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STATE-OF-THE-ART SEISMIC EVALUATION OF REINFORCED CONCRETE BUILDINGS BASED ON BEAM-COLUMN JOINT CAPACITY.

PART 2: Application to Damaged Buildings During the January 17, 1995 Great Hanshin Earthquake

Tomoaki AKIYAMA, Masaya HIROSAWA and Silvestre BERSO

Structural Investigation Division
Tokyo Soil Research Co., Ltd.
2-11-16 Higashigaoka, Meguro-ku, Tokyo 152, JAPAN

ABSTRACT

In Japan the observed damages in the beam-column joint of reinforced concrete buildings are very limited, however, the January 17, 1995 Hyogo-Ken Earthquake (Called as Great Hanshin Earthquake) caused widespread damage to moment resisting joint panel of buildings.

This paper presents an extensive investigation of the joint panel in the damaged buildings.As a result of this investigation, many buildings designed according to the requirement ofthe current Seismic Code were severely damaged in the joint panel,in addition to slight damages in columns and beams mainly in the longitudinal direction of the medium-rise apartment buildings.

Also, the seismic performance of an eight-story building with respect to the joint shear capacity which was extremely damaged in the joint panel is investigated using the simplified method proposed in Ref[1].

From this result, it was made clear that the seismic performance of the damaged buildings are comparatively low in joint panel which explains the actual damage condition of buildings undergone a strong ground motion.

KEYWORDS

Beam-column joint, failure mechanism, base shear coefficient,ductility index, steel reinforced concrete (SRC)

INTRODUCTION

In the Hanshin-Awaji Great Earthquake, several damaged buildings similarly observed in thepast such as shear failure in column, buckling of steel braces and damage to steel column base plate were observed in addition to new types of failure patterns such as the many examples of shear failure in the beam-column joint of reinforced concrete group buildings.

According to the recent research, the level of seismic performance of a moment-resisting frame considering the shear failure in the joint panel under seismic load is comparativelylow. The factors that contribute to poor seismic performance of buildings failing in shearat the joint panel are the followings:

a) Low total column-to-floor area ratio
b) Low beam depth-to-story height ratio
c) Large amount of beam reinforcement or used of high strength material
d) Used of deformed bar in reinforced concrete structure
e) Low compressive strength of concrete

On the other hand, the buildings with severe damage in the joint panel caused by this earthquake are those which were designed according to the current provisions and also those medium-and high-rise housing structures damaged in the longitudinal direction.

In this paper, the examples of buildings failed in beam-column joint panel and the inves- tigation on possible factors affecting the joint shear failure are presented. And also, applying the simplified evaluation method proposed in Ref[1] and the results of the investigation of the seismic performance of an eight-story building damaged in this earthquake are discussed.

JOINT PANEL DAMAGE DURING THE PAST EARTHQUAKE

Most of the damages inflicted on reinforced concrete buildings are shear failure in columns and shear walls, and few examples of buildings which suffered beam-column joint failure. However, damage of beam-column joint failure are easily occurred in buildings with low horizontal rigidity,i.e., slender columns and beams. Experimental data also show many examples where drift angle of about 1/50 radian produced large horizontal deflection.

As one of the reasons why there are few examples of joint panel damage is because joint panel damage is easy to occur in medium-rise moment resisting frame buildings with low total column to floor area ratio, and on these buildings with few secondary walls which have never been experienced a very strong earthquake. Under this conditions the followings are several example of severe failure of joint panel on reinforced concrete buildings during the recent earthquake. Photo 1 shows the seven story building in Mexico City which was damaged just before its demolition during the 1985 Mexico earthquake. Photo 2 shows the three story building collapsed while under construction due to 1990 Philippine, Luzon earthquake. As shown in these photos damaged building are all moment resisting frame structures and the cross-sections of columns and beams are comparatively slender than used in Japan.

