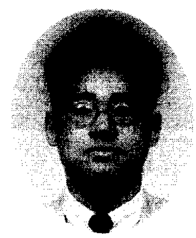
비내진 상세를 가진 1:12축소 10층 R.C.골조의 비선형 거동에 관한 실험 연구

Experimental Study on Nonlinear Behaviors of A 1:12 Scale 10-Story Reinforced Concrete Frame with Nonseismic Details

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ABSTRACT

The objective of this experiment is to observe the elastic and inelastic behaviors of high-rise reinforced concrete frames having non-seismic details. To do this, a building frame designed according to Korean seismic code and detailed in the Korean conventional practice was selected. A 1:12 scale plane frame model was manufactured according to similitude law. A reversed lateral load test and a monotonic pushover test were performed under the displacement control. To simulate the earthquake effects, the lateral force distribution was maintained to be an inverse triangle by using a whiffle tree.

From the tests, base shears, crack pattern, local rotations in the ends of critical members and the relations between interstory drift versus story shear are obtained. Based on the test results, conclusions are drawn on the implications of the elastic and inelastic behaviors of a high-rise reinforced concrete frame having non-seismic details.

keywords ; inelastic behavior, non-seismic detail, reversed lateral load test, monotonic pushover test, displacement control, whiffle tree, model, prototype, similitude law

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1. Introduction

Most of the high-rise reinforced concrete buildings designed and constructed in Korea generally have non-seismic details, and therefore may show seismic responses and failure modes different from those of buildings with seismic details in the regions of high seismicity.

The behaviors of building structures when subjected to severe earthquakes may undergo a non-linear plastic deformations beyond the linear elastic limit. In this case, to ensure the overall safety and to prepare for the brittleness of the structure with nonseismic details, it is necessary to predict its inelastic behaviors. But assumptions made in establishing analytic models for the available non-linear analysis programs are generally based on the test results obtained from the structures with seismic details. Therefore, this necessitates the calibration of these analysis programs through experimental study on the structures with nonseismic detail. Unlike the testing of a structural element, however, full-scale tests of structures are very difficult to perform due to the restrictions of available structural laboratories and other problems such as time and cost.

Accordingly, a 10-story reinforced concrete building frame with non-seismic details was selected for this study, and its test model was manufactured at the scale of 1:12 to suit the given conditions of the laboratory. And lateral load tests(reversed lateral load test and monotonic pushover test) were performed under the displacement control on the roof to experimentally observe the responses of the reinforced concrete frame to lateral forces in the linear elastic and inelastic regions.

2. Design and Manufacture of Test Model

2.1 Selection of Test Model

The test model selected for this study was a 10-story reinforced concrete building with non-seismic details which has actually been built and is now in service as an office building in Korea. **Fig. 1** shows the plan of this building.

The design base shear of the moment frame as shown in the shaded region in **Fig. 1** is 46.07 tonf which is calculated using Eq. (1) in Korean seismic code[7].

$$V = \frac{AICS}{R} W \qquad (1)$$

where,

A(Zone factor) : 0.12

I(Importance factor) : 1.2

S(Soil factor) : 1.0

R(Response modification factor) : 3.5[8]

T: $0.06h_n^{3/4} = 0.843$ sec.

h_n(Height of building):33.9 m

C(Dynamic factor): $1/1.2\sqrt{T} = 0.907$

W(Effective weight of frame) : 1,234.6 tonf.

The corresponding design base shear of the 1:12 model is 0.32 tonf according to the similitude law. It is interesting to note that this structure was designed using the response modification factor, 3.5, which corresponds to the bearing wall system with the boundary elements in Korean seismic code.

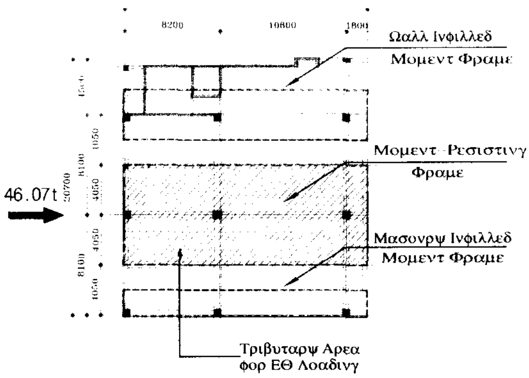


Fig. 1 Plan of the actual R.C. building and its moment-resisting frame (unit : mm)

2.2 Manufacture of Test Model[3]

Types of concrete and reinforcements used for manufacturing the model are shown in **Table 1**. Model reinforcements were deformed with a deforming device to simulate the deformation on the surface and heat-treated by a vacuum electric furnace to get the clear yield point and ductility. Heat-treatment was so performed that the yield force (kgf) rather than the yield stress (kgf/cm^2) of model reinforcement meet the value required by the similitude law because of the difficulty in making the section area of model reinforcements exactly comply with this law. The temperature and the duration of the corresponding temperature were controlling parameters.

