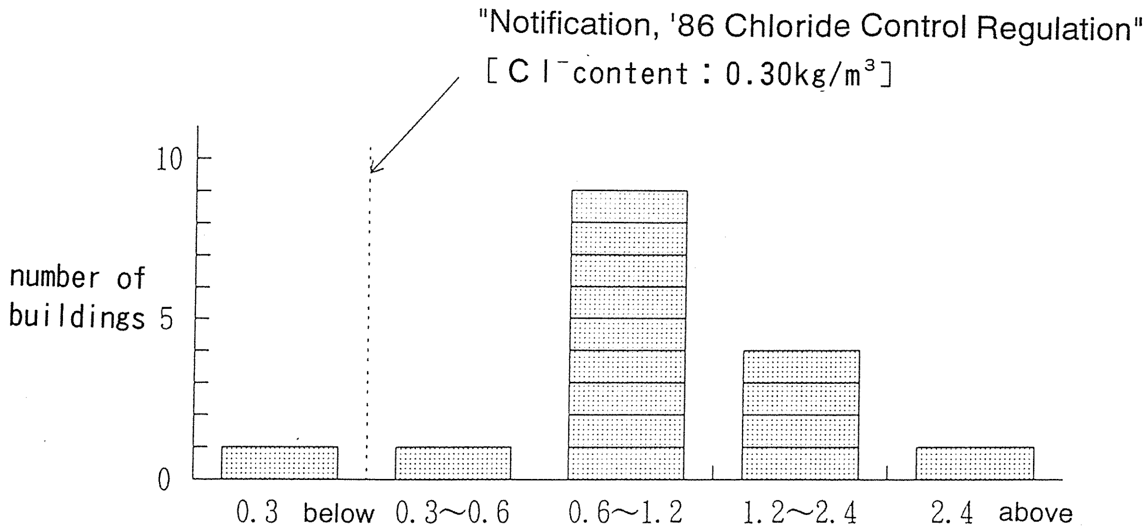


Fig. 3. 2. 1. 7 Chloride Ion (Cl^-) (kg/m^3)

Photo.3.2.1.67 damaged hook by corrosion

Photo.3.2.1.68 cracks along the axial direction
Of column by alkali aggregate reaction

Photo.3.2.1.69 reaction rim around coarse aggregate by alkali aggregate reaction

Photo.3.2.1.70 135degree hook in
Reinforcement end

Photo.3.2.1.71 damaged bent part in reinforcement end

Photo.3.2.1.72 90 degree hook in reinforcement end (in
the case of opening)

Photo.3.2.1.73 90 degree hook in reinforcement end (in the case of no opening)

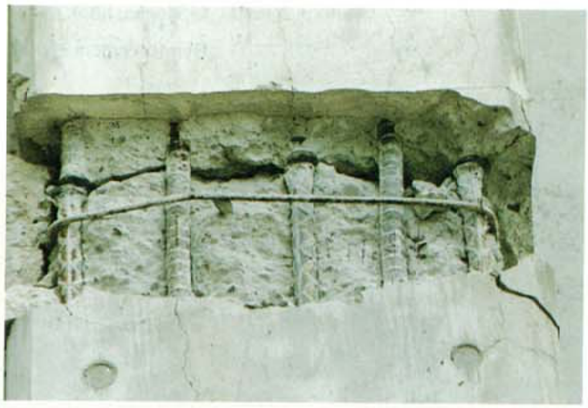


Photo.3.2.1.74 damaged joint in gas pressured welded parts

Photo.3.2.1.75 damaged gas pressure welded parts and not damage ones

(5) Conclusion

Almost all the damages ever seen in the past earthquake disaster on RC buildings were seen in the earthquake disaster of this time such as the shear failure of columns and walls, for example , of the characteristic ones.

characteristic damages of this time are severe damage of the piloti part of the buildings with piloti on the 1st floor, story collapse of the specific floors of middle and high rise buildings, failure of SRC steel joint, damage on the anchors of column bases, etc. A strong interrelationship between the damaged buildings and the design standard which they followed was recognized. Namely, the design standard of RC structured buildings was revised twice in 1971 and 1981, which influenced damages are not seen in almost all the buildings built after 1981 except the buildings with piloties. much to the types of the damaged buildings. When the RC buildings are divided into three categories like 1. before 1971, 2. between 1971 and 1981, and 3 after 1981 , it is clarified that the damage ratio of these buildings classified in each group is outstandingly different through the macro analysis result of this investigation. The result is as follows:

The damage ratio of the buildings built before 1971 is quite large.

The ratio of the buildings built after 1971 is small.

Especially, as stated in the section (3), any enormous damages are not seen in almost all the buildings built after 1981 except the buildings with piloties. Based on the fact, the characteristic cause factors of the damages of this time are pointed out as follows:

a) Collapse of the Middle Floors

i) This damage is often seen in the buildings built based on the old standard especially the ones built before 1971. As far as the information ever collected, the damage has not been reported on the buildings designed based on the present standard. The type of the collapse is column collapse.

ii) The same shearing force coefficient is set in every height of the buildings in the old standard, while in the present standard, the story in the buildings becomes higher the bigger the shear force coefficient is set.

iii) The maximum stress can be satisfied with the minimum reinforcement the structural regulation such distribution only against the external force required in the old standard.

iv) However, the strength expected in the structural regulation such as the minimum reinforcement distribution is lower than the value of the minimum strength required for the middle floors in the present standard (Ds=0.3)

v) Therefore, the middle floors which are lack of horizontal strength collapsed.

vi) The structural conversion from SRC to RC and the contraction of column cross sections accelerated the collapse.

vii) The influence of vertical motion (up and down, the change of axial force) also accelerated the collapse.

b) Damage of Piloti Buildings

It is assumed that the damage of piloti buildings such as collapsed and rupture were caused by the neglect of the following points for the design:

i) dynamic behavior of piloti buildings

ii) cross sectional design of piloti columns fully considered the interaction of axial force and moment

iii) rigidity evaluation method of bearing walls and the secondary walls

iv) P -delta effect followed by excess deformation

c) Damage of SRC

i) The damage occurred on the converting (bordering) floors from SRC to RC in the SRC and mixed structured buildings, caused by the sudden change of possessing horizontal strength.

ii) The failure of the steel column bases and the joints of SRC columns because of the unsuitable design detail for those.

iii) The damage caused by the unsatisfactory anchorage of the wall reinforcement to the columns and beams of continuous shear walls.

iv)The failure and deformation of non structural walls.

(6) Countermeasures

a) It can be said that especially the collapse of the buildings based on the old standard before 1971 was caused by the shortage of the strength of the lower stories in low rise buildings and the one of the middle stories in middle and high rise buildings. Especially the collapse of the middle stories often leads to the collapse shaped like pan cakes and to loss of many people's lives. An reinforcing countermeasure should be studied and established from those view points.

b) The piloti buildings built under the present standard as well as the o ones under the old standard were damaged due to this earthquake. The collapse of piloti buildings are caused generally by the original characteristic of the structural form. Almost all the buildings of more than large scale damage which were designed even under the present standard are piloti typed buildings. Therefore, the following points should be considered for establishing a suitable strength design method for piloti buildings:

- dynamic behavior
- interaction between axial force and moment
- increase of overturning moment followed by excessive horizontal deformation
- stiffness of shear walls and the secondary walls of upper stories

c) The following countermeasures are also needed for the future:

- The design method for SRC column base anchors and joints with considering proper tensile force should be reviewed for the tensile failure of the column base anchors and joints of SRC and the accompanied failure of columns and shear walls.

- For the fall damage caused by the unsuitable attachment of pre-cast roofs, the protection method against the fall such as setting up the horizontal frames and diagrams, reinforcement of column rigidity, and securing should be completely established.

- In the heavily buildings, many hoop edges of 90 degree are seen. It is necessary to enforce establishing the regulation of 135 hook and using the welded close type shear reinforcement.

- In some of RC buildings, the following phenomena are seen :

i) The compressive strength of concrete is low,

ii) The chloride ion content in concrete is high

iii) Alkali aggregate response is occurred.

The relationship between those phenomena and the earthquake damage should be studied in the future, but it is essential to secure the proper quality of concrete completely.

3.2.2 Steel Buildings

(1) Outline

The damage investigation of steel buildings was conducted as follows:

a) Emergency Investigation by the Earthquake Damaged Buildings Investigation Committee:
the period from the middle of February to the beginning of March, 1995

b) Damage Investigation:
mainly the period from the 25th to the 29th, January, 1995

c) Investigation on All Steel Buildings at Specific Areas:
the period from the 20th to the 23th, February, 1995

d) Detailed Investigation on Specific Damaged Buildings:
mainly from the 24th to the 25th, February, 1995

As stated in Chapter 2, the emergency investigation by the Earthquake Damaged Buildings Investigation Committee is the reviewed investigation on 1231 buildings which were assessed as prohibition of the use and those of nearly the same assessment just after the earthquake. Among the 1231 buildings, the damage of 316 steel buildings was analyzed for inspecting the correlation ofdamage level/aspects, year of construction and size of structures/configuration statistically. The details of the damages are :

collapse and severe damage - 55%
moderate damage - 16%
minor, light and no damage - 27%
unidentified - 2%

Out of the buildings of which year of construction was identified as "before 1981" or "after 1981", the buildings of collapse and severe damage were taken to be compared the ratio of damage according to the damaged parts. (The number of the buildings is 70 for the ones built before 1981 and 25 for the ones after 1981.)