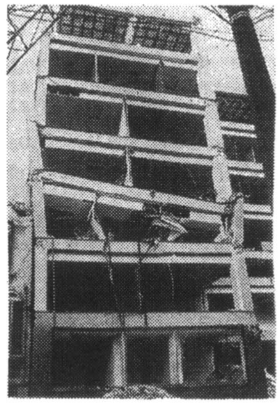


Photo 1. Single span seven-story building showing the heavy damage at the joints of the third floor due tothe 1985 Mexico earthquake

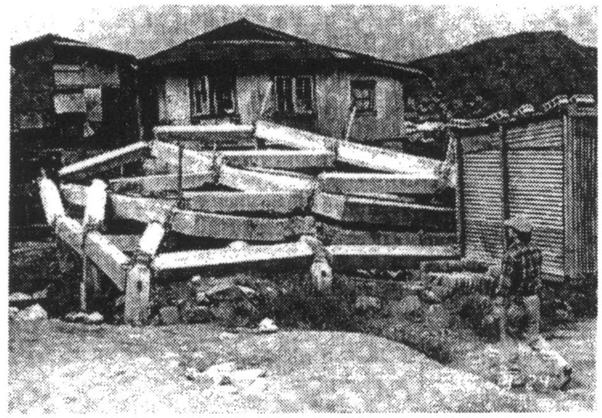


Photo 2. Three-story building with inter-story collapse due to the severe deterioration of the joints during the 1995 Philippine, Luzon earthquake

Compared to these collapsed buildings, Japanese reinforced concrete buildings, except high-rise buildings in the center of large city, have several secondary walls and the beamand column cross sections are large. Therefore, examples of joint panel failure of building in the past were only few.

INVESTIGATION OF DAMAGED BEAM-COLUMN JOINT PANEL OF RC GROUP STRUCTURES

Buildings with Joint Failure

Examples of joint failure are gradually increasing and the published data in Refs.[4] and [5] should be given a careful attention. In Ref.[4] the outline of report on damage degreeof investigated buildings by the volunteer action group and detailed investigation are described. In this report 86 buildings were inspected and nine buildings were reported with joint panel damage, especially, the six buildings out of 26 reinforced concrete private housing structures which were designed according to current code. While in Ref.[5] field experiment and detailed analysis on a nine-story RC building constructed 10 years ago, and suffered severe damages in most of the joint panels are reported.

In addition to the above data, more than 20 buildings with damage joint panel investigatedby each organization reported in Joint Committee of AIJ are included. From these data, the related statistics of buildings with joint damage reported in Ref. [4] are summarized in Table 1, and the typical examples of damage are shown in Photos 3 and 4. According tothese data, buildings with joint failure correspond to the following items:

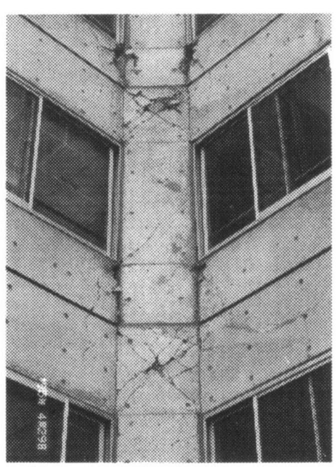


Photo 3.a A panoramic view of an eight-story RC housing building showing the joint damage in different locations Photo 3.b. Close-up view of the damaged joint condition at the reentrant corner

Photo 4. An 11-story SRC City Office building with damaged secondary walls and severe damage of joints

a) In most buildings constructed after 1982 are designed and constructed in accordance with the current standards.
b) In most medium-rise SRC or RC structures from 6- to 14-story with pure moment resisting frame.
c) In buildings with one span or two spans in short direction,and two to six spans in the long direction and unit weight per floor area is large.
d) In a building with widespread joint panel failures
e) In building with remarkable damage occurred in the short direction of single span building.

Table 1. Examples of the beam-column joint panel failure of RC group structures (Investigation result of the Volunteer group JSPROVER)

Building name	Location	Structural type	Number of stories	Number of spans	Long.direction	Construction year	Seismic intensity	Damage degree (Construction)	Failure degree	
									Column	Joint
K.A.-A apartment	Higashinada-ku	SRC-RC	11/0	5×1	N-S	After 1982	7	Severe	V	III
K.A.-B apartment	Higashinada-ku	SRC-RC	11/0	7×1	E-W	After 1982	7	Medium	V	II
A.P.-N apartment	Hyougo-ku	SRC	13/0	2×1	N-S	After 1982	7	Medium	II	IV
G.P.-N apartment	Hyougo-ku	SRC-RC	10/0	4×1	N-S	After 1982	7	Severe	IV	III
G.P.K.building	Hyougo-ku	SRC	14/1	7×2	N-W	After 1982	6～7	Medium	IV	II
R.R.apartment	Nada-ku	SRC-RC	10/0	4×3	N-S	After 1982	7	Medium	II	II
K-2 high.apartment	Nada-ku	HFW	10/0	7×2	N-S	After 1982	7	Severe	IV	II
H primary school	Amagasaki-city	RCF	4/0	31×1	E-W	Before 1971	5	Medium	III	I
J.R.apartment	Nada-ku	RCF	6/0	6×1	N-S	After 1982	7	Severe	II	IV