Since the vacuum electric furnace was tube-shaped, it was very difficult to maintain uniform temperature throughout the whole length of the tube. **Fig. 2** shows the results of tension test of D2B model reinforcement zone by zone. Zone 1, the part of model reinforcement in the entrance portion which has always shown distorted results, was not used in manufacturing the test model. In **Fig. 3**, the force-strain relation of actual and model reinforcements are compared. It is seen that the model yield forces meet the requirement of the similitude law to the reasonable order of errors.

The size of sand and aggregate used for the model concrete was controlled to meet the scale factor. Aggregate which passed through the 2.08mm sieve and remained on the 0.42mm sieve is classified as coarse aggregate; aggregate which passed through the 0.42mm sieve is defined as sand. The mix ratio by weight is shown in **Table 2.**

The form of this model was constructed so that the model lay on its side in the horizontal surface for convenience. After the cages of model reinforcements were assembled in the vertical positions and laid down into this form, concrete was placed (see **Photo 1**). The curing temperature was maintained at about 20℃. After an elapse of

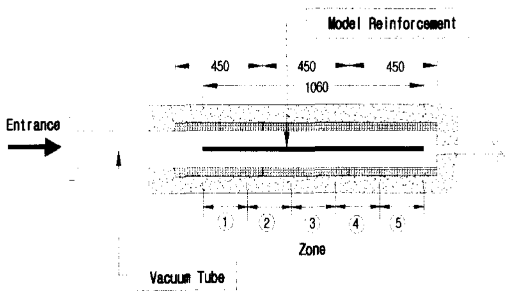
Table 1 Types of concrete and reinforcements

Concrete (f_c' : kgf/cm^2)		Reinforcement			
		Prototype		1:12 Model	
Prototype	1:12 Model	Main Bars (Column, Beam)	Hoop, Stirrup	Main Bars (Column, Beam)	Hoop, Stirrup
210	245.1*	D25(SD40)	D10(SD30)	D2A	ϕ1.0
		D22(SD40)		D2B	

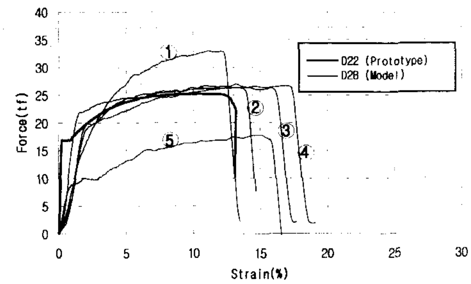
* Mean value of compressive strength(f_c') based on ϕ 5cm×10cm cylinder (28 days)

Table 2 Mix ratio of model concrete

	W/C(%)	Water(g)	Cement(g)	Sand(g)	Coarse Aggregate(g)	Super-Plasticizer(g)
Model Concrete	57	162	256	300	338	2.56



(a) Zones in tube of electric furnace



(b) Force-strain relation of model reinforcement for each zone

Fig. 2 Results of tension test for D2B (gauge length : 10cm)

Photo 1 View of form and arrangement of model reinforcement

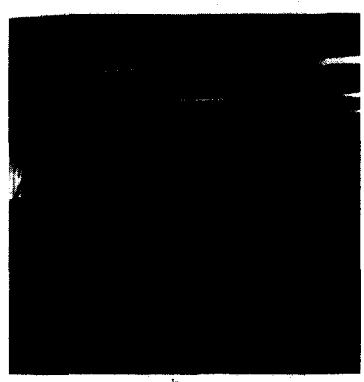


Photo 2 View of removed form and model

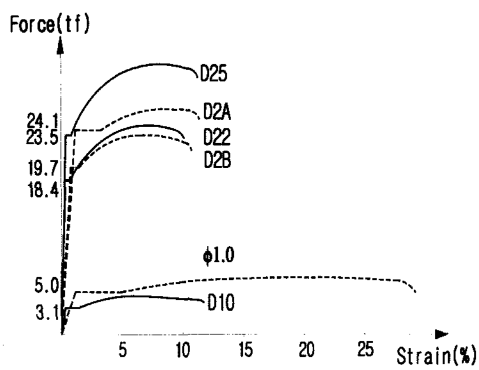


Fig. 3 Force-strain relation of model and prototype reinforcement
(adjusted according to the similitude rule)

about 3 weeks, the model was erected together with the form and the form was removed as shown in **Photo 2**. Curing in air continued in this state. After an elapse of about 50 days from the time of the removal of the form, tests were carried out.

