EARTHQUAKE RESPONSE ANALYSES OF PRECAST REINFORCED CONCRETE BUILDINGS DAMAGED DUE TO ARMENIA SPITAK EARTHQUAKE

by

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INTRODUCTION

An earthquake with magnitude of 7 occurred near Spitak city located in the northern part of the Republic of Armenia, U.S.S.R. on December 7, 1988, and caused huge damages. About 30,000 lives were lost and 500,000 people were involved in this disaster. It was reported that nearly 270 buildings, of reinforced masonry structures and reinforced precast concrete (as referred to RPC hereafter) frame structures, collapsed [Ref. 1]. In order to investigate the reasons of the collapse of 9 storied standardized RPC structure, which sustained severe damages, non-linear earthquake response analyses were carried out.

OUTLINE OF RPC STRUCTURES AND CORRESPONDING SEISMIC FORCES

Fig. 1 shows a plan-view and elevation-views of a typical apartment building constructed with RPC members. Horizontal elements of this 9 storied building consist of girders in the longitudinal direction (L-direction) and of one-way void slab element in the transverse direction (T-direction), and structural walls are located in T-direction. Section properties of a girder and a column are shown in Fig. 2. Non-structural panels are attached loosely on the exterior frames and many non-structural panel walls are located inside of the structure.

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The seismic intensity is classified by seismic codes in the Soviet Union, and high seismic zones like Armenia are specified as 7, 8 and 9 on MSK scale. Those intensities 7, 8 and 9 correspond to 100gal, 200gal and 400gal of the peak ground acceleration, respectively. The zone with intensity 9 is not assigned in Armenia. Maximum ground accelerations due to the Spitak earthquake were estimated as shown in Table 1 [Ref. 2]. It shows that the ground motion in most cities exceeds the seismic intensity specified in the codes.

EARTHQUAKE RESPONSE ANALYSES

Analytical Models

These structures were designed according to Armenian S.S.R. codes, based on the elastic design concept. In order to estimate the seismic capacity of the structures, non-linear analyses were carried out.

Fig. 3 illustrates the schematic model of this structure. Calculated natural periods of the 9 storied structure based on the assumption of rigid slab system corresponds well with the observed ones [Ref. 3] as shown in Table 2. The mass-spring shear system was used herein.

A seismic preparedness of buildings to a future major earthquake is an important issue for the damage mitigation. The reduction of number of stories may be a simple way to reduce seismic forces to buildings, when the existing precast concrete members are available. The effects on earthquake responses to buildings of which stories are reduced to 2 and 5 are also investigated. Table 3 shows natural periods of those buildings.

Each story was supposed to be constructed with 2 or 3 types of members; column, structural wall and non-structural wall. Degrading Tri-linear Model and a new model [Ref. 4], which was named as Melkumian Model, were used to represent column and wall characteristics, respectively as a hysteresis rule. Fig. 4 shows the outline of Melkumian Model, where slip behavior is taken into consideration in the model based on experimental results of shear walls used in Armenia.

Initial stiffness and stiffness changing points for each member are shown in Table 4, and the values were calculated based on experimental results of frames and structural walls conducted in Armenia [Ref. 5]. And it was assumed that non-structural walls will lose its carrying load capacity, when it reached the yielding point.

Non-linear Earthquake Response Analyses

Figs. 5 and 6 show earthquake accelerograms which were recorded at Gukasian located about 35km from the epicenter (digitized at Okada lab. I.I.S, Univ. of Tokyo), and at Yerevan located about 80km from the epicenter (digitized at ArmNIISA in Yerevan, Armenia [Ref. 6]). Fig. 7 shows the response acceleration spectra. In the earthquake response analyses, these digitized accelerograms were used with the peak acceleration scaled to 100gal to 800gal. Newmark's β-method ($\beta=1/6$) was used in numerical integration and 4% to the critical damping was taken as the damping factor.

In the results of these analyses, maximum inter-story displacement at the first story is the largest in most cases.

Fig. 8 shows the calculated maximum response displacements to the accelerogram recorded in Yerevan. Non-structural elements do not give significant effects on the response in T-direction, while significant in L-direction, especially when the peak acceleration is 200gal to 500gal. Fig. 9 illustrates distributions of maximum displacements and restoring forces to the accelerogram with the peak acceleration of 400gal. In the model with non-structural walls, collapse of non-structural elements occurred at the first to the third stories, and deformations were concentrated in those stories as shown in Fig. 9 b). This shows that the influence of non-structural elements on the seismic behavior of the structure is not negligible in L-direction.