Photo 5. A single span RC building with heavily damaged exterior joint at 2nd floor

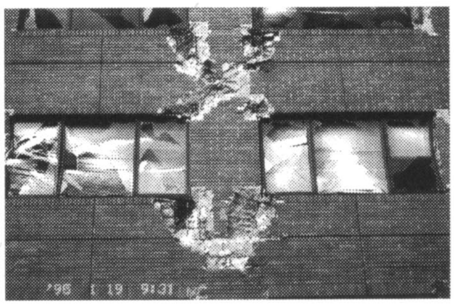


Photo 6. Cross diagonal compression cracks on the interior joint of 11-story SRC building

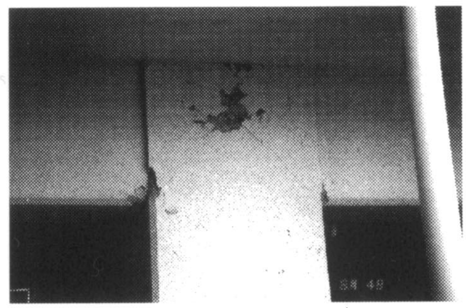


Photo 7. Severe compression failure on the interior joint of an 11-story SRC building at the 3rd floor, although the cracks were not too wide.

Failure Pattern and Condition of Joint Panel

Damage degree of joint panel as shown in Photos 5～7 is deviated from slight cracking toa total collapse.
a) Remarkable failure in (┣) -type joint of single span building [Photo 5].
b) Remarkable failure in (╋) -type joint of multi-span building [Photo 6].
c) Many wide diagonal cracks in compressive zone of (╋) -type joint [Photo 3.b].
d) Visible cracks in compression zone of (╋) -type joint [Photo 7].
e) Relatively visible cracks pattern.

Moreover, in these joint damage examples as shown in Photo 6, even the frame is infilledby secondary walls on beam and column side, a deterioration of the secondary wall is followed by damage in joint panel. Thus, it can be seen that secondary walls did not have themuch effect to prevent the damage of the joint panel at all.

Fig.1 is the damage condition of a nine-story RC building with one by five span in the North-South direction of the longitudinal frame as shown in Photo 3.

From these data, the damage degree of joint panel can be enumerated as follows:
a) The extent of damage of joint panel is relatively extended not only on the horizontal direction but also in the vertical direction.
b) Severe damage in the interior column than the exterior column.
c) Very intense damage from the second floor to lower floor
d) Damage of joint panel on external frames is more apparent than the interior frames

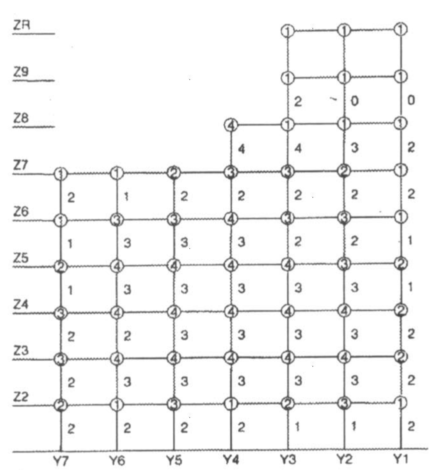


Fig.1. The failure distribution of each joint panel and column of a nine-story RC building in the North-South direction of the longitudinal frame (East side frame of Junes Rokko)

Investigation of Joint Panel Damage

Investigation Method. In the investigation of reinforced concrete beam-column joint, the stress in the joint panel is first calculated. Then, the cracking strength and the shear strength are determined, and from these values it can be determined if cracking or shear failure occurred. Since stress of joint panel varies with external forces, in order to specify a maximum joint panel stress, generally beam main bar is stressed at yield point. Therefore, in order to decide the relationship between stress and capacity, it is necessaryto know the actual strength of reinforcement and concrete. And in the calculation of tensile yielding stress in beam top bar, the slab reinforcement shall be considered. Consequently, it is important to decide the effective width of floor slab to determine the truebeam moment capacity, but still the slab contribution is not yet clear.
Based on this consideration, the investigation of the actual damage building can be quantified using the above values. In relation to the recent damaged buildings these data are so limited, only the building called Junes Rokko provides the necessary data. The result of investigation based on actual capacity of joint panel of this building are described.