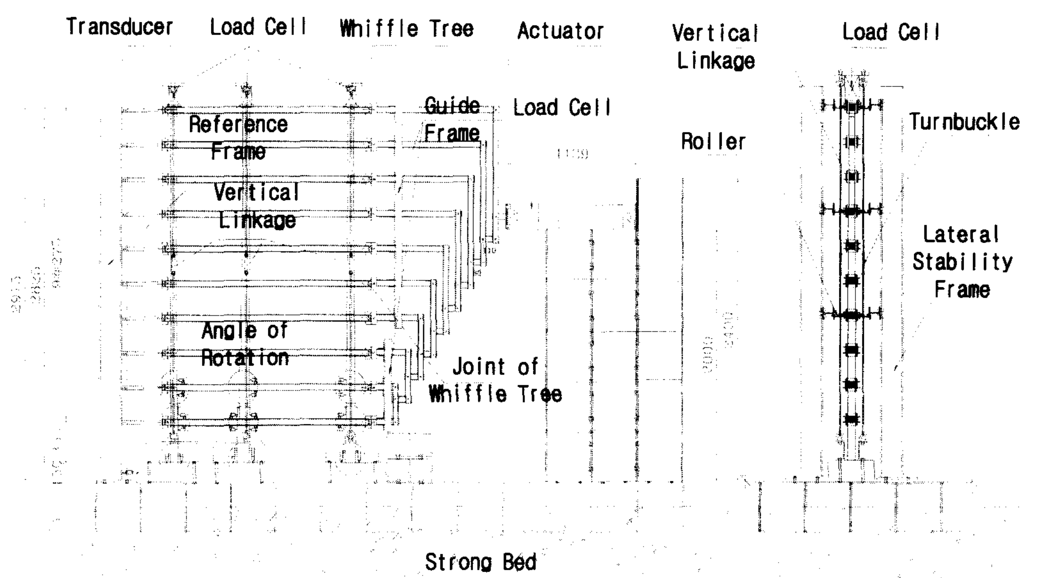
3. Experiment

3.1 Testing Setup and Equipment

Fig. 4 shows the whole experimental setting of the model. The model was so slender that it swayed laterally. To provide stability and to secure the safety in testing, a frame against lateral displacement was set up with 4 rollers furnished on the sides of the 4th, 7th floors and the roof. The lateral load tests under the control of the lateral displacement at the roof were done quasi-statically by an actuator. The distribution of lateral forces was made to be in the shape of an inverse triangle by using the whiffle tree. To prevent the lateral movement of the whiffle tree, a special guide frame was installed. **Fig. 5** shows the relative ratios of the story forces and shears transmitted to each story by the whiffle tree. The joints between the whiffle tree and the model were made to act as hinges by the careful detailing as shown in **Fig. 6.**

Displacement transducers were instrumen--ted to measure the lateral displacements of the floors and the rotational angles in the ends of the column of the first floor and of the beams of the second and third floors.

The effects of gravity load are accounted for only in the columns by applying directly the axial forces on the columns. However, these axial forces were kept uniform throughout the height of the model by the use of the vertical linkage composed of loadcells, steel bars and turn-buckles as shown in **Fig. 4.** The axial loads required in each column at the first story were 1.6 tonf, 3.3 tonf and 2.7 tonf, respectively. The use of the vertical linkage, however, means a constant axial force throughout the entire height. Since it was thought to beexcessive to apply the axial force of the first story throughout the height of the model, axial loads which seem to act at the intermediate stories - 0.68 tonf, 1.4 tonf and 1.15 tonf, respectively - were selected.



(a) Front view (b) Side view

Fig. 4 Experimental setup of test model (unit : mm)

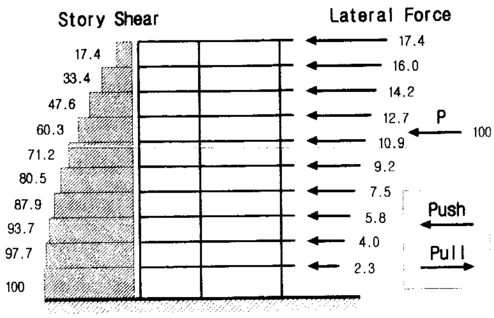


Fig. 5 Relative story shear and floor forces (%)