Figs. 10 and 11 show ductility factors of the structural wall in T-direction and of the column in L-direction at the first story to Gukasian accelerogram. When the peak acceleration is smaller than 400gal, the ductility factor is lower than 2 in both directions. When the peak acceleration is 600gal which is equal to the maximum of the estimated peak accelerations in Table 1, the ductility factor of the structural wall (T-direction) is around 3. In the experimental results of the wall, it showed good ductile behavior; ductility factor was around 3.75 as shown in Table 4. It means that if the member of real buildings had exhibited such a good ductile behavior, those buildings would not have collapsed. Many buildings of this structural type, however, collapsed actually, and hence it is important to investigate the behavior after yielding of these members provided in the actual buildings.

Finally the responses of lower storied buildings, the one is 2 storied and the other is 5 storied, are also investigated. Figs. 10 and 11 show the maximum ductility factors of lower storied buildings together with those of the 9 storied building. It is found that 2 storied building does not reach yielding point even to 400gal input motion corresponding to maximum design force level. It should be noted however that the reduction of number of stories is not always effective to reduce response values, especially in L-direction.

CONCLUDING REMARKS

(1) The estimated peak ground accelerations due to Spitak earthquake are two times or even four times larger than expected in seismic design codes in the Republic of Armenia. This is the main reason which caused collapse of constructions.

(2) Ductility factor of at least 2.0 is required to resist the seismic forces caused by Spitak earthquake. It is important to investigate the behavior after yielding of members including structural and non structural walls and columns provided in the actual buildings.

(3) Non-structural elements may cause concentration of damages in lower stories, and it is concluded that the effects of these elements are not negligible in analyses.

(4) The 2 storied building with structural walls does not reach yielding points even to Gukasian 400gal earthquake. It should be noted, however, that simple reduction of number of stories is not always effective to reduce maximum response displacements.

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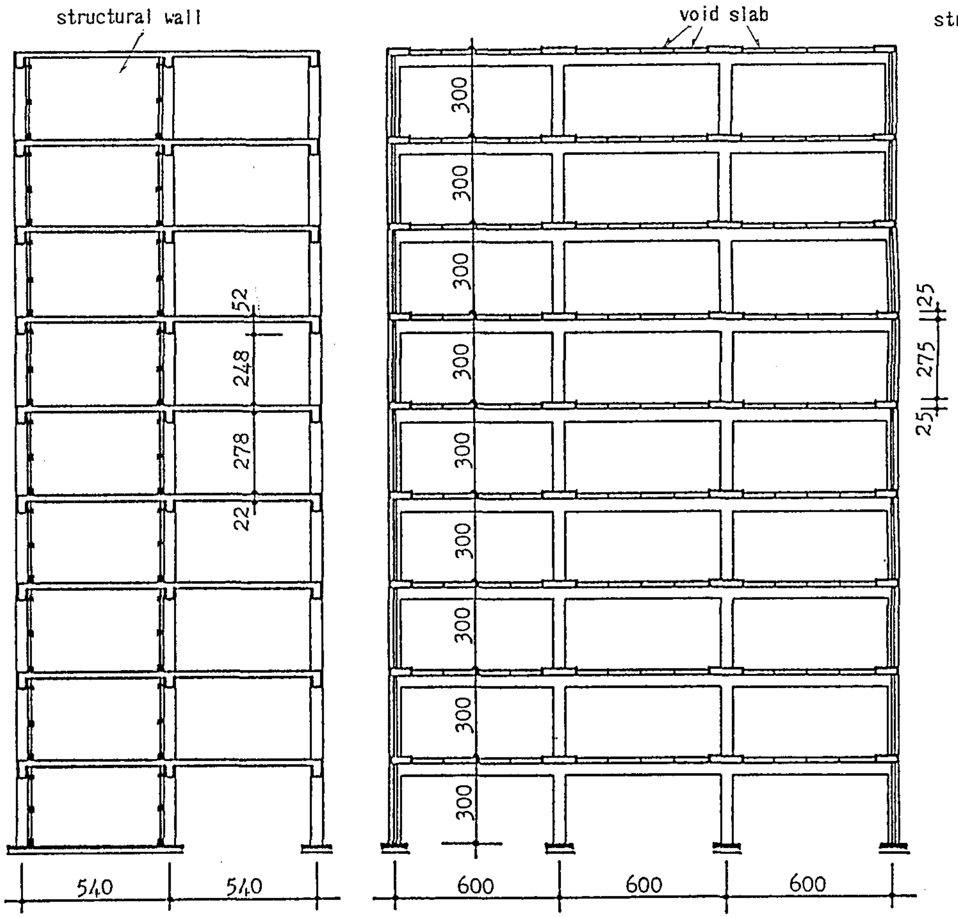
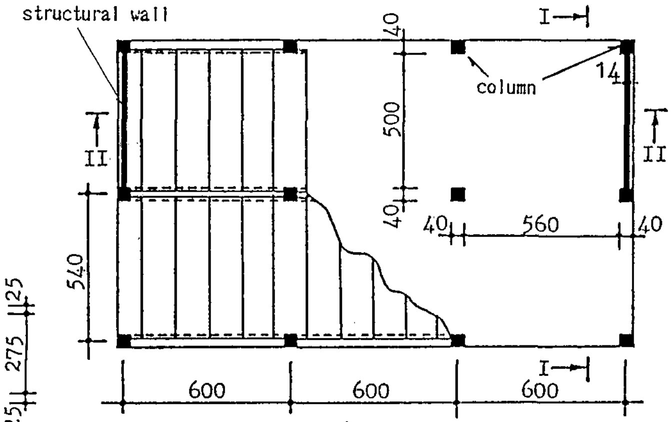