The result is as follows:

fracture of high strength bolts at joints
before '81 - 6/70=9%
after '81 - 1/25=4%

fracture of welded joints
before '81 - 16/70=23%
after '81 - 10/25=40%

damage to column bases
before '81 - 27/70=39%
after '81 - 9/25 =36%

The main damage types of the steel buildings designed according to the current seismic code and with collapse and severe damage are fracture of the welded joints of the 1st story column top and

elongation and fracture of the anchor bolts of column bases.
The damage investigation was done mainly by the Building Research Institute mostly in Chuo-ward and Higashinada-ward, Kobe City and Ashiya City. Through the result, the damages are roughly divided into several groups and the typical damages are shown.

The classified groups are ; 1)buildings with square tube columns, 2)buildings using light gauge steel and with much secular degradation, braced frame buildings, and high rise buildings. The damage of the column bases and the surroundings, and the fracture of the welded beam-to-column connections are often seen in the buildings with square tube columns.

The investigation on all steel buildings at specific areas was done by the Building Research Institute, the Kozai Club and Lath Sheet Industrial Association in a part of Higashinada-ward, a part of Chuo-ward (conducted as a collaborative survey with the Architectural Institute of Japan and in a part of Hyogo-ward, Kobe City to investigate the damage ratio among the steel buildings taller than or equal to 3 story (except light gage steel structures) and the damage ratio in each area. The total number of investigated buildings is 655 and about 85% of them are not taller than 5 stories. The ratio of the damage level is about 1:1:4 for collapse/severe damage, moderate damage and minor /light/no damage, respectively. The ratio of collapse/severe damage in each area is 26% for Higashinada-ward, 20% for Chuo-ward and 12% for Hyogo-ward.

The detailed investigation on specific damage buildings was done mainly on the buildings designed according to the current seismic code to investigate the causes of damage in detail referring the drawings and specifications at the Building Research Institute. Investigated are 9 buildings most of which are in Kobe City and 6 of them are the public buildings. As the detailed analysis is still on the process, only the outline of the investigated buildings and the damage is described.

(2) The Macro Analysis of the Steel Buildings of the Urgent Investigation

The result of the macro analysis of the steel buildings of the urgent investigation is shown in the table from 3.2.2.1 to 3.2.2.7. The steel buildings in this investigation do not include the mixed and composite structures of steel and reinforced concrete, and the number is 316. 219 of them is the moment resisting frame, and the ratio to the total number is more than 2/3. (Table 3.2.2.1) 60% of them are 4 ~ 6 stories buildings. (Table 3.2.2.2) The followings are the definitions of the damage level of the steel buildings of the urgent investigation. (cited from the Table 2.2.1.2)

a) Collapse: a building which is totally fallen or collapsed or of which one part is fallen or collapsed

b) Severe damage:

1. There is more than 1/30 radian inclination in the stories.
2. There are outstandingly large local buckling and flexural buckling in the main structural components. (incl. the buildings of which bracing was broken in more than 50%).
3. External protruded structures like a penthouse are collapsed or fallen

c) Moderate damage: not categorized ones in more than severe damaged ones and less than minor damaged one

1. There are the damages like local buckling on the main structural components (incl. the case of failure which is more than 20% and less than 50% of bracing is damaged)
2. One thirds of exterior wall ALC board is fallen or almost fallen

d) Minor or light damage:

1. Components except bracing, junction and foundation are not deformed, and the fracture ratio of the bracing is less than 20%
2. Even though the total surface of the exterior wall of mortar finish or the partial surface is fallen, there is not any damage on the main components

The buildings with at least one of the characteristic stated as above can be categorized in one of those damage level.

The followings are recognized by the macro analysis.

a) 174 buildings of the 316 urgent investigated steel buildings are categorized as collapse or severe damage, which is more than half of the investigated buildings. But, only 25 buildings are recognized as the buildings built after the establishment of the current seismic resistant standard (after 1982). (Table 3.2.2.3)

b) Among the buildings built after 1982, the damage of the moment resisting frame structures is 9 times more than the one of the brace structures, but among damage of the buildings built before 1981, the damage of the moment resisting frame structures is 2 times and little more than the one of the brace structures. (Table 3.2.2.4)

c) The collapse in the middle floor is 2.2% of the investigated buildings. (Table 3.2.2.5)

d) 50 buildings of the investigated buildings are identified as welding fracture, while 88 buildings are identified non weling fracture. The number of the buildings of which damage level is collapse or severe and in which the welding failure is recognized is 43, while the number of the buildings of which damage level is also collapse or severe but in which the welding failure is not recognized is 32. (Table 3.2.2.6)

e) The number of the buildings which are damaged at the column base anchor bolts and foundation concrete is 71. Most of the damages are pull-out and fracture of the anchor bolts. (Table 3.2.2.7)
In the Table 3.2.2.8 shows the damage summary of the collapse or severe damaged buildings built after 1982. Almost all the buildings are the moment resisting frame with square-tube column and H shaped beam. The main damage types of those buildings are the welding fracture at the column top in 1st floor and the elongation and fracture of the column base anchor bolts.

Table 3.2.2.1 Structure type due to the constructed year

	Structure type			
Constructed year	Moment resisting frame	Braced frame	Unidentified	Total
～1971	64	29	4	97
1972～1981	27	12	8	47
1981～	39	4	3	46
Unidentified	89	23	14	126
Total	219	68	29	316

Table 3.2.2.2 Damage level due to the story number

	Damage level					
Story number	Collapse	Severe damage	Moderate damage	Minor or light damage	Unidentified	Total
～3	9	34	23	30	2	98
4～6	30	88	22	52	3	195
7～	4	9	4	4	0	21
Unidentified	0	0	0	0	2	2
Total	43	131	49	86	7	316

Table 3.2.2.3 Damage level due to the constructed year

	Damage level					
constructed year	Collapse	Severe damage	Moderate damage	Minor or light damage	Unidentified	Total
～1971	10	41	17	29	0	97
1972～1981	3	16	9	19	0	47
1981～	10	15	8	12	1	46
Unidentified	20	59	15	26	6	126
Total	43	131	49	86	7	316

Table 3.2.2.4 Damage level due to the structure type

		Damage level					
structure type	constructed year	Collapse	Severe damage	Moderate damage	Minor or light damage	Unidentified	Total
Moment resisting frame	～1971	7	28	10	19	0	64
	1972～1981	1	9	3	14	0	27
	1981～	9	13	6	11	0	39
	Unidentified	15	45	8	17	4	89
Braced frame	～1971	3	13	4	9	0	29
	1972～1981	1	4	3	4	0	12
	1981～	1	1	1	1	0	4
	Unidentified	3	8	5	6	1	23
Unidentified	～1971	0	0	3	1	0	4
	1972～1981	1	3	3	1	0	8
	1981～	0	1	1	0	1	3
	Unidentified	2	6	2	3	1	14
Total		43	131	49	86	7	316

Table 3.2.2.5 Damage of structure

	Damage of structure						
	Collapse in first story	Collapse in middle story	Column buckling	Composite damage	others	Unidentified	Total
	38	7	38	12	166	55	316

Table 3.2.2.6 Damage to connections due to the damage level

Damage level		Damage to connections				
		Fracture of welding	Fracture of high strength bolts	No damage	Unidentified	Total
Collapse	～1981	5	0	1	7	13
	1981～	7	1	1	1	10
	Unidentified	9	0	2	9	20
Severe damage	～1981	11	6	11	29	57
	1981～	3	0	4	8	15
	Unidentified	8	1	13	37	59
Moderate damage	～1981	2	0	9	15	26
	1981～	1	0	1	6	8
	Unidentified	3	1	4	7	15
Minor or light damage	～1981	1	1	26	20	48
	1981～	0	0	4	8	12
	Unidentified	0	0	12	14	26
Unidentified		0	0	0	7	7
Total		50	10	88	168	316

Table 3.2.2.7 Damage of column base due to the damage level

Damage level		Damage of column base: Failure of column-base concrete	Failure of anchor bolts	Failure of column-base concrete and anchor bolts	No damage	Unidentified	Total
Collapse	～1981	1	5	0	1	6	13
	1981～	0	4	0	1	5	10
	Unidentified	1	6	1	1	11	20
Severe damage	～1981	8	13	0	16	20	57
	1981～	1	3	1	2	8	15
	Unidentified	7	8	1	11	32	59
Moderate damage	～1981	3	1	0	11	11	26
	1981～	1	0	0	1	6	8
	Unidentified	2	1	0	3	9	15
Minor or light damage	～1981	0	1	0	27	20	48
	1981～	0	0	0	5	7	12
	Unidentified	1	1	0	15	9	26
Unidentified		0	0	0	0	7	7
Total		25	43	3	94	151	316

Table 3.2.2.8 Serious damaged buildings designed according to the current seismic code

