Seismic Capacity of Reinforced Concrete Apartment Buildings Damaged due to 1992 Erzincan Earthquake, Turkey

by

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ABSTRACT

This paper describes the seismic capacity of reinforced concrete apartment buildings which were damaged due to 1992 Erzincan Earthquake, Turkey. The seismic capacity was evaluated according to the Japanese Standard and the correlation between the seismic capacity and damage levels observed in the field survey was discussed.

Introduction

On March 13, 1992, 19:19 local time, an earthquake of magnitude Ms 6.9 (USGS) struck the city of Erzincan, located in the eastern part of Turkey. The epicenter was near the city and collapse and heavy damage of reinforced concrete buildings were reported with a large number of deaths.

The Architectural Institute of Japan (AIJ) and the Japan Society of Civil Engineers (JSCE) dispatched a joint reconnaissance team and investigated the affected area under the cooperation of Bogazici University, Istanbul, Turkey. The author participated in the investigation as a member of the team.

As described in Ref. [1] in detail, many reinforced concrete buildings with three to four stories were severely damaged. In this report, the seismic capacity of reinforced concrete apartment buildings which was located in the severely affected area of the city was evaluated, and the correlation between the seismic capacity and the damage levels was investigated.

Description of Damaged Apartment Buildings

The structure investigated herein is three-story reinforced concrete apartment building, located in the central area of Yavuz Selim District in Erzincan City (***Fig. 1*** and ***Photo 1***). In this area, approximately 40 buildings were under construction at the time of the quake.

Two structural types were found in the area; the first type has two bays in the transverse direction whereas the second has three bays. Each building consisted of two identical but

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independent structural units adjacent to each other. ***Fig. 2*** shows the details of the one structural unit of two-bay type apartment building.

The building was partially infilled with concrete blocks in the basement and hollow bricks (***Photo 2***) in upper stories which were both unreinforced. The story height was approximately 280 cm in each story and the cross section of a typical columns was 40 cm x 25 cm or 60 cm x 25 cm. As commonly observed in the city, each building in this area also had a basement at about 2 m below the ground level. During the earthquake, however, the basements of most buildings were still exposed since the excavation had not been refilled as shown in ***Photo 3***.

Major damage to the buildings was generally observed in the columns of basement, and some of them totally collapsed as shown in ***Photo 4***. ***Table 1*** shows the damage statistics according to the damage classification used in Japan shown in ***Table 2***. Extensive cracks in the columns of basement, poor confinement resulting from inadequate detailing of reinforcement (***Photo 5***), buckling of longitudinal reinforcement due to the absence of lateral reinforcement in the beam–column joints (***Photo 6***), and crushing of concrete in the columns (***Photo 7***) were the commonly observed damage types. In some buildings, the soft story mechanism (flexural failure) was found in the basement columns as shown in ***Photo 8***.

In general, the quality of concrete was poor and honeycombs were observed in many buildings as shown in ***Photo 9***. It is interesting to note that the damage levels of the buildings ranged from "*slight damage*" to "*total collapse*" although the buildings in the area had identical structural plans. To investigate the correlation between the strength of concrete and the damage level, Schmidt hammer tests (non–destructive) on several buildings in the area were carried out. The test results are summarized in ***Table 3***. It is clearly seen from the table that the concrete strength was generally low irrespective of the damage level and no obvious correlation between the damage level and concrete strength was found. However, upon close examination, it was revealed that buildings which suffered slight damage already had concrete block infilled walls in the basement and these walls seemed to have contributed to the strength and stiffness of the building. Furthermore, buildings with basement around which the soil had been already refilled before the quake suffered almost no damage. ***Fig. 3*** illustrates the arrangement of non–structural walls, basement condition and corresponding damage levels.

To understand the differences in damage levels, the seismic capacity of the building was evaluated and the correlation between the damage level and the structural condition, i.e., the arrangement of non–structural walls and the basement condition, was studied in the subsequent section.

Seismic Evaluation of Damaged Building

As stated earlier, the building had two structural types in the transverse direction, i.e., two–bay type and three–bay type, but the former was evaluated herein since most of the surveyed buildings were categorized in the two-bay type building. In the seismic evaluation of the building, *the Japanese Standard for Evaluation of Seismic Capacity of Existing Reinforced Concrete Buildings*[2] was applied.