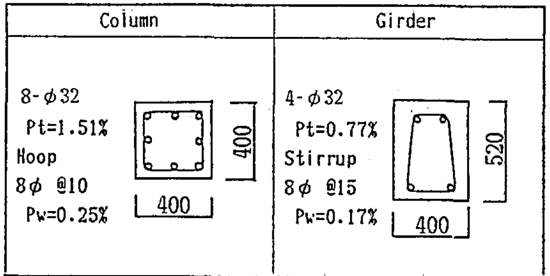
a) T-direction

b) L-direction



c) Plan

Fig. 1 RPC Frame Structure



Concrete strength	Fc = 300	[kg/cm²]
Young's modulus of concrete	$E = 1.9 \cdot 10^5$	[kg/cm²]
Strength of main bar	σy = 3400	[kg/cm²]
Strength of hoop and stirrup	σy = 2100	[kg/cm²]

Fig. 2 Section Properties of Members

Table 1 Estimated Maximum Ground Accelerations

Location (Epicentral distansce)	Seismic Design Intensity	Estimated Max. Acc.
SPITAK (7km)	7(100gal)	600gal~
LENINAKAN (35km)	8(200gal)	530gal~
KIROVAKAN (20km)	7(100gal)	360gal~
GUKASIAN (35km)	8(200gal)	228gal~
YEREVAN (80km)	8(200gal)	65gal~