Junes Rokko Building. This is a nine-story RC building without basement with one span in East-West direction and six spans in North-South direction. This building also has a partial infilled shear walls in the longitudinal direction of first floor and at both end frames along the short direction, and ALC (Auto-clave Lightweight Concrete) wall is used as partition non-structural wall. The lateral load resisting system consists of moment resisting frames designed with structural performance coefficient, $D_S = 0.3$.
This building suffered very severe earthquake motion in the North-South direction causing flexural yielding at the base of the ground floor columns and beams at all floors. Many visible cracks in slab connecting the flexural cracks in beams appeared only in East-West direction, and concrete crushing developed at beam soffit. In majority of the beam-column joint panel of the interior column from third to fifth floor, 2 mm. wide cross type diagonal shear cracks were formed, and concrete crushing developed on the cover concrete.
From the inspection of damage in 3rd floor of interior column the followings are obtained:

Beam b× D = 50 × 85 - cm 7-D25 Top Bar (8-D13 slab reinforcement) 6-D25 Bottom Bar
Column b × D = 80 × 70 - cm. (with beam eccentricity, bj=67.5 cm.)
Column Axial Load N=137.0～175.5 t (Average = 156.0 t)

The result of calculation of stress at the failure mechanism ofthe interior joint at the 3rd floor (V_j), cracking strength (V_{jsu}) and shear strength capacity (V_{jsu}) are summarized in Table 2. And the calculation was based on the following three cases.

Case 1: Material strength is assumed as the design strength
Case 2: Material strength is assumed as the actual strength (reinforcement is at yield strength)
Case 3: Material strength is assumed as the actual strength (reinforcement is at ultimate strength).

A ratio of V_{jsc}/V_j about 0.75 as shown in Table 2, shows that shear crack will form at joint mechanism. On the contrary, the joint panel shear capacity in which the actual strength at yield point are 1.5 times of the shear stress at the joint mechanism shows that joint panel will not fail at joint mechanism. However, the joints in actually suffered severe damage, thus, the calculated values did not agree with the actualresult. However, further investigation revealed that reinforcement experienced a very large strain up to the maximum strength which caused the joint panel reached its maximum capacity. As discussed in Ref. [5], a very large residual strain was observed in the test specimen obtained from the site and the beam tensile reinforcement reached the strain hardening zone. This investigated joint panel stress condition is consistent with Case 3 assumptions and the effective width of floor slab extended widely.

Table 2.
The shear crack strength (V_{jsc}) and the shear capacity (V_{jsu}) of the joint panel of the interior column of third floor (Junes Rokko)

Case	Strength of material (kg/cm²)		V_j	V_{jsc}	V_{jsu}	$\frac{V_{jsc}}{V_j}$	$\frac{V_{jsu}}{V_j}$
	Fc(σ_B)	(σ_s):D25(D13)	(t)	(t)	(t)		
Case 1	240	3,850(3,300)	235.9	170.6	306.2	0.72	1.30
Case 2	278	3,680(3,600)	232.0	180.2	354.7	0.78	1.53
Case 3	278	5,255(5,370)	334.8	180.2	353.7	0.54	1.06

Presumption on the Damage Condition of Joint Panel

There are few buildings in which the joint panels were confirmed damaged by this earthquake, roughly 20 to 30 buildings. However, based on the following reasons, it can be presumed that there were many slight damaged buildings and extensive damaged buildings with the joint panel deterioration.