Basic Concept of the Standard

The *Standard* evaluates the seismic capacity at each story and in each direction of the building by the following index;

$$I_S = E_O \cdot S_D \cdot T \qquad (1)$$

where,

E_O = basic structural index calculated by ultimate horizontal strength, ductility, number of stories and story level concerned

S_D = structural design index to modify the Eo–index due to the grade of the irregularity of the building shape and distribution of stiffness along the height

T = time index to modify the Eo–index due to the deterioration of strength and ductility

The standard values of the S_D– and T–indices are 1.0. The Eo–index for the single structural system can be expressed by the product of the ultimate horizontal strength index in terms of story shear coefficient (C), ductility index (F) and story index ϕ. Story index (ϕ) at the first floor level is 1.0. Therefore, the Eo–index at the first floor level of the simple structure can be defined as;

$$E_O = C \cdot F \qquad (2)$$

In evaluating F–index in Eq.(2), the shear–span–to–depth ratio, flexural strength, shear strength etc. are considered. Basically, F = 1.0 for brittle (shear failure type) members and F = 1.27 to 3.2 for ductile (flexural failure type) members in the *Standard*.

Assumptions in Seismic Evaluation

To evaluate the seismic capacity of the apartment building, the following assumptions were employed.

1) Dead loads were calculated based on the structural dimensions measured at the site, including the non–structural brick walls.
2) Live loads were neglected and T–index in ***Eq. (1)*** was assumed 1.0 since the buildings were under construction at the time of the earthquake.
3) The strength of concrete was assumed 100 kgf/cm^2 based on the site tests. (***Table 3***)
4) The strengths of reinforcing bars were assumed 3000 kgf/cm^2 for those with a diameter less than 9 mm and 2800 kgf/cm^2 for others, based on the tensile test results carried out in Japan

using some sample rebars obtained at the site.

5) Evaluation was carried out using the computational program[3] coded according to the *Standard.*

Table 4 shows the total and unit weight of each floor. The unit weight is approximately 0.6 tonf/m^2 and axial forces are 10 to 27 tonf (19 to 27 kgf/cm^2).

Preliminary analyses based on the assumptions above showed that the F–index ranged from 2.0 to 3.2, which means that the failure mode of columns was ductile flexural type. In evaluating the ductility index of columns, the *Standard* assumes that members are properly reinforced and confined using such as lateral reinforcement with 135–degree hooks at both ends. As stated earlier, however, the lateral resistance was insufficient due to poor bar arrangement such as 90–degree hooks and the sufficient ductility was not provided in damaged buildings, showing brittle shear failure. Considering the features above, F–index was assumed 1.0 in all columns.

Evaluation Results

To investigate the effects by non–structural walls and refilling around the basement on the seismic capacity, four cases were investigated as shown in ***Table 5***. Note that cases (a) to (c) correspond to sketches shown in ***Fig. 3*** and that case (d) was studied to investigate the seismic capacity of structure without non–structural walls. In cases with non–structural walls, i.e., cases (a) to (c), their contribution to lateral resistance was allowed for assuming that the ultimate shear capacity was 5 kgf/cm^2 for concrete blocks[4] and 1 kgf/cm^2 for hollow bricks.

Fig. 4. shows the results in each story and each direction, i.e., the longitudinal and transverse direction. From the figure, the following findings can be obtained.

(1) The seismic capacity of building without non–structural walls is significantly lower than that with non–structural walls and the contribution of walls can not be neglected to understand the discrepancy of damage levels of buildings under construction. [case (b) vs. case (d)]

(2) When non–structural walls are provided in all stories but the basement is not refilled and hence the building is four–story [case (b)], Is–index is larger than 0.5 in any stories and directions except for the longitudinal direction of the first story level. When the building has non–structural walls only in the upper stories as was observed in the heavily damaged or collapsed buildings [case (c)], Is–index in the basement is significantly lower than that in the upper stories.

(3) When the basement is refilled and hence the building is three–story [case (a)], Is–index is larger than 0.6 even in the longitudinal direction of the first story level, where the value was the smallest in the building. This case results in better seismic capacity than any other assumptions in any story levels.

(4) These results mentioned above correspond to the observed facts that (i) the damage level

was severest in buildings which had no non–structural walls in the exposed basement, but (ii) almost no damage was found in buildings which had the refilled basement.

Conclusions

The seismic capacity of reinforced concrete apartment buildings which suffered 1992 Erzincan Earthquake was evaluated and the correlation between the seismic capacity and the damage level was discussed. The results can be summarized as follows.

(1) Schmidt hammer test results of buildings with different damage levels show that the strength of concrete was generally low irrespective of the damage level, and that no obvious correlation between the damage level and the concrete strength was found.