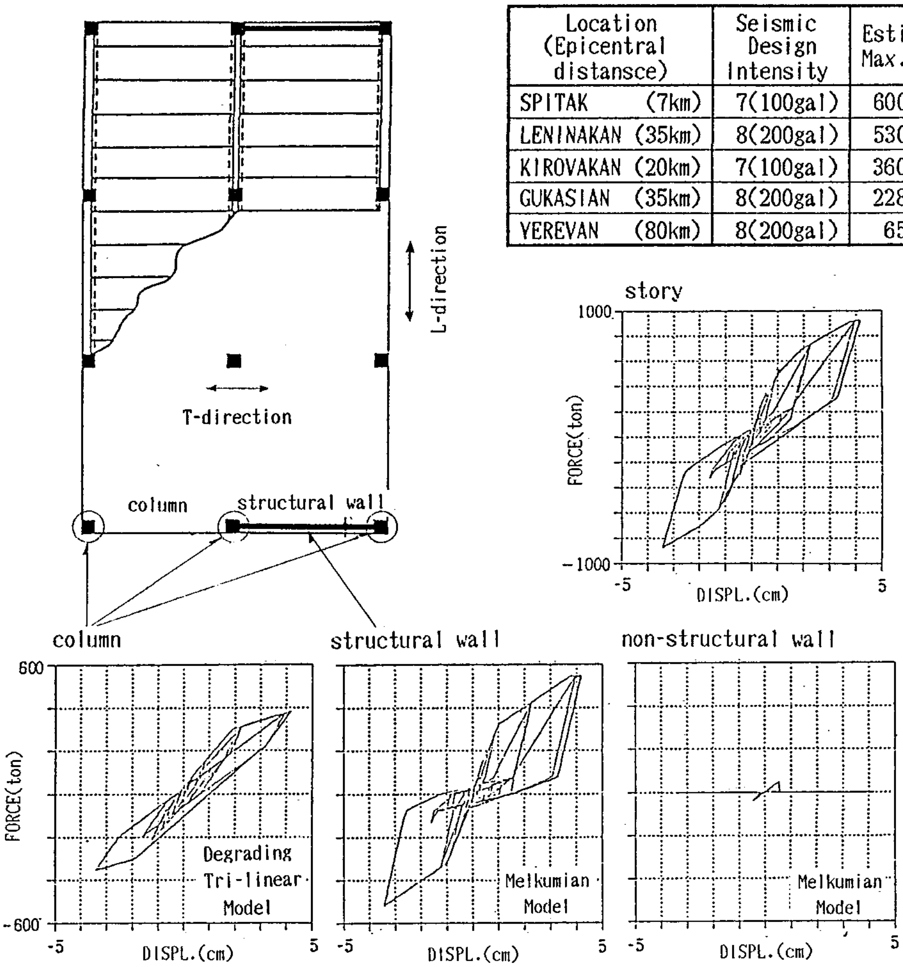


Fig. 3 Schematic Model

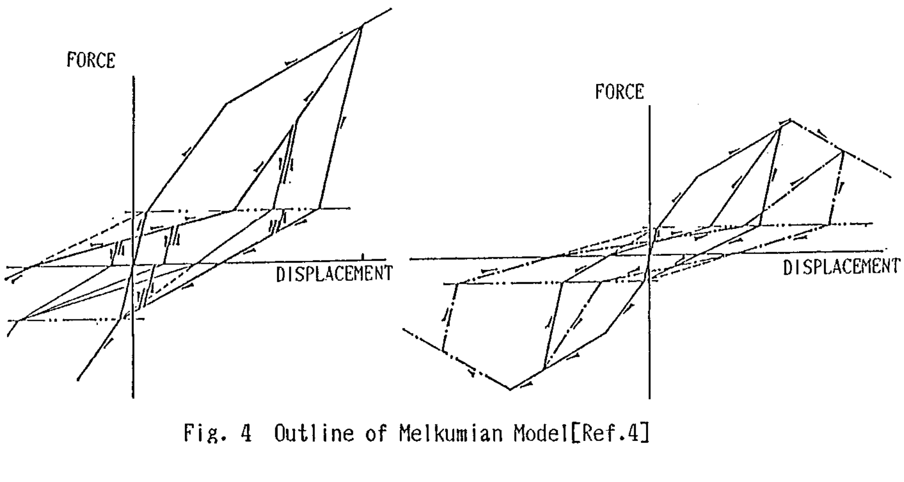


Fig. 4 Outline of Melkumian Model[Ref.4]

Table 4 Initial Stiffness and Stiffness Changing Points

Dir.	Member	I. stiffness (ton/cm)		Displacement (cm)			Force (ton)		
				crack	yield	ultimate	crack	yield	ultimate
T	Column	276	956	0.24	1.88	7.50	66.24	298.08	503.76
	S.Wall	534		0.15	1.00	3.75	80.01	320.40	544.68
	N.S.Wall	146		0.09	0.55	----	13.14	48.62	----
L	Column	404	673	0.20	1.50	6.00	80.80	339.36	573.52
	N.S.Wall	269		0.09	0.55	----	24.21	89.58	----

S.Wall indicates Structural Wall and N.S.Wall Non-Structural Wall.