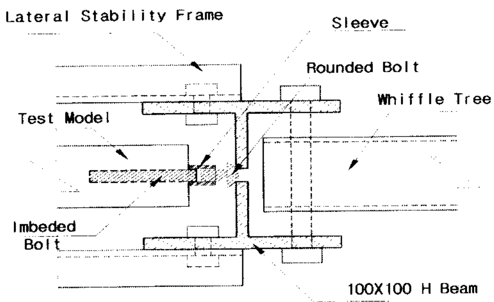


Fig. 6 Joint between whiffle tree and model

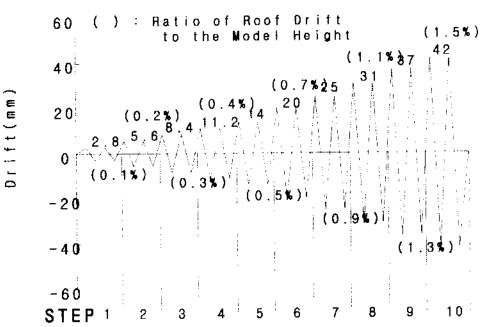


Fig. 7 Impsed drift history for RLL test

3.2 Test Program

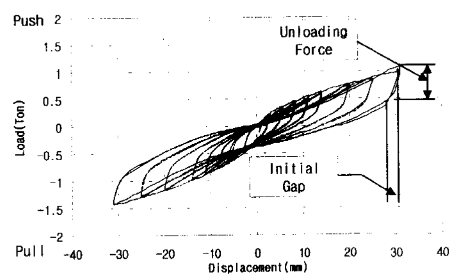
To observe the responses of the concrete frame to the lateral force, the lateral-load tests were performed by the two procedures using the 30-tonf loadcell, the actuator and the whiffle tree connected with the model.

(1) Reversed lateral load test(RLL test) : this test was carried out under the control of the displacement at the roof. **Fig. 7** shows the imposed displacement history[2].

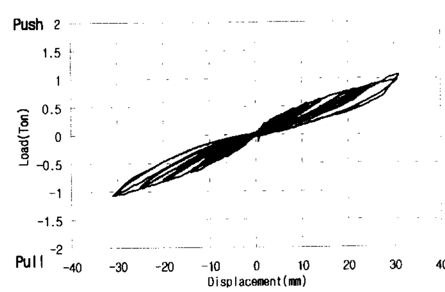
(2) Monotonic pushover test(MPO test) : this test was performed up to the measurable range of the used displacement transducer at the roof.

4. Test Results and Interpretations

The initial intention of the program for the reversed lateral load test was to apply the lateral roof displacement up to ±42mm, 1.5% of the height of the model. But at step 9(±37mm, 1.3%), the test was interrupted due to the local failure in the cantilever beams at the 8th and 10th floors during pushing stage. After repairing these portions, the MPO test was performed in the pulling direction lest it should affect these damaged beams.



(a) Before reduction



(b) After reduction

Fig. 8 Roof drift versus load relation (RLL test)

4.1 Errors in Measurement

We had two sources of errors in the measurement in RLL test. The first is the error in the loading stage which may originate from the friction force between the whiffle tree and the corresponding guide frame. The second is the error from the initial softening under the unloading stage due to the invisible gaps in the connections between the model and the whiffle tree or between the pieces in the whiffle tree. However, the errors of the second kind looked significant. So, we reduced the test data through the subtraction of unloading forces and initial gaps as shown in **Fig. 8(a)** to obtain the result of **Fig. 8(b)**.

4.2 Story Shear versus Inter-Story Drift

Fig. 9 shows the envelope of drifts for each displacement step in RLL test and for the maximum roof displacement in MPO test. **Fig. 10** shows the hysteretic relation between the roof drift and applied total lateral force for RLL test and the superposed monotonically increasing relation for MPO test. It can be found that the envelope to the hyteresis curves by RLL test looks to be the initial portion of the curve which would be obtained if MPO test only were implemented without prior RLL test.

The relations between story shear versus interstory drift are shown in **Fig. 11**. In **Table 3**, the maximum interstory drift ratio(I.D.R.), the maximum story shear, and the story ductility ratio of each story, are listed. **Fig. 10** reveals that the yielding of the whole structure occurred progressively with a rounded curve, while **Fig. 11** indicates that the yielded parts are concentrated in the middle stories rather than in the lower stories. From **Fig. 11** and **Table 3**, it can be observed that the 6th story has the largest interstory drift (about 19mm), and I.D.R. (6.95%) where the ductility ratio is about 2.4. The story stiffness tends to be lower when the story goes higher. Though the drift ratio of the whole structure is about 3.7%, the I.D.R. of the same structure vary from 1.02% to 6.95%. In this particular case of the structure, as shown in **Fig. 11(e)**, the yielding point of the 6th story appears clear and to be 1.09 tonf. So the yielding load of the whole model structure is defined 1.09 tonf.