a) Base shear coefficients at joint panel failure simply calculated based on usual crosssectional dimensions of column and beam (C_o =0.3～0.4) are very small compared to the standard shear coefficient C_o =1.0.
b) Since damage of joint panel is difficult to ascertain, specially in case of slight damage, it is generally overlook at the judgment of damage degree.
c) Joint panel failure and cracking are generally occurred not only in one portion but all over the building. Accordingly, if damage degree become very severe, it resulted in total collapse of the building. In building with very severe damage or total collapse, a possible joint panel failure can be considered.
Based on this consideration, the buildings collapsed or heavily damaged which were designed under current seismic provisions with complete detail of column dimensions and plans asreported in Ref.[5] were investigated. From the analyses, it was clear that majority of the buildings suffered very severe damage on the top of column and extended up to joint.
As shown in Table 3, base shear coefficient at joint panel shear failure of each buildingsare calculated. However, the dimensions of beam and actual strength of concrete were not obtained except for two examples . The values within the parenthesis in Table 3 are assumed. As a result, the following items are obtained:
a) The values of CB are mostly less than 0.5 except two building No.6 and No.9, and these values are lower than the assumed lower limit $F_s \cdot D_s$, which is 0.45.
b) The buildings with joint damage rank of IV did not experienced total collapse, and the damaged occurred along the North-South direction having multi-spans. On the contrary, buildings with damage on top of column are single span which were heavily damaged along North-South direction.
a) The concrete spalling occurred not only on the top of column of single span building but extended up to the portion of joint panel, and column intermediate reinforcement were buckled.
In the case of single span building listed in Table 3, majority are medium- and high-rise buildings and effect of vertical ground motion is significant. However, all the damaged beam-column joint are (├) -type, and in the case where the values of column to floor arearatio, a_c and the ratio of beam depth to story height, q' are low, the observed damage on top of column may be influenced by joint panel deterioration. Based on this investigation, further detailed analysis is necessary.

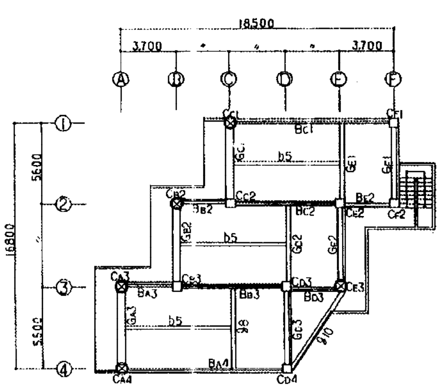
Table 3. Base shear coefficient at the joint panel shear failure of the buildings built by the current seismic code and suffered heavily damage on top of first floor column and/or the joint panel.

No.	Bldg. name	Location	Construction year	Structure	Number of stories	Number of spans	$b_c \times D_c$ (cm)	α_{c1} (cm^2/m^2)	q	Fc (σ_B)	W (t/cm^2)	σ_{ij}	C_B	Existence of pilotis	Damege degree (Failure member)
1	F. bldg.	Chuuou-ku	1978	SRC	10/1	5×3	80×80	15.4	(0.25 ～0.3)	225	(1.2)	1.86	0.21 ～0.25	No	Severe (Joint: Ⅳ)
2	W. apart.	Nada-ku	1986	RC	7/0	1～2 ×3	85×85	49.5	(0.2 ～0.3)	210	(1.3)	1.63	0.39 ～0.47	All	Collapse (Clum: Ⅴ)
3	N・I. house	Nada-ku	1990	RC	5/0	1×1	60×60	45.7	(0.25 ～0.3)	210	(1.4)	1.01	0.21 ～0.25	All	Collapse (Clum: Ⅴ)
4	S. house	Chuuou-ku	(1988)	RC	10/0	1×1	90×90	61.1	(0.25 ～0.3)	210	(1.4)	1.01	0.28 ～0.34	All	Collapse (Clum: Ⅴ)
5	S. bldg.	Nada-ku		RC	4/0	(1×2)	65×65	115.0	(0.25 ～0.3)	210	(1.4)	1.09	0.53 ～0.64	Part	Severe (Clum: Ⅴ)
6	FY. bldg.	Hyougo-ku	1986	RC	7/0	1×6	70×70	31.8	(0.25 ～0.3)	210	(1.3)	1.16	0.18 ～0.21	Part	Severe (Clum: Ⅴ)
7	KO. bldg.	Higashinada-ku	1985	RC	6/0	1×8	68×68	52.0	(0.25 ～0.3)	210	(1.3)	1.18	0.30 ～0.36	Part	Severe (Clum: Ⅴ)
8	HA. house	Hyougo-ku	1988	RC	10/0	1×6	100×90	39.2	(0.25 ～0.3)	225	(1.3)	1.16	0.24 ～0.29	Part	Severe (Clum: Ⅴ)
9	JR. apart.	Nada-ku	1985	RC	9/0	6×1	80×80	67.5	0.225	240 (272)	1.78	1.73 (1.16)	0.60 (0.40)	No	Severe (Joint: Ⅳ)
10	CA. apart.	Nada-ku	1985	SRC	11/0	5(6) ×1(2)	65×80	31.2	0.25	240	1.37	1.73 (1.17)	0.40 (0.27)	No	Severe (Joint: Ⅲ)