(2) The presence of the non–structural walls significantly contributed to the seismic capacity of the building. Especially when the building had non–structural walls only in the upper stories, as was observed in the severely damaged buildings, the evaluated seismic capacity in the basement was significantly lower than that in the upper stories.

(3) The building with refilled basement showed the highest seismic capacity.

(4) The seismic evaluation results correspond to the surveyed damage levels, and it can be concluded that the difference of damage levels in the surveyed buildings under construction is mainly attributed to the arrangement of non–structural walls and the basement condition.

References

[1] Joint Reconnaissance Team of Architectural Institute of Japan, Japan Society of Civil Engineers, and Bogazici University, Istanbul, Turkey, "Damage Report on 1992 Erzincan Earthquake, Turkey," 1993

[2] Japan Building Disaster Prevention Association, "Standard for Evaluation of Seismic Capacity of Existing Reinforced Concrete Buildings," 1977, revised in 1990. (in Japanese)

[3] SPRC Committee, "Computational Program for Seismic Evaluation of Existing Reinforced Concrete Buildings (Screen Edition–2)," Japan Building Disaster Prevention Association, 1980

[4] Okada, T. et al., "Seismic Capacity of Reinforced Concrete Buildings Which Suffered 1985 Mexico Earthquake in Mexico City, Part 1 – Part 13," Proceedings of the Annual Convention of Architectural Institute of Japan, 1986 August (in Japanese).

Table 1: Damage Statistics in central Yavuz Selim District [1]

	Damage Levels				
	slight	light	moderate	heavy	collapse
Number of Buildings	17	8	3	2	11
(%)	(41.5)	(19.5)	(7.3)	(4.9)	(26.8)

Table 2: Damage classification of reinforced concrete buildings used in the survey [1]

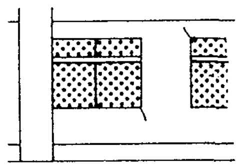
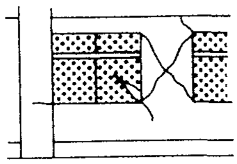
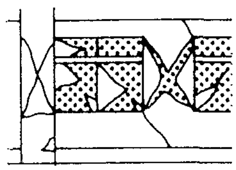
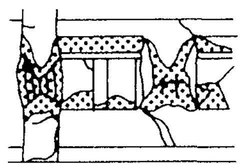
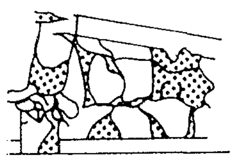
damage level	damage in members	Illustration of damage
no damage	No damage is found.	------------
slight damage	Columns, shear walls or non-structural walls are slightly damaged.	
light damage	Columns or shear walls are slightly damaged. Some shear cracks in non-structural walls are found.	
moderate damage	Typical shear and flexural cracks in columns, shear cracks in shear walls, or severe damage in non-structural walls are found.	
heavy damage	Spalling of concrete, buckling of reinforcement, and crushing or shear failure in columns are found. Lateral resistance of shear walls is reduced due to heavy shear cracks.	
partial collapse	The building is partially collapsed due to severely damaged columns and/or shear walls.	
total collapse	The building is totally collapsed due to severely damaged columns and/or shear walls.	

Table 3: Concrete Strength in the three-story apartment buildings in central Yavuz Selim District (unit: kgf/cm^2) [1]

Story level	Building Numbers and Damage Levels*			
	No. 1 / heavy**	No. 2 / heavy**	No. 3 / moderate***	No. 4 / slight**
3	80	95	80	80
2	80	95	less than 80	80
1	100	120	80	80
BF	150	185	100	120
average	103	124	85	90

* four buildings in central Yavuz Selim District were investigated
** buildings with two bays in the transverse direction
*** buildings with three bays in the transverse direction

Table 4: Weight of the Structure

Story Level	Floor Area (m^2)	Weight (tonf)	Σ Weight (tonf)	Weight in Unit Area ($tonf/m^2$)
3 F	215	119	119	0.55
2 F	215	133	252	0.62
1 F	215	133	385	0.62
B F	215	133	518	0.62

Table 5: Analysis Cases

case	Infilling of non-structural walls		Refilling around basement	Number of Stories
	1 F to 3 F*	B F**		
(a)	yes	yes	yes	3
(b)	yes	yes	no	4
(c)	yes	no	no	4
(d)	no	no	no	4

*: hollow bricks **: concrete blocks

Photo 1: Three-story apartment building under construction in central Yavuz Selim District