No.	Damage level	Address	Constructed year	Purpose	Story	Structure	Outline of damage
1	Collapse	Shinkaiti,Hyogo-ku	1983	Hotel	4	Square tube column, H-shaped beam	Weld fracture at column top in first story
2	〃	Kamisawa-dori,Hyogo-ku	1987	House	4	〃	Weld fracture at column top in first story
3	〃	Shimosawa-dori,Hyogo-ku	1991	House	4	〃	Weld fracture at panel-to-through-diaphragm joint in first story
4	〃	Shimosawa-dori,Hyogo-ku	1982	Office	4	〃	Weld fracture at column top in first story
5	〃	Kanou-cho,Chuo-ku	1985	Shop	7	〃	Weld fracture at panel-to-through-diaphragm joint in first story, pull-out of concrete encased type column base
6	〃	Kotonoo-cho,Chuo-ku	1989	School	8	〃	Fracture of anchor bolt, weld fracture at column top in first story
7	〃	Terada,Suma-ku	1988	House	5	〃	fracture at panel-to-through-diaphragm joint in first story
8	〃	Honjyo,Higashinada-ku	1991	Shop	4	Square tube column, H-shaped beam, brace	Fracture of bracing member
9	〃	Ooishihigashi,Nada-ku	1995	House	4	Square tube column, H-shaped beam	Weld fracture at column top in first story
10	〃	Kitanagasa-dori,Chuo-ku	1984	Shop, Office	10	〃	Weld fracture at column-t- column splice in 3rd story
11	Severe damage	Shimosawa-dori,Hyogo-ku	1989	House	5	〃	Fracture and pull-out of anchor bolt
12	〃	Kgura-cho,Nagata-ku	1983	Factory	4	Square tube column, H-shaped beam, brace	Building inclination
13	〃	Hosoda,Nagata-ku	1988	Office	4	H-shaped column with plate,H-shaped beam	Remaining deformation
14	〃	Hiyoshi,Nagata-ku	1985	Shop	2	Square tube column, H-shaped beam	Remaining deformation
15	〃	Wakamatsu-cho,Nagata-ku	1990	Shop	4	〃	Remaining deformation (more than 1/30 rad.)
16	〃	Nishidai-dori,Nagata-ku	1987	Office	4	?	Remaining deformation
17	〃	Oota,Suma-ku	1990	House	4	Square tube column, H-shaped beam	Weld fracture at panel-to-through-diaphragm joint in first story
18	〃	Mikageishi,Higashinada-ku		House	3	〃	Weld fracture at column top in first story
19	〃	Sumiyoshigu,Higashinada-ku	1988	Office	5	〃	Fracture of anchor bolt
20	〃	Oishiminami,Nada-ku	1990	Office	4	〃	Collapse in first story
21	〃	Warituka-dori,Chuo-ku	1985	House	5	〃	Remaining deformation
22	〃	Kitanagasa-dori,Chuo-ku		Shop	8	〃	Remaining deformation
23	〃	Kitanagasa-dori,Chuo-ku	1985	Shop	6	〃	Fracture and elongation of anchor bolt
24	〃	Nakayamate-dori,Chuo-ku	1985	Office	5	〃	Fracture of anchor bolt

(Pounding)

25	Severe damage	Mikagenaka,Higashinada-ku		Office	4	Square tube column, H-shaped beam	Remaining deformation

(3) Damage Investigation

a) Classification of the Damage of the Moment-resisting Steel frames

1)Damages of structures with columns of square-hollow-box sections

The damage is roughly classified into 3 categories; Those are the ones on the column bases and their surrounding parts, the one the beam-to-column connections and those combined ones.

i). Damages on the column bases and their surrounding parts

The details of those damages are mainly on the failure of the concrete part of column bases, that of anchor bolts (drawing out and fracture). The damage level is varied like heavy damage of the first story, inclination of whole buildings, only the damage of column bases, etc. These damages were often observed in the buildings, what we call pencil-type buildings, of which width is very small compared with the building height.

Building A (Kano-cho, Chuo-ward, Kobe city)

This is a 7-story moment-resisting steel frame, so called pencil-type building, with one span in the span direction and three spans in the ridge direction. In the first story, drawing out of anchor bolts (around 10 cm pull out from basement), failure of surrounding concrete, flaking of stone patched wall and partial damage of ALC wall were observed, while in the floors upper from the second story, only a slight crack was observed on finishing materials. (Photo 3.2.2.1)

Building B (Kano-cho, Chuo-ward, Kobe city)

This is a 4-story office building of moment-resisting steel frames in both directions using the first story as a parking space. Shear cracks were observed in the foundation concrete in upper from the ground level. Also on the border of foundation beam and the ALC wall, which is a finishing material, a crack was observed. A column was slightly moved toward the east side. (Photo 3.2.2.2)

Building C (Kano-cho, Chuo-ward, Kobe City)

This is a 7-story so called pencil-type shop building of the moment-resisting steel frames in both directions with one span in the span direction and two spans in the ridge directions. The quoin column of the first story was pushed out from the foundation and came out to the front road, but any bending deformation was not recognized. The number of the anchor bolts of the column in Photo 3.2.2.3 is 4. (Photo 3.2.2.3) The covering concrete of the middle column base of the first story was damaged and collapsed. (Photo 3.2.2.4) The lateral deformation was slightly observed in the stories upper from the second story, but the damage was concentrated in the first story.

Building D (Yamate-dori, Chuo-ward, Kobe City)

This is a 4-story so called pencil-type shop building of the moment-resisting steel frame structures in both directions with one span in the span direction and one span (partially 2 spans) in the ridge direction. The quoin column in the first story was pushed out from the foundation at the base plate and collapsed toward the road side (north side). The anchor bolts of the quoin column were all fractured. (Photo 3.2.2.5) The welding joints at the top of the column was also fractured.

Building E (Miyuki-dori, Chuo-ward, Kobe City)

This is a 8-story parking building of moment-resisting steel frame structure with diagonal braces. Though any damage was not recognized on the super structure, the first story column which has a brace was bent and buckled, and a tensile fracture was seen at the part covered by the foundation concrete, which supposed to be occurred after the local buckling. (Photo 3.2.2.6)

ii) Failure of welding joints of beam-to-column connections

This is the failure at the column joint of beam-to-column connections (column side of diaphragm, panel side of that and/or the both sides), or the failure at the ends of beams. Columns were suffered from a large deformation due to large story drift. There were some cases showing the fracture of fillet welding, and some other cases showing the joint fracture of complete penetration welding and the brittle failure happened in the vicinity.

Building F (Nakayamate-dori, Chuo-ward, Kobe City)

This is a 7-story so called pencil-type shop building of moment-resisting steel frame structures in both directions. Lateral deformation occurred from the first story to the third story, especially concentrated in the first story. At the beam-to-column connection of the top of the interior column in the first story, some cracks were recognized on the welding joints at the beam end as well as the column end. In the first and second stories, fractures were observed at the welding joints of the beam ends of the beam-to-column connections in the quoin columns. As shown in the Photo 3.2.2.7, the lower flange of the beam was directly welded to the panel of the beam-to-column connection. (Photo 3.2.2.7)

Building C (Kano-cho, Chuo-ward, Kobe city)

A horizontal crack was recognized, running through the entire surface of the panel of the beam-to-column connection which is at the top of the interior column in the first story. The crack was seen on the welding joint near diaphragm and was evaluated as a brittle fracture through observation of the fractured section. (Photo 3.2.2.8)

Building G (Shimoyamate-dori, Chuo-ward, Kobe City)

This is a 7-story building with moment-resisting steel frames in both direction. Fractures were observed at the top of the first story column and at the welding joints of the diaphragm in the beam-to-column connection. The column was also slipped at the top. (Photo 3.2.2.9)

Building H (Iwayanakamachi, Nada-ward, Kobe City)

This is a 4-story moment-resisting steel frame building consisted of square-hollow-box section columns and H-shaped beams. Fractures were recognized at the welding joints between the column, beam and diaphragm of the beam-to-column connection. The structure upper from the second story was collapsed. (Photo 3.2.2.10)

2) Damages of rusted buildings with thin plate elements

Buildings in this category are apartment house buildings and buildings for apartment houses and shops of less than 5-story, using thin sectional steel members. Most of them are old buildings and rusted severely. The columns and the beams are consisted of square-hollow-box sections, H-shaped sections, lattice assembling, etc. with thin plate elements. The sections of those are all small and light in height, width and thickness. Damage types are such as the first story collapse, tilt of the first and second stories and inclination of the entire building.

Building I (Chayano-cho, Ashiya city)

This is a 3-story apartment house building using lattice formed columns and beams with thin plate elements. The first story was collapsed. The upper stories remain sharply tilting. A lot of rusting was observed in the collapsed steel materials. (Photo 3.2.2.11)

Building J (Higashinada-ward, Kobe city)

This is a moment-resisting steel frame with braces, using thin sectional H-shaped steel columns and lattice formed beams. The finishing materials were almost torn down and the first story was enormously deformed. The steel frame materials rusted severely. (Photo 3.2.2.12)

3) Damages of buildings with diagonal braces

The damages were failures of braces and those of joint bolts.

Building K (Aoki, Ashiya city)

This is a 3-story building of which the first story was used for a parking space and the upper stories for offices. The structure is a moment-resisting steel frame of H-shaped columns in the span direction and diagonal brace system used by angle in the ridge direction. The damages were buckling of braces, fracture of the gusset plate from the column web, drawing out of the anchor bolts, tilt of the first story to the north side. (Photo 3.2.2.13) The gusset plate at the connection of the brace was torn off from the web of the column for lack of strength and a hole was observed in the column web. (Photo 3.2.2.14) It shows that a back up plate is necessary at the back side of the column web.

4) Enormous lateral deformation of upper stories of buildings

Enormous lateral deformations were observed in the upper stories or penthouses of buildings built before revise of the seismic regulation of the present building standard(1981). The external finishing materials were lath mortar and damaged quite heavily.