※ : The result of East-West direction reversed North-South direction suffered damage.

INVESTIGATION OF AN EIGHT-STORY RC BUILDING

<u>Building Description.</u> This eight-story apartment houses without basement built in 1985 has irregular shape in plan as shown in Fig.2. Basically one span in the North-South direction and two or three spans in East-West direction. There are infilled structural walls along the X-direction of the ground floor in Frames 3 and 4, but in Y-direction there are very few walls becauseof openings, hence, along this direction perfect moment resisting frame can be considered. Moreover, the story height of each story ranges from 2.7 m. or 2.8 m. and total height from the base to the top story is 22.35 m.



⊗ :Joint panel with a possibility of shear failure

Fig.2. Framing plan of typical floor

Damage Description. The typical damage condition at first floor column and joint panel at second floor are shown in Photo 8. As revealed in this Photo, concrete cover portion of beam-column joint were spalled-off, and critical damage by buckling of main bars was observed. These damage conditions were observed in every direction mainly in Y-direction of first and second floor joint panel. In addition there were several columns at first floor with compression failure on the top of column and base. Based on this damage condition, the damage degree was estimated as "collapse".

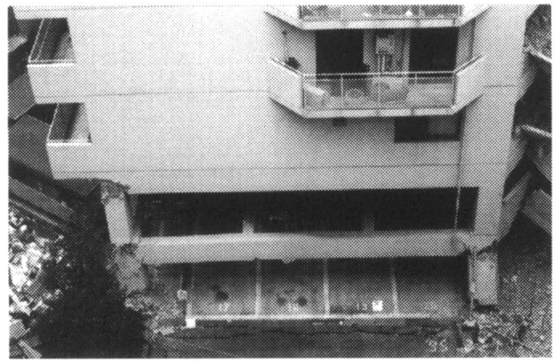


Photo 8. Actual damage between the ground and second floor joint and the second and third floor joint

Investigation by Second Level Seismic Diagnosis. Secondary level seismic diagnosis was carried out from the first to eighth floor of the building in X- and Y-direction Ref.[8]. The prediction of structural system seismic performance is determined by comparing the estimated structural seismic index Is expressed in Eq.1, with the required structural seismic performance index I_{so}.

$$I_{so} = E_o \times S_D \times T \qquad (1)$$

where:

E_o-subindex for basic structural performance is calculated using the ultimate strength index (C) and ductility index (F).

S_D-subindex for the structural configuration of building which is take into account the effect of irregularity of structure, stiffness or irregular distribution of mass.

T-subindex for time dependent deterioration of building.

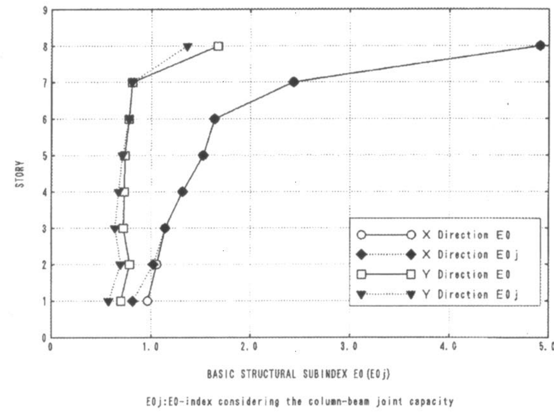


Fig.3. Distribution of E o -index and E o j -index considering the joint panel capacity

Index I_{so} is expressed in Eq. 2 based on the principle that each building shall have a comparable seismic performance as the building designed by the current seismic provisions.

$$I_{so} = E_s \times Z \times G \times U \qquad (2)$$

where:

E_s-seismic basic index $E_s = 0.6$

Z-zone index considering seismicity of site Z=1

G-ground condition index considering soil-structure interaction G=1

U-Usage index considering the importance of building U=1

The computed I_{so}=0.6 is decided in this building. Consequently, if I_s is greater than 0.6 this building is judged as SAFE against the designed earthquake motion assumed (300 gal ~ 400 gal).