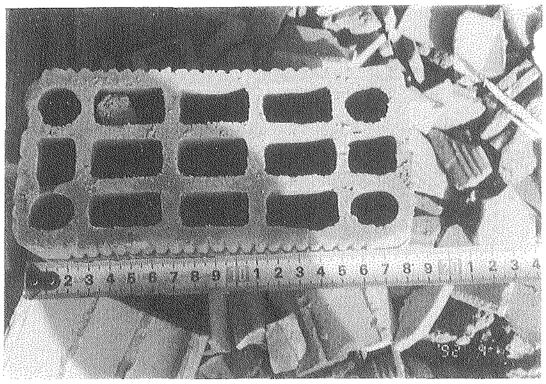


Photo 2: Brick wall

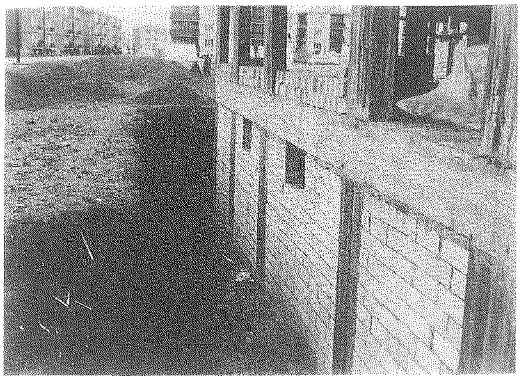


Photo 3: Basement and surrounding soil

Photo 4: Collapsed building

Photo 5: Sheer reinforcement with 90-degree hooks

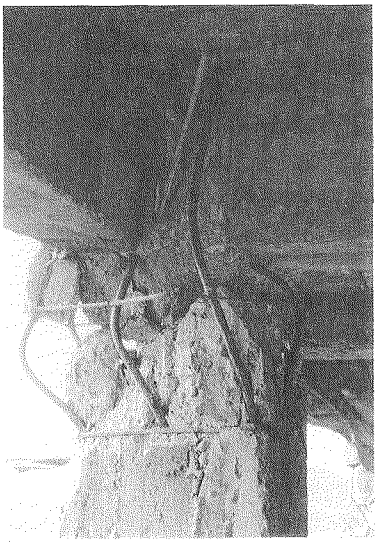


Photo 6: Failure in beam–column joint (no lateral reinforcement)