Building L (Chuo-ward, Kobe city)

This is a 7-story building with one span by two spans, using H-shaped sections for columns and beams. The ridge direction is a bracing system used by flat bar steel, and beams are connected to columns at their web by bolts. The flanges of beams are not welded to columns. The span direction is made by moment-resisting steel frames. The flat bar braces had a fracture in their joints (especially upper stories). The deformation of the double plate of the column joint was also observed on the 4th floor. (Photo 3.2.2.15)

5) Damages of super high rise buildings

Building M (Ashiya city)

This is a high rise mega-structured building consisted of columns of square-hollow-box sections, H-shaped beams, and braces of H-shapes. Those main structural parts were exposed. There were some columns of which middle part was fractured horizontally and the cracks extended to the braces. (Photo 3.2.2.16)

Building N (Kano-cho, Chuo-ward, Kobe city)

This is a 30-story building. No damage was observed in the exterior finishing materials and the base of the building, but a slight crack was observed in the interior finishing materials.

6) Damages caused by collision of neighboring buildings

Building O (Hobiki-cho, Chuo-ward, Kobe city)

This is a 8-story moment-resisting steel frame and so called pencil-type building with one span for the frontage, which is consisted of square-hollow-box sectional columns and H-shaped beams. The parapet edge of a 2-story building situated next to this building in the south side was collided with the third story column of this building to cause tearing down the finishing materials around the column. (Photo 3.2.2.17)

As another structural damage, a crack on the concrete of the column base was reported.

b) Summary of Damage Investigation

1) Damage of buildings built by square-hollow-box sections as columns

i) Damage of column bases and that of the vicinities

Moment-resisting steel frame structures with columns of square-hollow-box sections are often seen, located in narrow construction sites in cities, which are mostly multi-story buildings. As the most of them are also so called pencil-type buildings which have small number of spans, the large tensile force caused by a large overturning moment at the time of the earthquake applied to the columns. Therefore, the damage was concentrated into the column bases and the concrete around them. It was observed from big damages like the first story collapse to comparatively small ones like cracks in the column bases and the vicinity area.

ii) Failure of welding joints of beam-to-column connections

There are many cases observed at the welding joints of the beam-to-column connections of box sectioned columns and H-shaped steel beams, such as cracks and fractures at the welding joints between members and diaphragms of connections. The following causes for shortage of the strength at welding joints are pointed out such as lack of the welding size, wrong welding of the complete penetration, and improper welding, which means that some joints were actually welded by the fillet welding instead of the complete penetration welding. Some of damages were also showing brittle fracture. Those should also be studied the causes in the future.

2) Damage of buildings with thin structural plate elements rusted heavily

Those buildings have small and light sectional members, and have shortage both of strength and stiffness against lateral forces. Furthermore, the sectional area of the structural members have been reduced by heavy rust.

3) Damages of buildings with diagonal braces

The damages were buckling and tensile yield of braces in the long brace system by cyclic loading of the compressive force and the tensile force. This was also pointed out in the past disastrous earthquakes as one of damages.

4) Large lateral deformations on upper floors of buildings

In the former building seismic standard, the design seismic force for upper floors of buildings was set lower than the one of the present norms. Therefore, in the buildings designed according to the old standard, the strength (rigidity) of upper floors is lower in comparison with the one designed due to the current seismic cord.

5) Super high rise buildings

For the brittle failure of square-hollow-box section columns of Building M, further investigation should be needed especially on the materials and the construction.

6) Damages by collision of buildings located next doors

The damages by collision of the buildings which are constructed fully in each building site were reported. The structural regulation on story drift of structures under severe earthquakes should be investigated in the future.

Photo 3.2.2.1

Pull out of anchor bolts from the basement, crack of concrete around the anchor bolts, and a little damage of finishing materials above the second story.

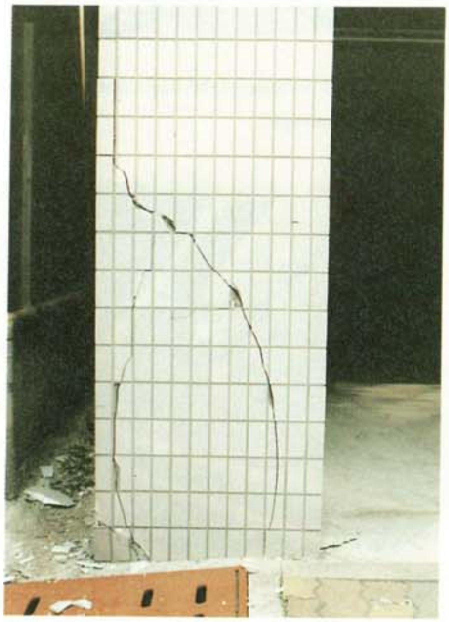


Photo 3.2.2.2

Crack of reinforced concrete foundation beam

Photo 3.2.2.3

The first story column at the corner was pull out from the foundation and moved to the front road, the reinforced concrete part around the column base was completely damaged.

Photo 3.2.2.4

The first story column was inclined because of the large story drift, and the reinforced concrete part of the column base was severely damaged.

Photo 3.2.2.5

Four anchor bolts of the first story column base was cut, and the structure was tilted.

Photo 3.2.2.6

The column with the square-hollow-box section was fractured in the foundation beam, where local buckling was observed.

Photo 3.2.2.7

The damage was concentrated into the lower three stories, especially into the first story. At the beam-to-column connection in the second floor, the welding joints were fractured. The lower flange of the beam was connected to the panel zone without diaphragm.

Photo 3.2.2.8

The panel plate of the beam-to-column connection in the second floor was fractured. The brittle crack was observed at the welding joint between panel plate and diaphragm.

Photo 3.2.2.9

The fracture of the welding joint between the column and diaphragm at the beam-to-column connection in the first story.

Photo 3.2.2.10

The welding joint between the column and diaphragm at the beam-to-column connection in the first story was fractured. The upper stories of the structure was collapsed.

Photo 3.2.2.11

The first story was collapsed. The structure has two stories of moment resisting frames with vertical braces of steel bars. The members rusted heavily.

Photo 3.2.2.12

Two storied house rusted severely.

Photo 3.2.2.13

The damage was concentrated into the first story. The story drift was 1/8.

Photo 3.2.2.14

The gusset plate was fractured by the tension force from the brace.

Photo 3.2.2.15

Large lateral displacement was observed in the upper stories. The long braces were fractured.

Photo 3.2.2.16
The fracture of vertical member with square-hollow-box section and H-shape brace.

Photo 3.2.2.17
Two buildings collided.

(4) Investigation of All Buildings in the Specified Area

From the 20th to the 23rd of February 1995 the all buildings investigation was done basically on taller or equal to 3 story buildings which were assumed steel frame structures in the following specified area of Kobe city except the ones of which main structural material was light-gauged steel. (Figure 3.2.2.1) The investigation was done mainly by a outward appearance investigation and each year of construction was unidentified.

Area	Date of Investigation	Number of Buildings
A part of Higashinada-ward	20/2, 21/2	171
A part of Chuo-ward	22/2	189
A part of Hyogo-ward	23/2	295

		Total 655

The investigation in Higashinada-ward was conducted together with the urgent investigation by the Committee on Earthquake Disaster Investigation on Buildings and the covering area includes 3 railways of Hankyu, JR and Hanshin. The investigation area of Chuo-ward covers from the north side of Sannomiya railway station to the Kobe Marine Meteorological Observatory. In Hyogo-ward, the investigation area is the area where the Municipal Subway, Kobe Express Railway and JR are passing through. This investigation is a collaborative work by the Building Research Institute, the Kozai Club and the Lath Sheet Kogyo-kai. The criteria of the general evaluation is as follows:

Severe damage:

Collapse

Unrepairable large deformed ones

Moderate damage:

Though the failure and the buckling of brace, and shifting to plastic range of columns and beams are recognized, the remained story drift is small and the damage is reparable.

Minor damage:

Though the cracks and flaking of exterior materials are recognized, any structural damage is not investigated.

The Table 3.2.2.9, 3.2.2.10 and 3.2.2.11 show the results of the investigation as the number due to the story, the one due to the utilization purpose and the one due to the damage level respectively. (The number in the parenthesis shows the ratio (%))

The number of the story slightly varies due to each area, the reason is considered as that in a part of Higashinada-ward and a part of Hyogo-ward, the ratio of residential area is large, while the one in a part of Chuo-ward is small. Totally, almost the half of the buildings investigated are 3 story and almost 85% of those are shorter or equal to 5 story.

As for the number of the buildings due to the utilization purpose, in a part of Chuo-ward, they are shops, residential houses and office buildings in order. On the contrary in a part of Higashinada-ward and in a part of Hyogo-ward they are residential houses, shops and office buildings in order. These results are also related with the ratio of residential area. Totally, the ratios are 35% for shops and residential houses, and 15% for office buildings. As for the damage level, the ratio is about 2:2:1:1 for No damage, Minor damage, Moderate damage and Severe damage in order. (Reference 3.2.2.1) According to the investigation (buildings of more than Minor damage) done by the Steel Structures Meeting in the Kinki branch of the Architectural Institute of Japan, the sum of the investigated steel structure buildings is 988 and 90 for collapse, 322 for Severe damage, 266 for Moderate damage and 300 for Minor damage out of the total number. Following to this result, the ratio of Minor damage, Moderate damage and Severe damage/collapse is approximately 2:2:3, which shows the lager ratio in Severe damage / collapse, and the smaller ratio in Minor damage than the ones in this investigation. In these three areas, the ratio of Severe damage increases in a part of Hyogo-ward, then in a part of Chuo-ward, and in a part of Higashinada-ward in order.