Table 2 Natural Periods of 9 Storied Building

Dir.	Natural Periods (sec)	
T	Calculated	0.60
	Observed	0.46~0.53
L	Calculated	0.71
	Observed	0.61~0.63

Table 3 Natural Periods of 2 and 5 Storied Buildings

St.	Natural Periods (sec)	
5	T-dir.	0.35
	L-dir.	0.42
2	T-dir.	0.17
	L-dir.	0.20

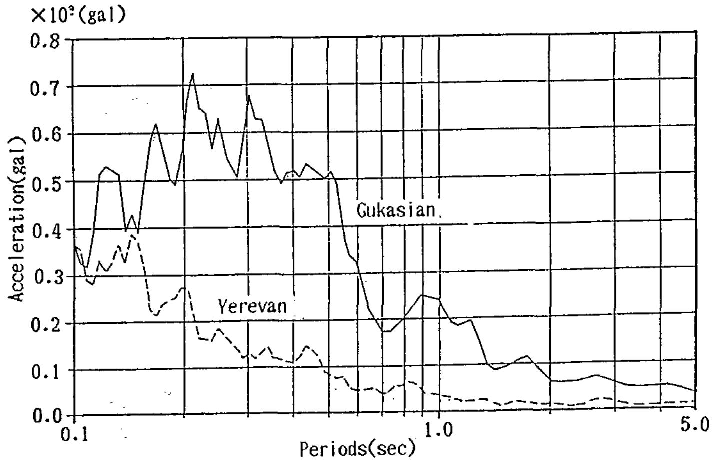


Fig. 7 Response Acceleration Spectra

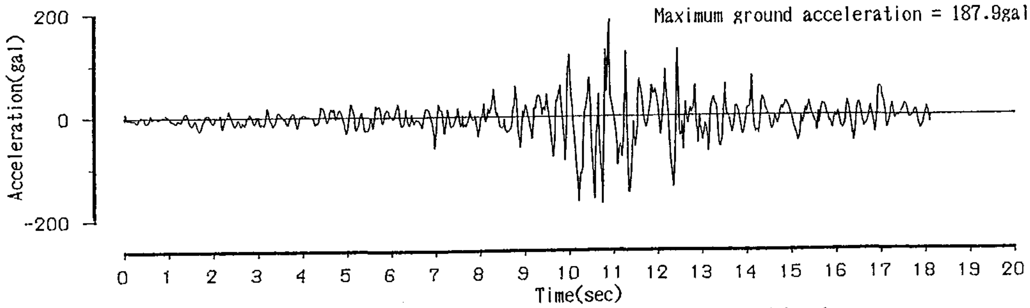


Fig. 5 Earthquake Accelerogram Recorded at Gukasian

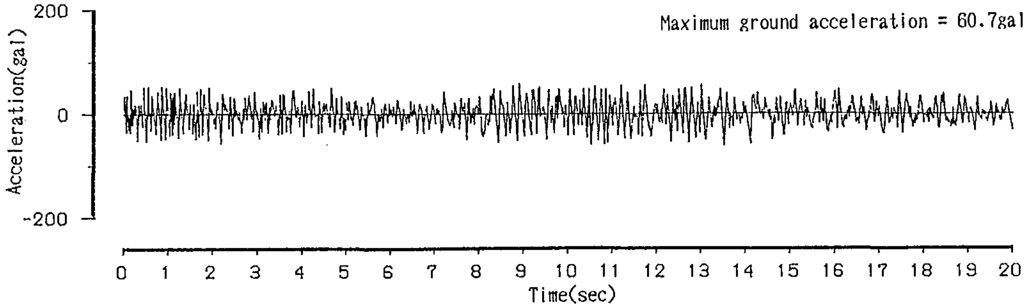


Fig. 6 Earthquake Accelerogram Recorded at Yerevan

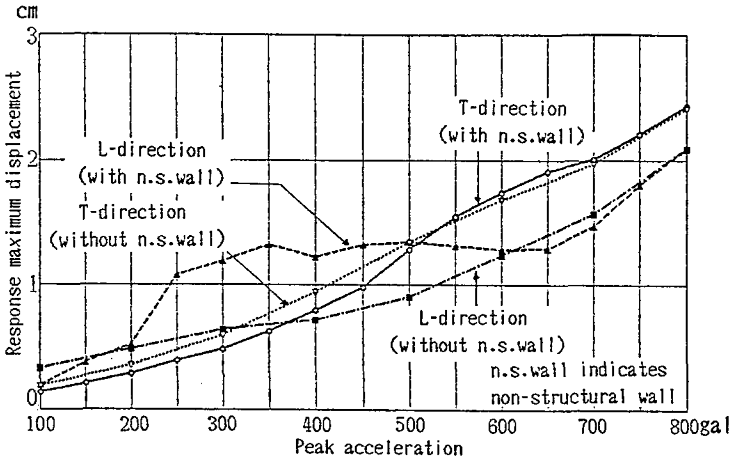


Fig. 8 Maximum Response Displacement at First Story
(9 Storied, to Yerevan Accelerogram)