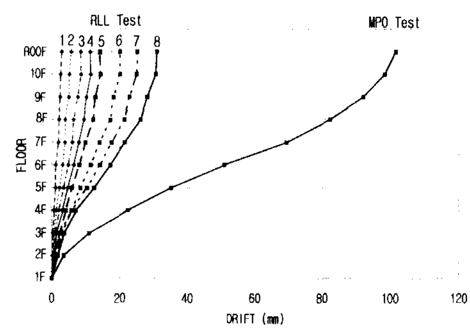


Fig. 9 Envelope of drifts

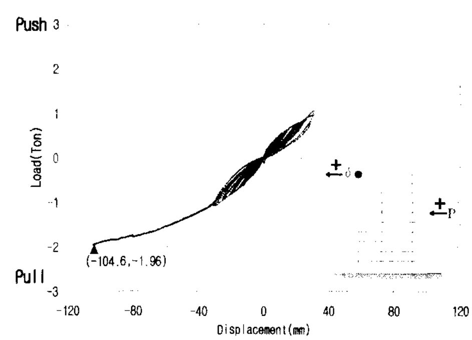
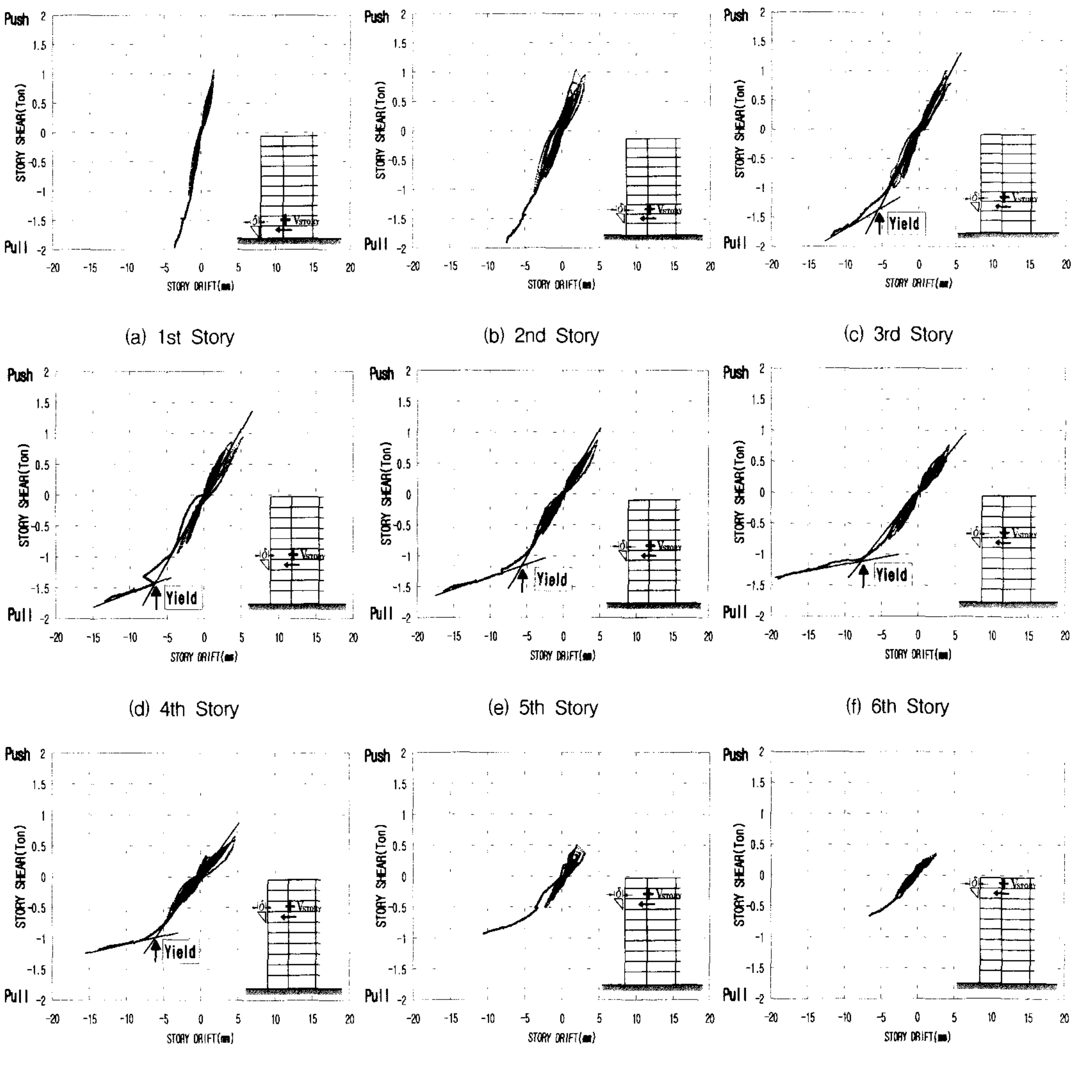