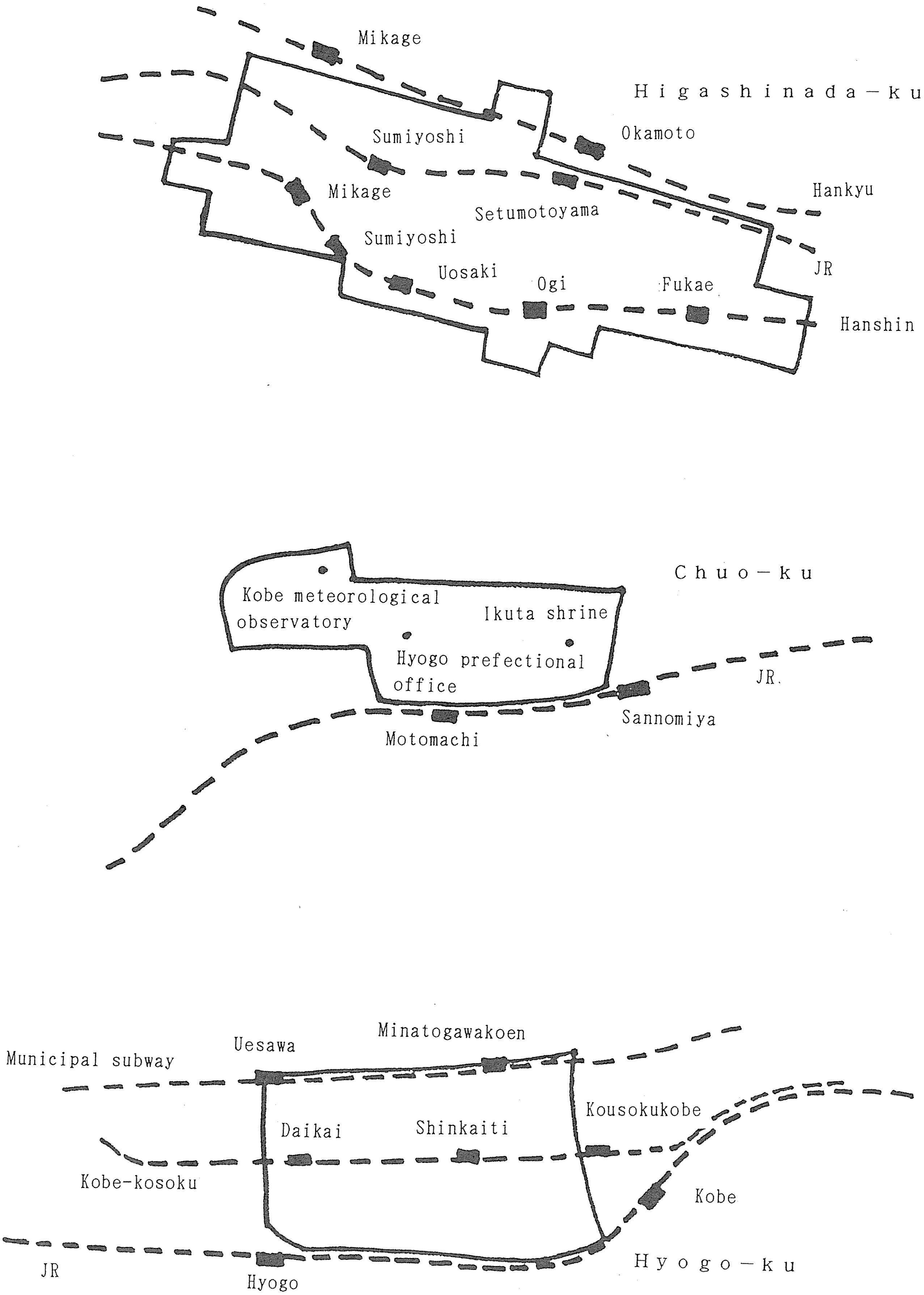


Fig. 3.2.2.1 Specified Area for the all buildings investigation

Table 3.2.2.9 The number due to the story

Story	Higashinada-ku	Chuo-ku	Hyogo-ku	Total
3	92 (53.8)	59 (31.2)	152 (51.5)	303 (46.3)
4	49 (28.7)	53 (28.0)	74 (25.1)	176 (26.9)
5	13 (7.6)	29 (15.3)	37 (12.5)	79 (12.1)
6	3 (1.8)	9 (4.8)	9 (3.1)	21 (3.2)
7	1 (0.6)	9 (4.8)	7 (2.4)	17 (2.6)
8	0 (0.0)	10 (5.3)	3 (1.0)	13 (2.0)
9	1 (0.6)	10 (5.3)	5 (1.7)	16 (2.4)
≧10	0 (0.0)	4 (2.1)	5 (1.7)	9 (1.4)
Uncerta-inty	12 (7.0)	6 (3.2)	3 (1.0)	21 (3.2)
Total	171 (100)	189 (100)	295 (100)	655 (100)

Table 3.2.2.10 The number due to the utilization purpose

Utilizatio	Higashinada-ku	Chuo-ku	Hyogo-ku	Total
Office	35 (15.8)	29 (12.2)	73 (16.7)	137 (15.3)
Shop	54 (24.3)	129 (54.7)	152 (34.7)	335 (37.4)
Residence	92 (41.4)	50 (21.2)	178 (40.6)	320 (35.7)
Hospital	5 (2.3)	3 (1.3)	5 (1.1)	13 (1.5)
School	0 (0.0)	1 (0.4)	0 (0.0)	1 (0.1)
Factory	10 (4.5)	0 (0.0)	8 (1.8)	18 (2.0)
Warehouse	13 (5.9)	0 (0.0)	7 (1.6)	20 (2.2)
Public office	0 (0.0)	2 (0.9)	0 (0.0)	2 (0.2)
Uncerta-inty	13 (5.9)	22 (9.3)	15 (3.4)	50 (5.6)
Total	222 (100)	236 (100)	438 (100)	896 (100)

Note: Including more than 2 utilization purposes per building

Table 3.2.2.11 The number due to the damage level

Damage	Higashinada-ku	Chuo-ku	Hyogo-ku	Total
No damage	60 (35.1)	72 (38.1)	98 (33.2)	230 (35.1)
Minor	33 (19.3)	52 (27.5)	128 (43.4)	213 (32.5)
Moderate	33 (19.3)	28 (14.8)	35 (11.9)	96 (14.7)
Severe	45 (26.3)	37 (19.6)	34 (11.5)	116 (17.7)
Total	171 (100)	189 (100)	295 (100)	655 (100)

(5) The Detailed Investigation of Specified Buildings

The detailed investigation of 9 structural steel buildings was done by the Building Research Institute with the cooperation of the Kozai club and the Lath Sheet Industrial Society, totally by 18 staff on the 24th and the 25th of February, 1995. Kobe city helped for the listing up the buildings to be investigated, but it was quite difficult to list up the severely damaged buildings to be investigated, because there was almost no public building showing such damage. Then the investigating group asked the Kozai Club to negotiate with the owners of the damaged buildings in which the inside investigation was possible. But the negotiation was not so succeeded, and therefore the detailed investigation was quite limited. Moreover, as the buildings with small damages were with fire protective covers for structural members and with interior and exterior finishings, the structural members were not directly able to be observed. The structural damage of these buildings were not grasped.

The followings are the summary of the detailed investigation of the damaged buildings.

a) Building A

This is a 5 storied office building consisted of square hollow section columns and H-shaped structural steel beams with 2 x 4 spans. The year of construction is 1984. By the visual inspection from the outside of the building, only some out-of-plane displacement is seen on the PC curtain wall and the building looks almost no damage with the remained comparatively small horizontal deformation of 1/100, while many neighboring buildings are inclined. However, by the inside inspection, at almost all the flanges of the end of the 2nd and the 3rd floor beams, the local buckling after full plastic yielding and the final fracture are observed. Photo 3.2.2.18 shows the damaged structural steel beam. According to a simple calculation such as the material strength is assumed as the specified minimum value, the ultimate lateral shear strength of the building is about 0.5 in the base shear coefficient. After this calculation, some coupon test specimen could be taken from the beam sections and a more precise evaluation of the ultimate lateral shear strength is planned in the future based on their test results.

b) Building B

This building has been utilized as a ward inhabitants7 center since 1992. The locating situation and the structural characteristics of the building is as follows:

Both sides of a 4 storied structural steel parking building, two 10 storied towers are built.
Just like linking the towers, a space for the office use is lifted of the towers by the gigantic trusses called" mega-truss "which connects the tops of the two towers.