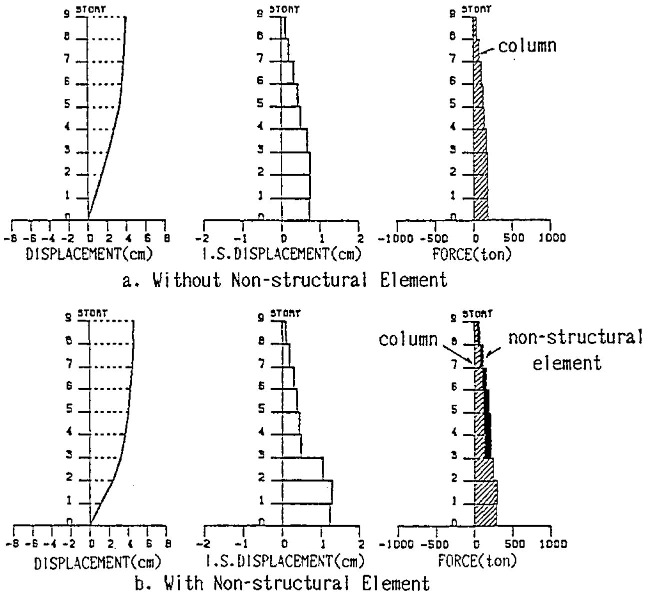


Fig. 9 Distribution of Displacements and Forces
(9 Storied,L-direction, to Yerevan 400gal)

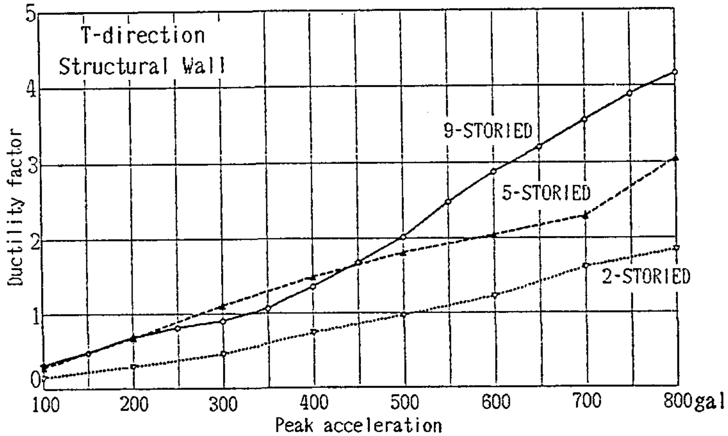


Fig. 10 Maximum Ductility Factor of Structural Wall at the First Story
(to Gukasian Accelerogram)

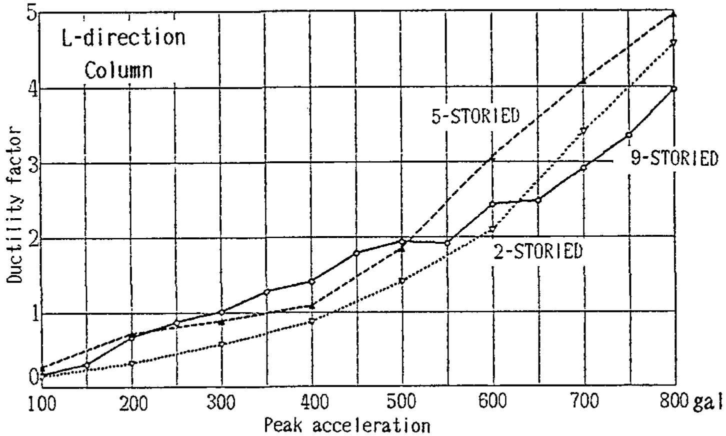


Fig. 11 Maximum Ductility Factor of Column at the First Story
(to Gukasian Accelerogram)