Fig. 10 Relation of roof drift versus lateral load

Table 3 Maximum interstory drift ratio and story shear

Story	Initial Stiffness k (tf/mm)	Yield Inter-Story Drift (mm)	Max. Inter -Story Drift (mm)	Max. Inter-Story Drift Ratio(%)	Story Ductility Ratio	Absorbed Energy (tf-mm)	Max. Story Shear (tonf)
1st	**0.606**	-	3.57	1.02	-	4.04	**1.96**
2nd	0.290	-	7.46	2.71	-	7.57	1.91
3rd	0.279	5.75	11.93	4.34	2.07	13.29	1.83
4th	0.243	6.65	13.34	4.85	2.01	14.07	1.72
5th	0.188	6.09	16.31	5.93	**2.68**	17.41	1.58
6th	0.184	**8.46**	**19.1**	**6.95**	2.26	**18.32**	1.39
7th	0.140	6.87	13.75	5.0	2.00	10.83	1.18
8th	0.156	-	10.6	3.85	-	6.29	0.93
9th	0.129	-	6.61	2.40	-	2.53	0.65
10th	0.045	-	2.6	0.95	-	0.55	0.34

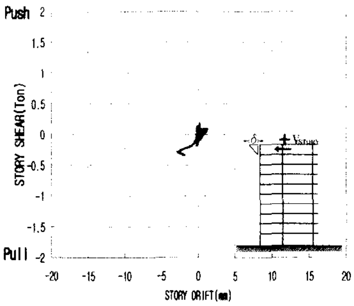


(a) 1st Story (b) 2nd Story (c) 3rd Story

(d) 4th Story (e) 5th Story (f) 6th Story

(g) 7th Story (h) 8th Story (i) 9th Story

Fig. 11 Interstory versus story shear



(j) 10th Story

Fig. 11 Interstory versus story shear (continued).

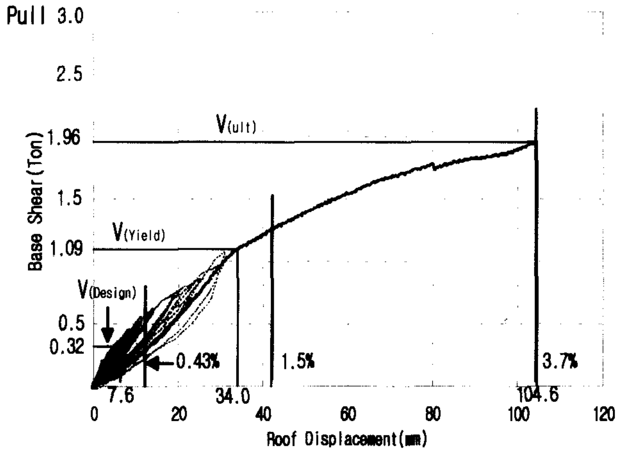
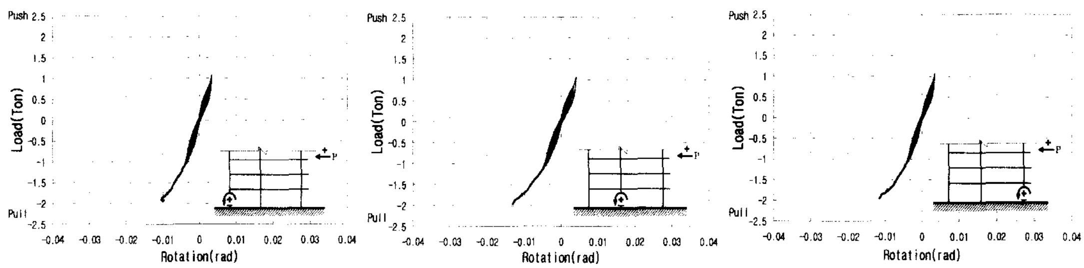
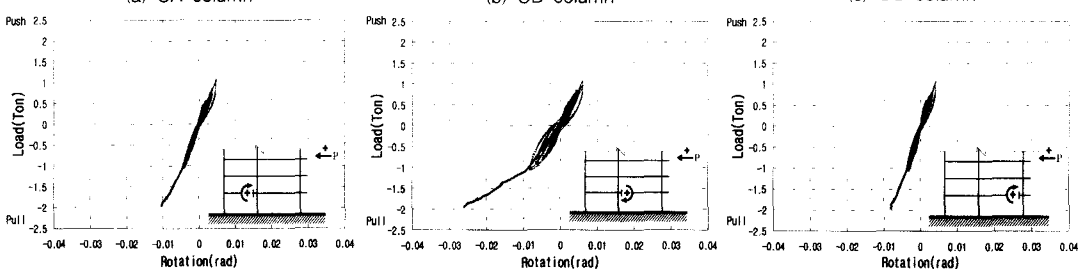