As for the damage on the on-structural elements, the damage on the finishing materials for interior and exterior is observed, which was caused by the pounding of the lifted part by the mega-trusses in the middle with the roof of the parking building beneath. Photo 3.2.2.19 shows the damaged interior of the building. The damage on the structural elements could not be observed, because they were covered by the fire protective covers. Since any cracks and flaking are not observed on the fire protective covers, the structure itself is considered as no damage.

c) Building C

This is a 10 storied parking building (partially shops and offices)
The columns are square hollow sections and the beams are H-shaped ones. The K-braces or X-braces are placed in the both directions of X and Y. The year of construction is 1993. The main damages are as follows such as the shear yielding of a gusset plate corresponding to the "shear link" in the eccentrically braced frames, its shear buckling (partially cracked), and a local bucking of the H-shaped braces which were placed about their strong axes in the plane of the frame and were buckled out of the structural plane. Photo 3.2.2.20 shows the shear yielding of the gusset plate at the crossing point of the braces. Slippages of the high strength bolts of the braces are also observed. The damage level of the building is evaluated as "moderate".

d) Building D

This is a single storied rigid frame store house for keeping foodstuff, consisted of built-up box section columns 100cm in width and H-shaped structural steel beams. It was built in Heisei era, but the year of construction is unidentified. The damage condition of the structural components is not able to be evaluated because they are covered by the fire protective covers. However, any cracks and flaking are not observed on the interior and exterior finishings, which indicates no damage. Photo 3.2.2.21 shows the interior of the building.

e) Building E

This is a fire station building consisted of 4 story above the ground and 1 story in the basement, which are reinforced concrete structures, and of a hut on the top of the building, which is pure structural steel structure. The hut is fixed on the reinforced concrete structure by anchor bolts through base plates. The year of construction is 1990. Although the building is located in the severely damaged area, it has only a slight damage on the stairs and on the tiles of the exterior wall. The structural steel members of the hut are also no damage. However, the air conditioner on the 4th floor is moved by the failure of anchors.

f) Building F

This building called "clean center" is consisted of 5 storied super structure, are story in the basement and 2 storied penthouse. The structure is reinforced concrete or steel reinforced concrete up to 5th story above the ground, and combined structure of steel reinforced concrete with steel braces for the penthouse. The year of construction is April, 1990. There is no damage on the main structural elements, but is slight damages such as the falling down of the ceiling caused by the pounding of the main building with the administration building, and the damage on the expansion joints.

g) Building G

This building is utilized as a cultural center, which is consisted of one big hall, are middle hall, those entrances and some attached facilities. Each hall is constructed by the steel reinforced concrete columns which support the above structural steel trusses. The year of construction is unidentified. There is no single damage observed on the main structure, the trusses consisted of H-shaped structural steel and on the steel reinforced concrete columns, even without the cracks on the concrete. However, a very slight damage is seen on the non-structural components such as the non-structural wall and the ceiling.

h) Building H

This 7 storied with one story penthouse building is used as a center for children. The overall plan configuration is L-shaped and the structure is moment-resisting frames in both directions. The columns are square hollow sections and the H-shaped structural steel beams are flange welded and web bolted to the columns with interior diaphragm. For the column bases, the exposed and the embedded are both used. The year of construction is 1987. The damage of the structural components is not so serious but only the slight failure of concrete of exposed column base is observed. But in the middle floors, the damage of partition wall and the damage of the steel doors which are unable to open or close are observed. Also in the beams of the evacuation stairs, the local buckling and the flaking of paint due to yielding are seen. Through this observation, there must be some damages in the structural components inÅ@spite that the structural components can not be seen for the fire protective covers.

i) Building I

This education center is consisted of 10 story above the ground and one story in the basement. The plan configuration is an ordinary rectangular shape with a round shaped atrium at the corner of the building. Rectangular hollow sections are used as columns in the part of the rectangular plan and circular hollow sections are used as columns of the facade of the round shaped. H-shaped structural steel is used for the beams. The beam-to-column joint detail is the through diaphragm type. The year of construction is 1990. The structural components can not directly be observed, because those parts are covered by fire protective covers, but at the exterior beam-to-column joints of the 2nd floor and the 6th floor, the damage condition can be observed, because the finishing and fire protective covers are taken out at these locations. The flaking of the paint caused by yielding is observed on the beam web adjacent to the beam-to-column joints. There are also some welding fractures observed in the part of the partial penetration welding of the beams of the evacuation stairs. Other damages of the building are the out-of-plane displacement of a metal curtain wall due to the failure of the joints and some cracks on the partition walls.

The above is a brief summary of the 9 buildings and the damages investigated in detail. In this investigation, many of the buildings have comparatively small damage. This is because those buildings are mainly the buildings for the public use and was carefully designed according to the current seismic resistant regulation. Among those 9 buildings, the building A is rather severely damaged. The coupon test specimens were also taken from this damaged buildings to conduct the re-evaluation of the possessing ultimate lateral shear strength in the future.

Photo 3.2.2.18 Fractured lower beam flange after largely plastic yielding (Building A)

Photo 3.2.2.19 Damaged building interior due to pounding on parking building below (Building B)

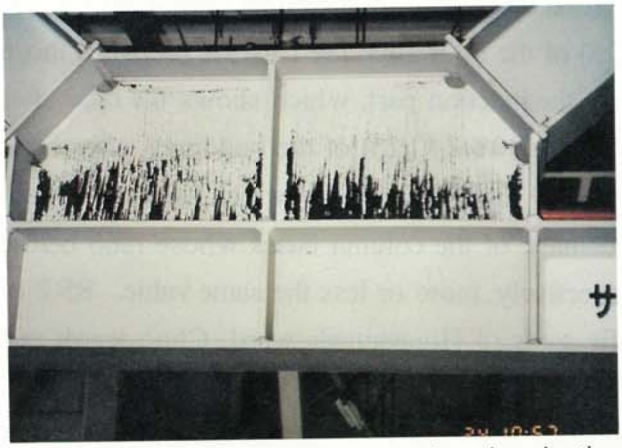


Photo 3.2.2.20 Shear yielding in the gusset plate at brace junction (Building C)

Photo 3.2.2.21 No damaged storage building (Building D)

Photo 3.2.2.22 Damage of exposed column base (Building H)

(6) Summary of the Investigation Result

Among 1231 buildings investigated by the urgent investigation project of the Committee for the Investigation of the Earthquake Disaster on Buildings, the number of the buildings which were designed according to the current seismic and had collapse severe damage is 25 (see Table 3.2.2.8). The main damages of those buildings are the fracture of the welded joints of the 1st floor column top and the elongation and the fracture of the anchor bolts of column bases. According to the investigation results shown in Table 3.2.2.1 - 3.2.2.7, the details of the damage level on 316 steel buildings and the ratio are 55% for collapse/severe damage, 16% for moderate damage, 27% for minor/light/no/ damage and 2% for unidentified damage. The result of the investigation on the buildings designed before the current seismic code (before 1981) and after the one (after 1981) and sustained collapse/severe damage (70 buildings before 1981 and 25 buildings after 1981) is the followings due to the damage part such as 4% (1/25) of the ones after 1981 vs 9% (6/70) of those before 1981 for the fracture of high strength bolts at the junction part, which shows the ratio after 1981 is decreased into the half of the ratio before 1981, 40% (10/25) of the buildings, after 1981 vs 23% (16/70) of those before 1981 for the fracture of the welded joints which shows the ratio after 1981 exceeds the one before 1981, and the damage of the column bases whose ratio before and after 1981 is 39% (27/70) and 36% (9/25) respectively; more or less the same value. 85% of all the steel buildings of 655 investigated in specific areas of Higashinada-ward, Chuo-ward, and Hyogo-ward in Kobe City is are 3 to 5 stories except the light gage steel structures. The ratio of the damage level in all areas is about 1:1:4 for collapse/severe damage, moderate damage and minor /light/no damage respectively. The ratio of collapse/severe damage in each area is 26% for Higashinada-ward, 20% for Chuo-ward, and 12% for Hyogo-ward The characteristics of the damage can be summarized as follows through the investigation results.

a) Damage to Column Bases and Its Vicinity.

There are 3 types of column bases, which are the exposed type, concrete encased type and embedded type. Much damage is observed in the exposed type column bases. Especially in the buildings so called pencil type buildings whose width is small compared with the height, the partial fracture of column base concrete, the failure of anchor bolts (pull out and fracture) and the fracture of columns around column bases are often seen. It is estimated that those pencil type buildings had the shortage of strength in the column bases against the large axial change by overturning moment.

b) Damage to Beam-to-Column Connections

Fracture is observed at welds parts of beam-to-column connections. Those damages are often observed in moment-resisting buildings with square tube columns and only the welded joints fractured without any plastic deformation of members. The typical fractured locations are the column-to-through-diaphragm joints, the panel-to-through-diaphragm joints, the lower beam flange-to-column joints, and beam web-to-column joints. The damage causes are assumed that the welded joints had the shortage of strength against large bending moments. Although the further investigation should be needed, the causes of the strength shortage at the welded joints are the shortage of welding size, inadequate fillet welding, etc. According to the investigation by the Architectural Institute of Japan (Ref. 3.2.2.1), since the through-column type frame is often taken for moment-resisting buildings with wide-flange columns, the damage at the beam end connections is often observed.

However, among the buildings whose beam-to-column connections fractured, there were many buildings which were recognized the fully plasticity of the beams followed by local buckling and yielding before the buildings are finally fractured at the beam and connections. These buildings seem to satisfy the seismic performance required by the current seismic code.

They should be separately considered from those buildings stated above, which fractured at the welded joints of the members without any plastic deformation.

c) Damage of Brace end Connections, the Column Splices and Beam Splices

The fracture at the brace end connections, the column splices and beam splices. As well as the fracture of the welded joints of beam-to-column connections, the connections fractured without any plastic deformation of the members. The damage cause of these damages is assumed as the inadequate detailed design of the welding procedures and the joint locations, and the construction conditions.

d) Brittle Fracture of Thick and Large Section Members

Brittle failure of thick and large section members are observed. The followings are assumed as the damage causes such as the shortage of fracture toughness of base metal, residual stresses due to weld restraint resulting from the built-up of structural members and the erection of steel frames, metallurgical stress risers resulting from tack welds or arcstrikes, and strain concentration at the locations of shape discontinuity such as joints and connection. However, those factors are still only in the range of assumption, and the further and the detailed investigation on the quality of materials, design methods, construction methods, welding effects, and the behavior of structures during earthquakes should be conducted.

e)Damage on the Buildings Using Light Weight Steel Designed by Old Code

Damage to buildings using light gage steel designed according to the previous seismic code is observed. The damage was enhanced by the decrease of the effective section area caused by corrosion.