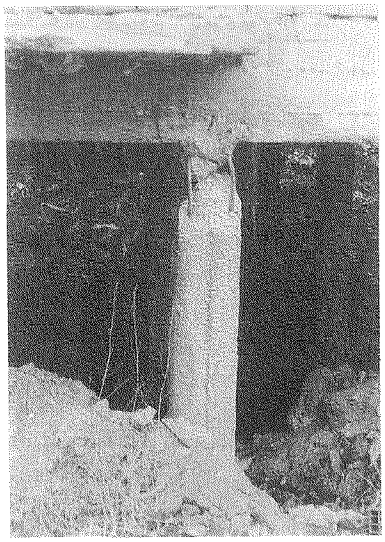


Photo 7: Shear failure at the top of column

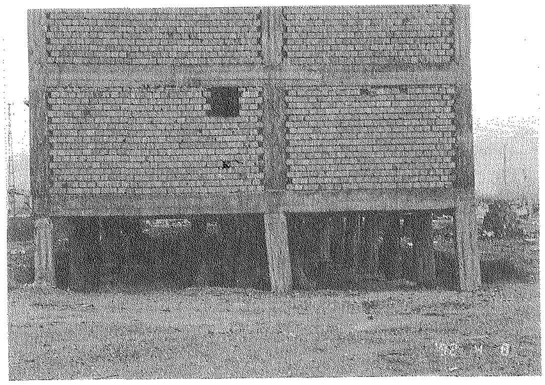


Photo 8: Side–sway caused by flexural failure of the columns at basement

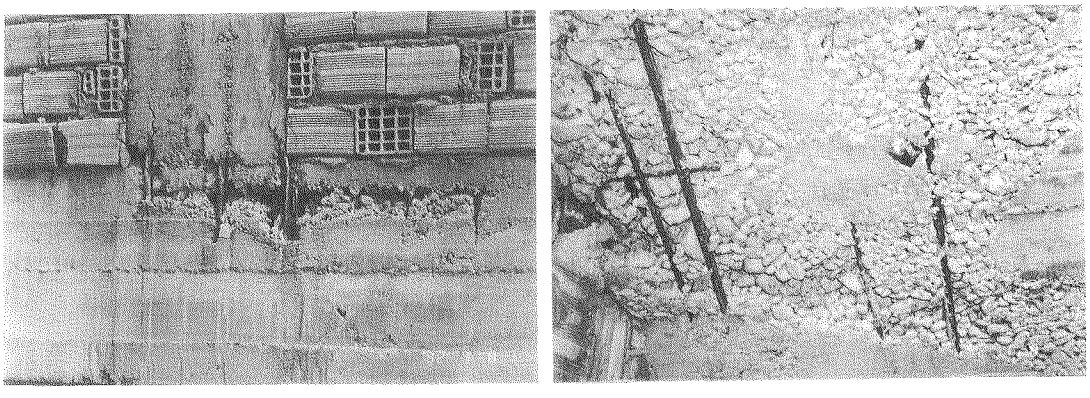
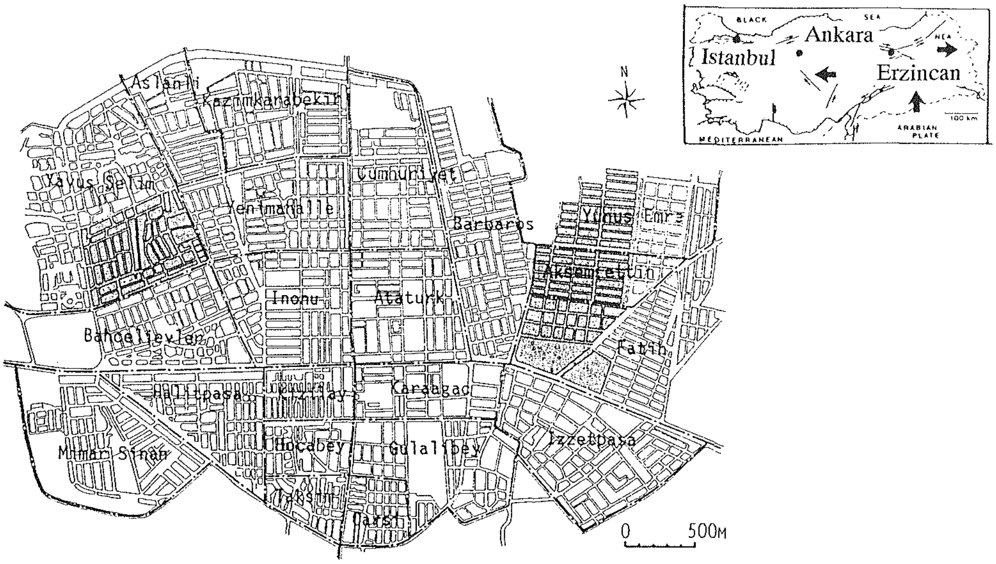


Photo 9: Honeycombs in concrete



Dotted zone : Area carried out damage survey of whole buildins

Fig. 1: Map of Erzincan City [1]

