

# SIMULATION OF DAMAGE PROGRESSION IN LOWER STORIES OF 11-STORY BUILDING

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## SUMMARY

Seismic behavior of two reinforced concrete frames with two stories and one span were investigated in Kyoto University. These frames were scaled to 1/4 and represented the lower part of an 11-story reinforced concrete frame building prototype. They were identical and designed with the 1999 Japanese guidelines. Axial load variation was the only test parameter for this experiment. From the test results it was found that, slight difference was observed between the two frames from the experimental load-drift relationship. Both frames did not show any strength degradation even though they were loaded beyond 6 % drift. The second floor beam elongated as much as 1.50% of the total span length for both frames. Some of the beam's longitudinal reinforcements buckled near the column face due to high compression. Frame under high axial load showed more cracks than the one under moderate axial load. Analysis of the frame specimens was carried out with the nonlinear IDARC program. The analytical curvature-drift relationships for frame components matched well the experimental ones, for a plastic hinge lengths equal half of the column depth and half of the beam height. Good agreement was also found for the load-drift at the first story, second story and the entire frame. Pushover analysis carried out using the nonlinear SAP2000 predicted with a very good accuracy the envelope curves of the experimental hysterises curves. The plastic hinge region was modeled in SAP2000 using the tri-linear model suggested in the Japanese design guideline.

### **INTRODUCTION**

Many researchers [1][2] investigated deeply the seismic behavior of cantilever column under different types of loading in the past. However, tests data for frame structures or beam-column assemblages are still not available in the same amount as that for isolated columns or beams. Presence of beams and slabs in structure may change the column's seismic behavior dramatically. During Northridge earthquake many buildings collapsed as a results of flooring units loosing their seating due to beam elongation [3].

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Stanton et al. [4] proposed a new "yielding gap frame" joint connection to avoid the beam elongation in the precast prestressed frame structures. The beam is connected to the column at the bottom by post tensioning tendon that passes through, and pre-compresses, a grout. At the top, the connection is made by deformed bars grouted into ducts. The grout pad exists only at the bottom of the beam, so there is no contact between the two concrete faces at the top of the beam. During earthquake, the end of the beam rotates about the grout pad, which acts as a hinge. The distinguishing characteristic of the connection is that it overcomes the problems associated with the beam elongation that typically occurs under plastic rotation.

This study is the second phase in our testing program that aims to establish in the future a procedure to predict damage level of moment resisting RC frame buildings. In the first phase, sixteen isolated full-scale and reduced-scale cantilever columns were investigated under different axial and horizontal loading patterns [5][6]. In this second phase, presented in this paper, two reinforced concrete frames with two stories and one span were designed and tested in Kyoto University to investigate the seismic behavior of the lower part of mid high-rise frame buildings. These frames were scaled to 1/4 in order to fit the loading system. The reinforced concrete frames were designed with the 1999 Japanese guidelines [7]. Quantification of bending moment, axial load and shear force distributions at the first story column bases was one of the author's main interests. Also, the test program aimed to measure beam and column elongation and shortening as well as comparing the axial strain of the first story columns to those of cantilever columns tested previously. The last target was the analytical prediction of shear force-drift at each story as well as the deformation of beams and columns.

## **EXPERIMENTAL PROGRAM**

#### **Test setup**

To evaluate the axial load, shear load and the bending moment at the first story column bases, four identical load cells were designed and calibrated before the test. A regression analysis was carried out to evaluate the coefficient for each set of load cells. Good agreement was found between the test results and the analytical results for the axial, shear and bending moment. An example for the prediction of bending moment is shown in Figure 1, where a  $45^{\circ}$  straight line can be seen.



Figure 1: Moment calibration result

The cross section was 270x270 mm for columns and 180x270 mm for beams. The heights of the first and second floor were 765 and 840 mm respectively. The span length was 1800 mm as shown in Figure 2 (a). Load cells were inserted under each foundation as shown in Figure 2 (b). The horizontal load was applied through a 1000 kN jack at mid height of the third floor. A 40 mm diameter high strength bar passing

through the column center was used to simulate the axial load variation of the third floor column based on the elastic frame analysis. The bar was used to apply either compression or tension to the columns by two jacks. The axial load, N, varied linearly with respect to the applied horizontal load, H, as follows:  $N = 239 + \Psi H (kN)$ , where  $\Psi = 2.30$  for SN30 frame and  $\Psi = 4.59$  for SN50 frame. The concrete and steel mechanical characteristics as well as the test variables are shown in Table 1.