(7) The Countermeasure

The countermeasures for the above characteristic damages are stated below, which are considered effective at present.

a) Column Bases

i) What especially be stated on the column base damage is the building with the anchor bolt extended in 10cm are seen at the exposed bases of columns. The damage is assumed to have been caused by the overturning moment of the earthquake to lift up partially the buildings. When such a big tensile force works on the column base especially at the exposed column base, the transmission of shearing force by the friction at the lower section of the base plate extremely slows down and the failure of anchor bolt as well as the transfer of the column base are occurred. This happens, because the design of column bases has based on the force working on the column bases at the final time of buildings. In the future, as well as the other junction parts, column bases should also be designed to satisfy the requirement for the possessing stress junction not to disturb the performance of toughness which the upper structures of buildings. That kind of review of designing is needed for the root covering column bases and the build-in column bases, not only for the exposed column bases.

ii) For the root covering column bases, considered non filling of concrete between the lower section of a base plate and the upper section of concrete, and the looseness at anchor bolts, safer to consider for the evaluation that either only th enroot covering reinforced concrete parts or the lower section of base plates can transmit the bending as well as the shearing force.

b) Connecting Parts of Beam and Column Junctions

i) Since the present seismic resistant regulation was enforced, the rigid structure consisted of H shaped steel beams and angle steel pipe columns has been widely disseminated. Many weld fracture are observed on and around the junctions of the columns and Through the junctions, rigid frame structures make the diaphragm form) The fractured places are on and around the welded parts of the angle steel pipe columns and diaphragms, and around the scallops at the edge of H shaped steel beam flange, which is welded with the diaphragms. Especially the failure at the welded points of the angle steel pipe columns and diaphragms occurs without any shift to plastic range on the angle steel pipe columns, if the weld is the fillet weld. As a countermeasure for those damages, the column penetration form can be applied for the column and beam since it is quite easy to adapt the column penetration form to the H shaped steel pipes used to used much before the establishment of the present standard, the method should be reevaluated. It is also possible to make the column penetration form by aparting the outer diaphragm, it angle steel pipe columns are used. Moreover, if even the penetration diaphragm form is applied, the performance of the welded parts can be outstandingly enhanced by the complete penetration weld not by the fillet weld for welding columns and diaphragms.

ii) On the other hand, the failure happened around the scallop at the H shaped steel beam flange edges does not make any problem, unless it happens finally after the full yielding of beam flanges and the collapse of a building never happens. However, it is much better to enhance the performance of the junction of the part, which also enhance the structural performance of beam components. For that, the following methods should be adapted such as the factory weld to reduce the potential effect of the notch around the backing strip, non scallop method, etc.

c) Brace Edges, Column Joints and Beam Joints

Many damages are also observed at brace junctions, column joints and beam joints. For the brace junctions, securing the possessing stress junction is basically inevitable. For column joints and beam joints, improving the construction quality and well knowing the knowledge of the detailed design points for joint positioning are essential.

d) Brittle Failure of Thick and Large Sectioned Components.

Several brittle and tensile failures occurred showing clear recognized. The damage cause factors have been discussed and pointed out by the experts, but none of those has not been proved yet. The countermeasures the proof of damage cause factors, improvement of design method, etc. should be urgently established.

e) Light Weight Section Steel

The countermeasures for the damages on the buildings which were built by using light weight section steel, for example, made before the establishment of the seismic resistant regulation of the present building standard.

Many old buildings are consisted of light weight section steel or thin H shaped steel and shortage of rigidity and strength. The damages in those buildings were multiplied by the secular degradation caused by rust.

As a countermeasure for those damages, the seismic resistant diagnosis and the reinforcement should be intentionally carried out. However, since the conventional reinforcement method forces the inside of buildings not to be used during reinforcement works, owners of buildings are often reluctant to the reinforcement. For accelerating the reinforcement, the development of a new reinforcement technology is desired, keeping the continuous use of the inside of buildings.

[References]

3.2.2.1)

" The Damage Investigation Report on Steel Structured Buildings caused by 1995 Southern Hyogo Prefecture Disastrous Earthquake " by the Steel Structure Panel of the Kinki Branch of the Japan Building Society, May, 1995.

3.2.3 Wooden Buildings

(1) Classification of Wooden Houses

Wooden houses in damaged area are briefly classified into the following three types according to the structure and the constructed period.

S1: Wooden houses having the clay wall and heavy tile roof with clay pad. In general they have very few or no diagonal braces.

S2: Wooden houses having the clay wall finished with the lath mortal and tile roof with clay pad. They have a certain amount of diagonal braces and continuous concrete foundation.

S3: Wooden houses having no clay wall sheathed with lath-mortal or sidings. Sometimes thermal insulation is used. Interior walls are generally sheathed with gypsumboard or lathboard. They have slates or tile roof without clay pad, and diagonal braces or plywood-sheathed shear walls.

It is estimated that houses of S1 type were constructed before or just after the World War II. Houses of S2 type might be constructed at the period from 1955 to 1975, and those of S2 were supposed to be constructed after 1975.

(2) Outlines of Damages

Damages of wooden houses in Nagata-ku("ku" means "ward" in Japanese), Nada-ku, and Higashinada-ku were investigated. Figs. 3.2.3.1 to 3.2.3.3 show the damages of each type of wooden houses in a block where wooden houses were severely damaged. Fig.1 shows that the number of S1 type houses was the highest in the investigated area of Nagata-ku, and followed by S2 and S3. In Nada-ku and Higashinada-ku, the number of S2 type houses was the highest, and followed by S3 and S1. The rate of heavily damaged wooden houses was approximately 80% in both types of S1 and S2. In S3 type, the rate of heavily damaged houses was 30 to 50%, and smaller than that of S1 and S2 in all the area. Here, the heavy damage is defined by the residual story drift of more than 1/20, the medium damage is defined by the residual story drift from 1/60 to 1/20, and the light damage is defined by the residual story drift of less than 1/60.

A large number of wooden buildings collapsed or were heavily damaged by this earthquake. Most of these buildings were one or two storey dwellings in which the structural calculation was not required. The degree and state of the damages depend on the kinds and the construction method.

Photo. 3.2.3.1 shows a typical example of the damaged wooden houses in Nagata district. Whole the building collapsed completely. This type of damage occurred mainly in old wooden houses constructed before or just after the World War II. Photos. 3.2.3.2 to 3.2.3.4 show the damages of wooden buildings whose first story collapsed. This damage took place generally in wooden houses constructed of the clay wall and heavy tile roof with clay pad. Some newly constructed wooden houses also collapsed as shown in Photo. 3.2.3.4. Having a garage in the first story, this building did not have the sufficient amount of shear walls, but also the shear walls were placed eccentrically in the first story. Photos. 3.2.3.5 and 3.2.3.6 show the damages of the comparatively new wooden houses having a reinforced concrete garage in the first story. The first level of wooden structure completely collapsed.

Photo. 3.2.3.7 shows the damages of a house whose first story was a shop. This type of buildings have generally a large space in the first story, and the disposition of the shear walls is eccentric. A number of buildings of this type were heavily damaged. Photos. 3.2.3.8 and 3.2.3.9 show the damages of the dwellings whose width was very narrow. This type of buildings have generally very few or no shear walls in front of the building. A number of dwellings of this type were also damaged.

Photo. 3.2.3.10 shows the damages of the exterior mortar. The lath-mortar has been used for the exterior in urban area since roughly 40 years ago because of the fire safety. In many houses, the exterior mortar was peeled off because of an inadequate connection between the lath-mortar and sheathings.

Photo. 3.2.3.11 shows a non-damaged three-storey house of conventional wooden post and beam structure. These buildings designed by the structural calculation showed in general very few or no damages even in high seismic area.

Photo. 3.2.3.12 shows the example of the damages of the foundation of wooden building. Although the damages of foundation in wooden building by this earthquake were comparatively small, the foundation of unreinforced concrete was damaged in some wooden buildings.

(3) Major Causes of Typical Damages

The majority of the collapsed wooden buildings were old houses of post and beam construction consisted of the clay walls and heavy tile roof with the clay pad. These buildings having no diagonal braces, or very few if any, had comparatively long natural period and insufficient lateral resistance to support the mass of the building. The insufficient reinforcement of the traditional tenon-type joints with steel plates, bolts, etc. is also one of the causes of the collapse. In some old houses, the length of the tenon was not sufficient but also the members were not connected in adequate way.

Some wooden buildings constructed recently also collapsed or were heavily damaged. Most of them were conventional post and beam construction having large openings in the first story. This design appears often in the dwellings whose width is extremely narrow or those whose first floor is a shop. As the shear stiffness of the floor diaphragm of conventional construction is generally low, a large horizontal displacement occurs at the front of the building which has very few or no shear walls.

The inadequate application of the diagonal braces is also one of the causes of the damage of wooden buildings. In most cases, braces were connected to the post and horizontal members such as a girder and a sill with only few nails, and the posts and the horizontal members themselves were not connected in an adequate way. These braces do not work sufficiently not only as tension member but also as compressive member.

The second story of some wooden buildings having a reinforced concrete garage in the first story were heavily damaged. It is supposed that the first level of the wooden structure was shaken excessively because of the difference of mass and stiffness between the reinforced concrete and the wooden structure, however further study should be done to verify this fact.

Generally speaking, wooden buildings constructed with the North American Wood Frame Construction Method, prefabricated panel structure, three storey wooden buildings designed by the structural calculation and well designed conventional wooden post and beam buildings resisted well against the strong earthquake.