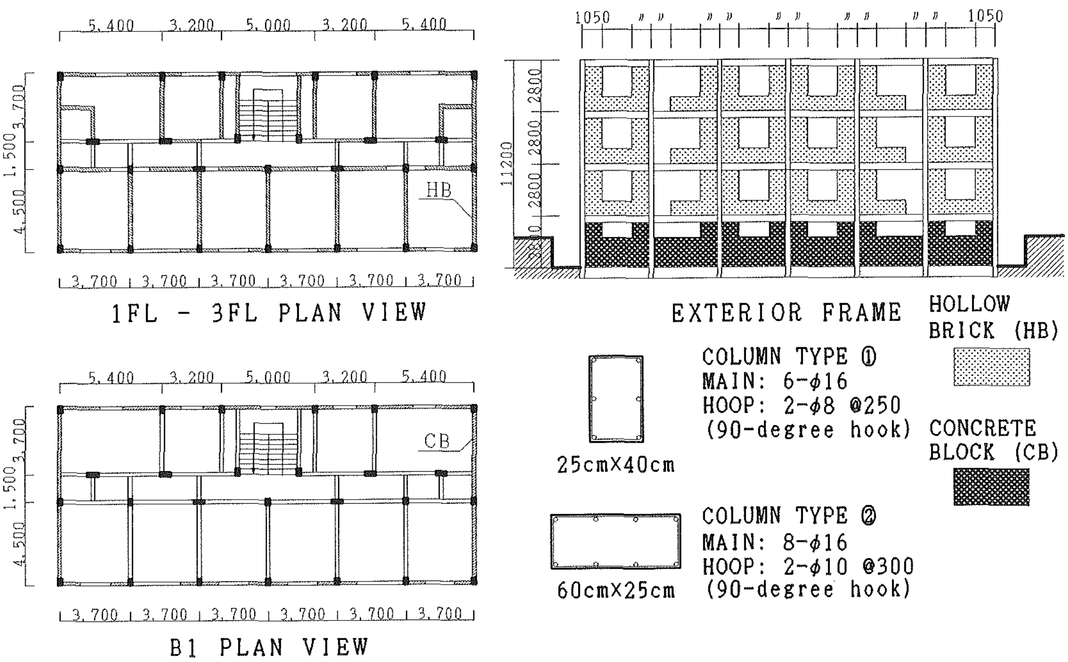


Fig. 2: Details of two-bay type apartment building

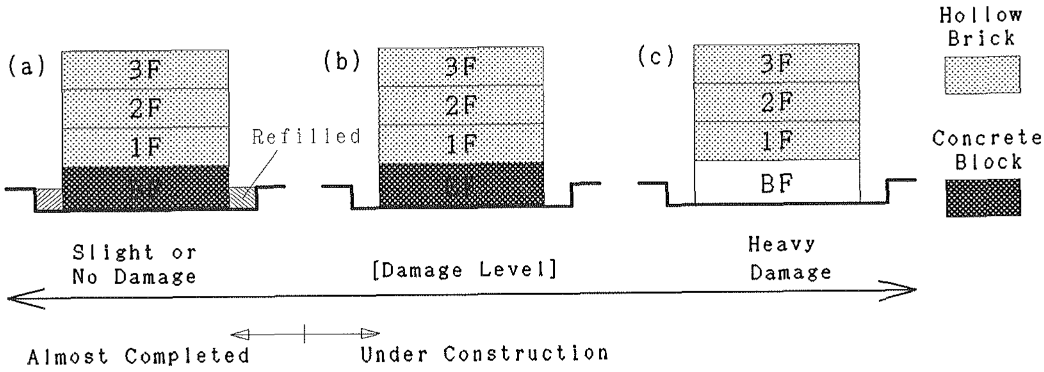


Fig. 3: Arrangement of non-structural walls, basement condition and corresponding damage levels

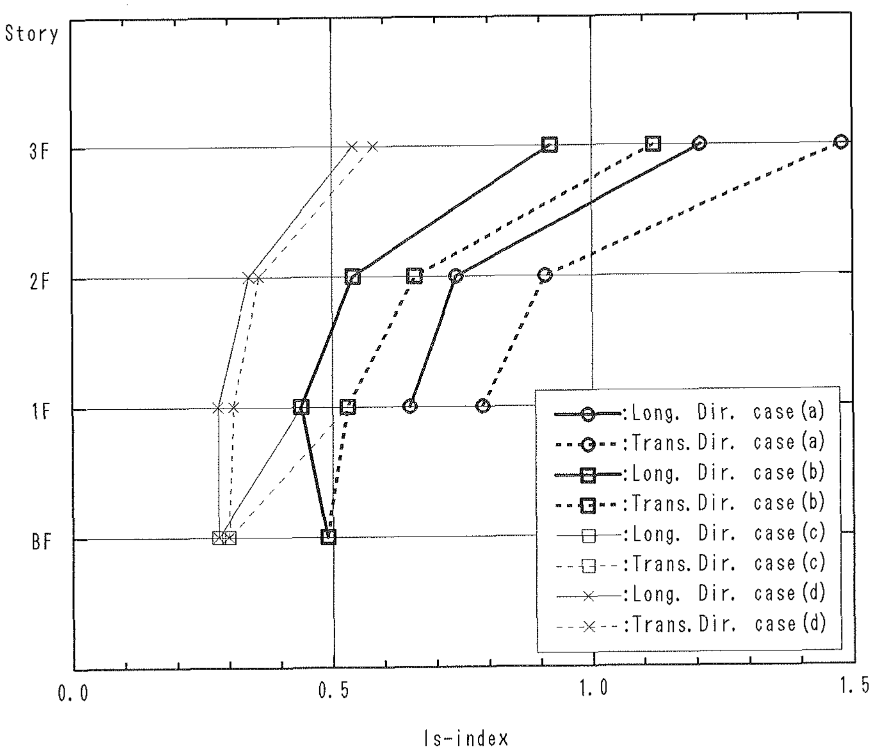


Fig. 4: Results of Seismic Evaluation