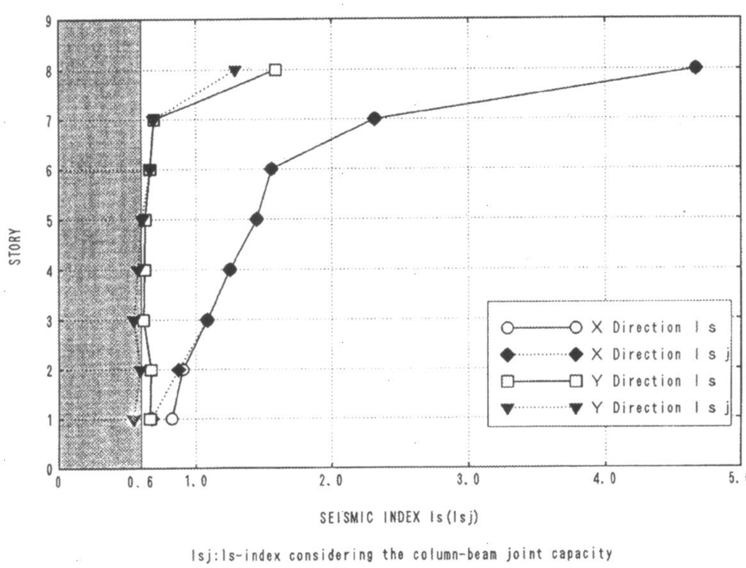


Fig.4. Distribution of Is-index and Isj-index considering the joint panel capacity

As a result of seismic diagnosis E_o index distribution is indicated in Fig.3 and Is index distribution is shown in Fig.4. As revealed in these figures, I_s values of each story in each direction are all greater than I_{so} =0.6. With this value, seismic performance based on capacity and ductility ability of vertical members were judged excellent. However, as described in the latter section this building suffered very severe damage mainly in Y-direction at the ground floor. Considering the seismic diagnosis result, I_s value of the first story in the Y-direction is 0.66. This value is relatively large which means a fairly excellent seismic performance. But if the experienced earthquake motion to this building is stronger than the one considered in the seismic diagnosis, the result of seismic diagnosis and the actual damage condition of the building will not agree with each other. For this reason, seismic performance is judged by the members capacity and deflection ability, but the actual damage is presumed inflicted by the joint panel failure between first story column and second floor beams.

Investigation of Seismic Performance Including the Joint Capacity. The result obtained by the secondary level seismic diagnosis method discussed latter section was modified by considering the beam-column joint capacity as proposed in Ref.[1]. Joint panel shear capacity is simply estimated by Eqs.3 and 4. On the other hand, joint panel shear force Q j and column shear force Q c relationship can be expressed in Eq.5.

$$V_{ju} = 0.135\, F_c (1-p) b_c \cdot D_c \quad \text{for } + \text{ type} \tag{3}$$

$$V_{ju} = 0.09\, F_c (1+p) b_c \cdot D_c \quad \text{for } \top, \vdash \text{ type} \tag{4}$$

$$Q_j = Q_c [0.5(1+z)(1-r) - z \cdot q] / q \tag{5}$$

Substituting V ju obtained from Eq. 3 or Eq. 4 in the Q j of Eq. 5, the column capacity Q cju at joint panel shear failure can be obtained. When Q cju is less than column capacity Q c joint panel will fail in shear prior to the members bending or shear failure. Moreover, the ductility index of column capacity with joint panel shear failure should be F = 1.0.