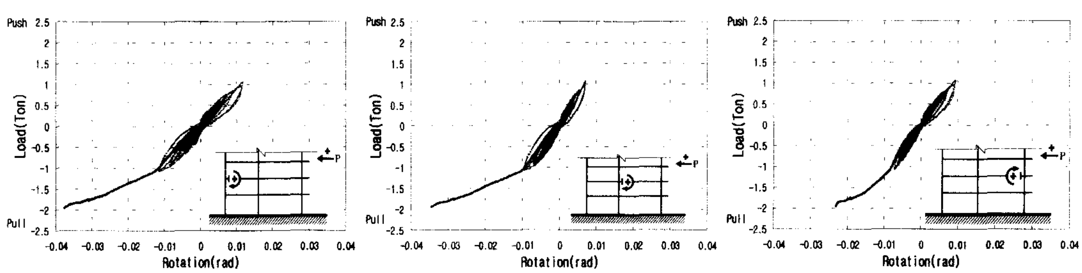
Fig. 12 Relation between base shear and roof drift



(a) CA column (b) CB column (c) CC column



(d) 2nd floor beam end I (e) 2nd floor beam end II (f) 2nd floor beam end III



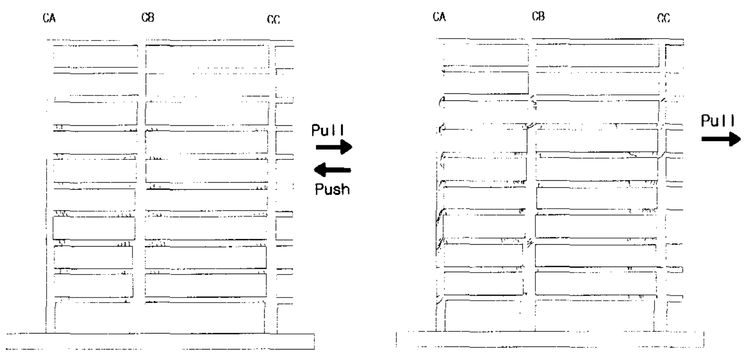
(g) 3rd floor beam end I (h) 3rd floor beam end II (i) 3rd floor beam end III

Fig. 13 Angular rotation in the ends of members

In **Fig. 12**, the design, yielding and ultimate base shears are plotted on the curve of the base shear versus the roof drift. The yielding base shear of the test model was about 1.09 tonf, which was about 3.41 times 0.32 tonf, the design base shear of the model. The ultimate base shear is about 1.96 tonf. It was about 6.13 times the design base shear and 1.80 times the yielding one. The I.D.R. corresponding to the design base shear is within 0.43% which is the allowable limit if the linear elastic analysis is used for the design base shear. It can be also noted that if the 1.5% limit on the lateral drift is assumed to be the maximum allowable under severe earthquakes, the usable ductility ratio seems to be about 1.24.

4.3 Local Deformations

To observe the formation of the plastic hinges in the critical regions of the structure, displacement transducers were instrumented as shown in circles in **Fig. 4**.