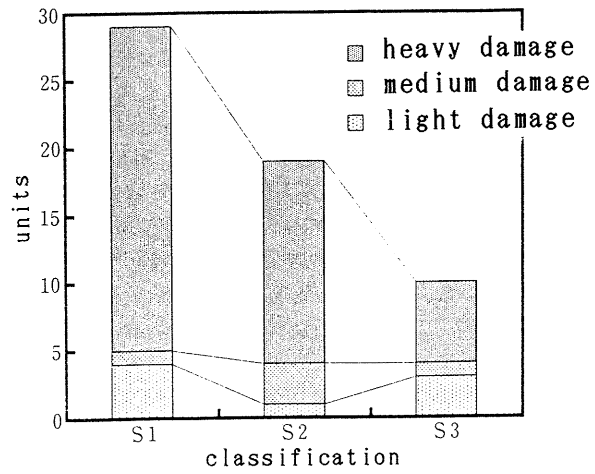


Fig.3.2.3.1 Damages of wooden houses in Nagata-ku

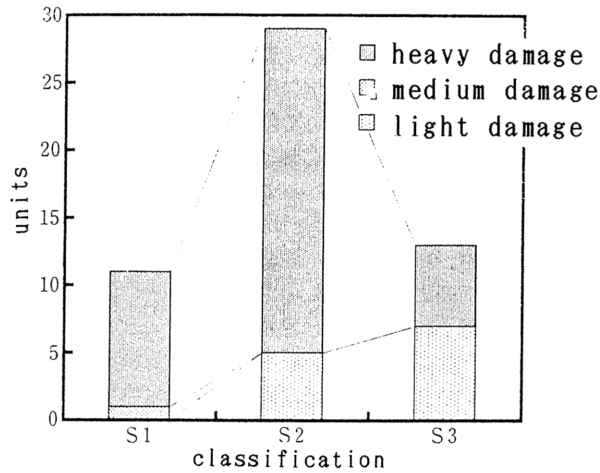


Fig.3.2.3.2 Damages of wooden houses in Nada-ku

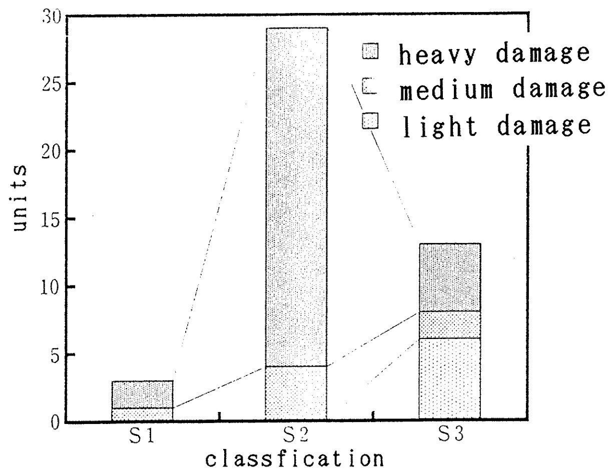


Fig.3.2.3.3 Damages of wooden houses in Higashinada-ku

Photo 3.2.3.1 Collapse of wooden houses

Photo 3.2.3.2 Collapse of the first story

Photo 3.2.3.3 Collapse of the first story

Photo 3.2.3.4 Collapse of the first story

Photo 3.2.3.5 Collapse of the second story
(a garage of RC in the first story)

Photo 3.2.3.6 Collapse of the second story
(a garage of RC in the first story)

Photo 3.2.3.7 Building with a large opening

Photo 3.2.3.8 Wooden building with a small width

Photo 3.2.3.9 Wooden building with few shear walls

Photo 3.2.3.10 Damage of the exterior lath-mortar

Photo 3.2.3.11 Three storey wooden building without damages

Photo 3.2.3.12 Damage of the foundation

3.2.4 Foundations and the Ground

(1) Special Features of the Damages

It is difficult to specify the damage of building foundation through spread observation. At first judging by the inclination of the building is done. In case of need, following survey is carried out; check of pile top, non-destructive inspection and taking a photograph of inside piles. However it is impossible to grasp all damages of foundation structures until now. The special features of the foundation damage are stated below from the surveyed data so far.

a) Spread Foundation

The damage pattern of the buildings supported by spread foundations is mainly the inclination and subsidence of the building. In the big buildings, there are few damages in foundation footings. However in small scale buildings such as private houses, the troubles of foundation beams and footings were observed.

b) Pile Foundation

The damage of steel pipe piles is hardly observed because of rare cases of their use. At the seashore area where liquefaction occurred, some of steel piles were pulled up, however severe damages of such damages were hardly seen. Some reports showed plastic deformation at the pile top due to flexural moment. In the cast-in-place reinforced concrete piles, it is reported that some piles top showed bending cracks and shear cracks. The compressive crush of pile top was observed a little. There were some cases that longitudinal reinforcement of the pile were ruptured at pile top. In ready-made concrete piles, severe damages were seen at pile top regions,such as compressive crush, shear crush and flexural rupture and vertical split. At the seashore area, furthermore above mentioned, flexural rupture at deeper parts of the piles and their pulling out were observed.

c) Soil Improvement

At the site where the soil improvement such as sand compaction work etc.was done, there were few or slight damages to buildings and their foundations.

d) Ground and Retaining Walls

At the land for housing on the foot of Mt. Rokko, the grounds slid to relatively loose slope direction which caused the severe damage to retaining walls and land for housing. Among retaining walls, masonry structured wall were severely damaged and reinforced concrete retaining walls were slightly done. And at reclaimed area near seashore, subsidence occurred due to liquefaction in wide area.

(2) Main Causes of Damages

a) Spread Foundation

The main causes of the damage of spread foundation is due to plasticity of soil and liquefaction of foundation ground.

b) Pile Foundation

It is often observed that severe damages of piles occurred due to soil liquefaction and land slide at sloped area. There are some cases that lateral remaining deformation of soil is more than 50 centimeters. On the ground without liquefaction, the piles were damaged also by inertia force from the super structure due to vibration. Lateral shear carrying capacity of piles and their ductility seemed to be insufficient.

(3) Countermeasures

a) Urgent Measures

i) Active enforcement of liquefaction countermeasure

ii) Examination of lateral force carrying capacity of piles

iii) Enforcement of seismic design to retaining walls

b) Long-Range Countermeasures

i) Careful consideration of building site condition

ii) Consideration of the effect of soil condition for piles

iii) Recommendation of spread foundation by soil improvement

iv) Enforcement of soil Improvement at reclamation area

Surveyed records are shown below as Figures and Photographs .

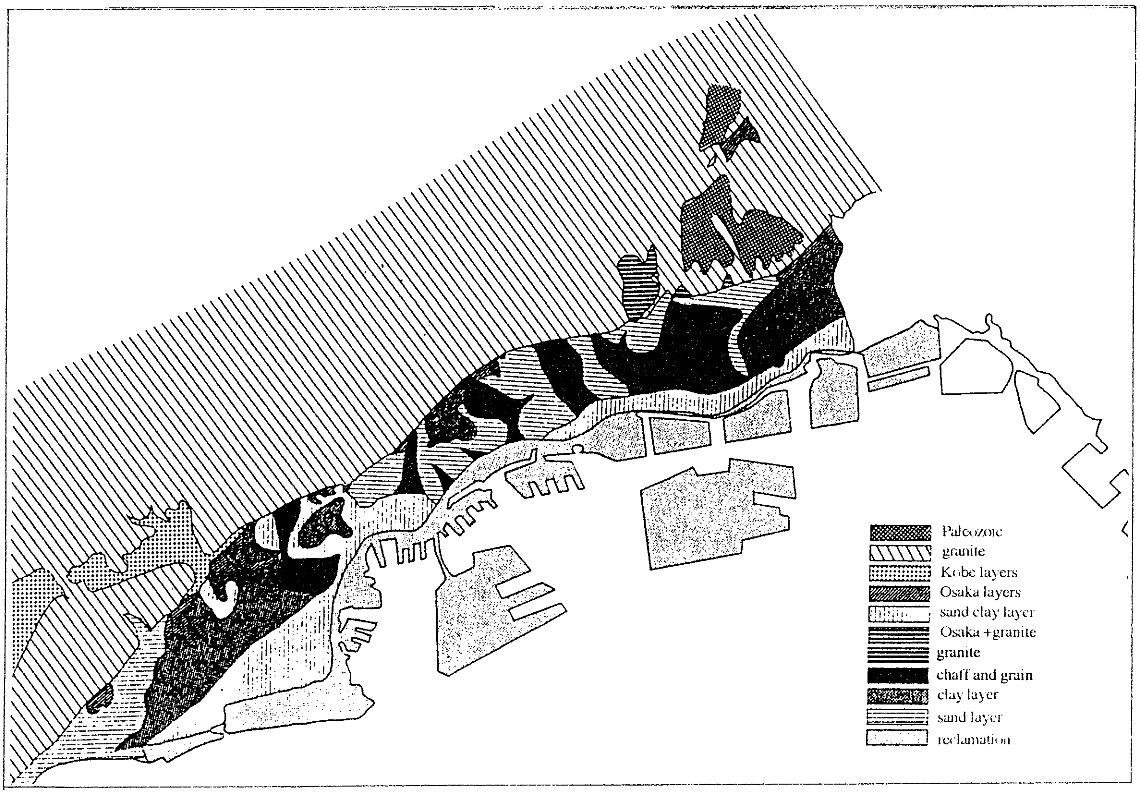


Fig. 3.2.4.1 Outlines of Surface Soil Condition