The results of seismic diagnosis considering the beam-column joint capacity in which the fundamental index E oj and structural seismic index E sj considering the joint panel capacity is shown in Fig.3 and Fig 4. Index E oj is 0.57 in the first storyin the Y-direction which is less than 0.6. Index I sj are mostly less than I s index, resultantly, joint panel capacity is lower than members capacity, especially at the firstfloor in both directions. Index I sj is 84% of I s in the X-direction and 82% in Y-direction,consequently, damage degree at first floor in the Y-direction is estimated to have a satisfying seismic performance with I s =0.66, is modified to I sj =0.57 which means "poor seismic performance".

As discussed above, damage of the building due to this earthquake coincides relatively with the result of seismic performance estimated by I sj index. In case of this building, due to the poor beam-column joint capacity building is presumed to initiate collapse.

Table 4. ( ): Calculated values at the seismic force from right to left direction (R-L)

Y-Direction 1st Floor Seismic load:L-R direction										
Joint	Mu (t·m)	bg×Dg (cm)	bc×Dc (cm)	Vj(t)	κ	φ	bj(cm)	Dj(cm)	Vju(t)	Vju/Vj
2JA3	129.0 (161.4)	55×80	80×80	156.5 (195.8)	2	0.9	67.5	58.4	170.3	1.088 (0.871)
2JA4	161.4 (129.0)	55×80	80×80	195.8 (156.5)	2	0.9	67.5	58.4	170.3	0.870 (1.088)
2JB2	135.2 (184.4)	55×80	80×80	164.0 (223.7)	2	0.9	67.5	58.4	170.3	1.038 (0.761)
2JB3	224.6 (191.1)	55×80	80×80	272.5 (231.8)	3	1.1	67.5	58.4	312.2	1.146 (1.347)
2JC1	134.9 (184.6)	55×80	80×80	163.7 (224.0)	2	0.9	67.5	58.4	170.3	1.040 (0.760)
2JC2	184.6 (134.9)	55×80	80×80	224.0 (163.7)	2	0.9	67.5	58.4	170.3	0.760 (1.040)
2JE2	303.5 (227.7)	55×80	80×80	368.2 (276.2)	3	1.1	67.5	58.4	312.2	0.848 (1.130)
2JE3	203.3 (252.0)	55×80	80×80	246.6 (305.7)	2	0.9	67.5	58.4	170.3	0.691 (0.557)

Investigation by Exact Calculation. The exact values of shear stress V j and shear capacity V ju of joint panel which is calculated from column capacity V c at the beam flexural yielding moment in the Y-direction at second floor are listed in Table 4. The L-R means seismic force will act from left to right direction. The number of joint panels with a possibility of shear failure (i.e., V ju/ V j < 1.0) were five out of total 12 joints inspected (See Fig.2).There were two joint panels which both have the possibility of shear failure especially at Frame A. This kind of damage was observed in the site. From the calculation results, this building was severely damaged at the first floor in Y-direction because of shear failure at joint panel.

CONCLUSION

Based on the investigation and calculation result concerning the RC buildings in which thebeam-column joint panel failure was observed during the 1995 Great Hanshin earthquake, thefollowing conclusions were obtained.

1) Buildings with joint panel damage can be cited from the following items
a) Some buildings designed in even accordance with the current seismic provisions
b) Buildings with moment resisting frame structure of medium- and high-rise story buildings (6 to 14 story high).
c) Buildings with one span or partially two spans in short direction and two or six spans in long direction.
d) Buildings with heavy unit weight per floor area.
e) Distinguished collapsed were observed in the short direction of single span buildings

2) In the investigated buildings, a great proportion of private housings designed in accordance with current seismic provisions have joint panel deterioration which six out of 26 (about 1/4).

3) The damage observed was more extensive than that predicted by structural performance Is value, but much closer than that predicted by structural performance index at joint failure I sj value.

4) Based on the observation of the damage conditions of the joint panel, secondary structural walls such as waist wall or side wall have insignificant effect to prevent the joint panel failure.

5) Base shear coefficient at the joint panel shear failure of the building (with first soft story) with joint panel damage is generally smaller than 0.5 in the 80% of all the buildings investigated and this values are slightly higher than lower limit of the presumed values of the F e ·D s =0.45.

6) The result of seismic performance calculated by the simplified method considering the beam-column joint capacity of a damaged eight-story RC housing structure is somewhat consistent with the actual damage condition of the building. Accordingly, these buildings suffered very severe damage in its entity mainly because of the beam-column joint panel failure.

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