(a) After RLL test

(b) After MPO test

Fig. 14 Crack Patterns

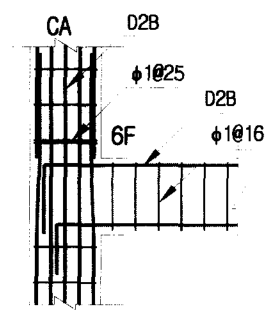


Fig. 15 Detail in external joint at 6th floor

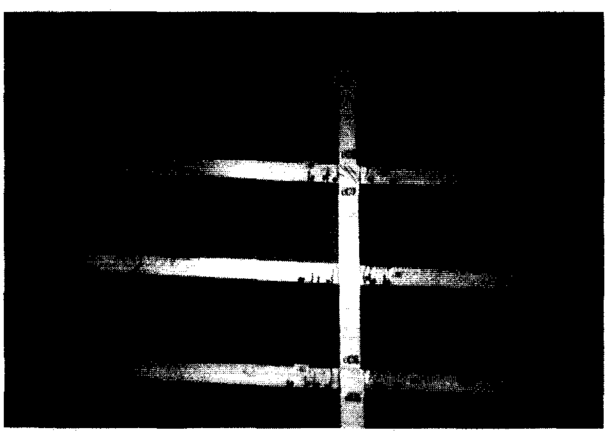


Photo 3 Cracks at middle stories

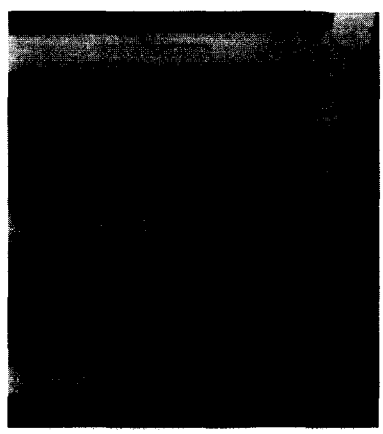


Photo 4 Expanded view of CA column's cracks

We can observe that plastic hinges have formed in one end of the long-span beam at the second floor and in the ends of short-span and long-span beams at the third floor while no hinge occurred at the base of the internal column. This means that the interior column has not yielded while most of yielding has occurred in the ends of beams.

As shown in **Fig. 13** the angles of rotations have the values ranging from 0.01 rad to 0.039 rad at the lower three stories. This implies the demand on the deformation capacity at the critical regions for the 3.7% of the roof drift ratio of the whole structure. Considering that most of inelastic deformations are concentrated in the middle stories in this model, the maximum demands on the plastic rotations corresponding to 3.7 % of structural drift ratio must be much higher in the middle stories though these were not measured in this test.

4.4 Crack Patterns

In the RLL test, the initial crack occurred at step 4(0.4%, ±11.2mm). The cracks propagated gradually to the higher stories until reaching the 9th floor at step 9 (1.3%, ±37mm) as shown in **Fig. 14(a)**. Cracks occurred mainly on the beams and were not visible on the columns.

The cracks caused by MPO test were found in the column and the beam-column joints as well as in the beams. Most of the flexural cracks were concentrated on the beams.

Particularly, many of them were found on the intermediate stories. A considerable level of damage was found on the columns and the phenomena of shear failure were observed in the external joints at the later stage of MPO test. **Photo 3** shows the crack patterns after the monotonic pushover test. **Photo 4** indicates the mode of shear failure which the details in this external joint as shown **Fig. 15** can undergo when subjected to the large yielding moments at the connected column and beam.

5. Conclusions

1) The pushover test performed on the 1:12 scale 10-story reinforced concrete frame reveals that the test errors may be developed due to (1) the invisible gaps between the model and whiffle tree or between the part of whiffle tree (2) and the friction force between whiffle tree and corresponding guide frame. Therefore, researchers should be very careful to reduce or prevent this kind of experimental error.

2) The model structure has shown gradual inelastic or yielding curve in the relation between the base shear and the roof drift though the 6th story indicate clear yielding point. Hence the yielding of any certain member or any story does not necessarily lead to the clear indication of the yield of the whole structure when we are dealing with high-rise structures.

3) In this particular case, the ratio of the yielding base shear to the ultimate appears to be 1.8 and the ratio of the yielding base shear to the design one is about 3.4. This means that even the R.C. frame with non-seismic details can resist quite high level of earthquakes(up to $V_{ult} / W = 1.96 / 8.57 = 0.229$).

4) The failure mode was generally flexural. However in the ultimate state, the

external joint revealed quite special shear failure mode due to the special detailing.

5) The ductility or deformation capacity of the model structure turns out quite high. However, considering the allowable limit on I.D.R., the usable portion of this ductility or deformation capacity will be limited.

Acknowledgments

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요 약

본 연구의 목적은 비내진 상세를 가진 고층 R.C골조의 탄성 및 비탄성 거동을 실험적으로 살펴보는 것이다. 따라서, 국내의 내진 설계규준에 따라 설계 및 시공된 건축물이 선정되었으며, 상사법칙에 따라 1:12의 축소율의 평면 골조모델이 제작되었다. 실험방법은 옥상층의 변위제어에 의해 반전횡하중 실험과 일방향 가력실험을 수행하였다. 지진효과를 나타내기 위하여, 횡력은 휘플트리를 이용하여 각층에 역삼각형 형태로 분포되었다.

실험으로부터 밑면전단력, 균열양상, 주요부재 단부에서의 국부 회전각 및 층간변위와 층전단력과의 관계를 얻을 수 있었다. 실험결과로부터 비내진 상세를 가진 고층 철근콘크리트 골조의 탄성 및 비탄성 거동에 대해 살펴보았다.

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