Figure 2: Frame geometry and test setup

Frame designation	Material			Test variable -Axial load-	
	Concrete strength	Longitudinal steel	Shear rebar	Max. Comp. N/f'cD <sup>2</sup>	Max. Tension N/f'cD <sup>2</sup>
SN30		Column 12D16 (3.27%) Fy=346 MPa Beam 8D13 (2.08%) Fy = 332 MPa	Column 4D6@50 (0.94%) Beam 2D6@80 (0.44%) Fy = 394 MPa	0.3	0.1
SN50	31.0 MPa			0.5	0.2

**Table 1: Materials characteristics** 

## **Experimental results**

The experimental load-drift relationship showed a slight difference between the two frames in term of peak load and the loading and unloading stiffness. This can be seen clearly through Figure 3 (b) where load-drift curves of the entire frame are shown. The maximum drift reached for specimen SN30 an SN50 were 6.08% and 7.09%, respectively. The tests were stopped due to the lack of enough space between the top jack that applied the axial load to the south column and an existing steel loading frame.

Shear force, axial load and bending moment at the column base were determined from load cells. It was observed that the total shear force was not distributed evenly to the column bases. The shear force was rather distributed as a function of the applied axial load intensity. As an example, Figure 4 shows the shear force variation at first story column of SN30 and the axial load variation in the north and south first story column of SN50.



(a) Shear force variation SN30 (b) Axial load variation SN50 Figure 4: Shear and axial load variation at first story column for SN30 and SN50

At the same drift angle and for a given story, column under tension showed more elongation than the one under compression as illustrated in Figure 5. However, the second-story columns for SN30 showed nearly the same amount of shortening and elongation as shown in Figure 5 (b). Mean strain was defined as the average of the measured elongation or shortening divided by the clear column height.



Figure 5: Columns shortening -elongation positive- (continue)



Figure 5: Columns shortening -elongation positive-

In both frames and under tensile axial loading, the first story column elongation followed almost identical linear line regardless the amount of applied axial load as illustrated in Figure 5 (a) and (c). Column axial stain under tension was compared to the measured strain of the longitudinal bar located at 35 mm from the column center. The strain gauge was set on this bar at 25 mm from the column base. Figure 6 shows a good matching between the column mean strain and the longitudinal center bar strain. Until this time, no reasonable explanation was found showing the reason of the good matching.



Figure 6: Comparison between the longitudinal bar strain and the north column mean strain

First floor column's mean strains were compared to two isolated cantilever columns tested earlier in Kyoto University. The two cantilever columns had a square section with 242 mm depth. The horizontal load was applied at 625 mm from the column base. Material and test variables are given in Table 2. Axial load varied from zero to  $0.6A_g f'_c$  for both columns but with different slope on the normalized M-N interaction curve as shown in Figure 7 (a).

	spec	Test variables		
Specimen designation	Concrete strength f <sup>c</sup> (MPa)	Longitudinal rebar (ratio) [Fy]	Shear rebar (ratio) [Fy]	Slope in normalized moment-axial force relation
D1NVA	26.8	12-D13 2.60% 467MPa	F4@40 0.52% 604MPa	1.39
D1NVB				2.79

Table 2: Cantilever column's characteristics



Figure 7: Axial load variation on the MN interaction curves

First story column mean strains were much higher than those for cantilever columns, especially for columns under high axial load of SN50 frame as illustrated in Figure 8. Even though the normalized compressive axial load,  $N/A_g f'_c$ , was 0.5 for specimen SN50's columns and 0.6 for the cantilever

columns, the axial strain of 1FNC-SN50 under compression at 3% drift was more than three times larger than those of D1NVA and D1NVB. Normalized moment-drift relationships for the first story south column (1FSC) of specimen SN30 and the cantilever column D1NVA are shown in Figure 9. D1NVA showed a drop after reaching the peak around 1% drift, however the south column did not show any strength degradation. As mentioned previously in section "experimental results", the shear force at the column base was not evenly distributed to columns and so is the bending moment. This can be seen clearly when comparing D1NVA and 1FSC in the negative cycles in Figure 9, where a big gap can be observed between the two curves. This behavior cannot be detected while testing a single isolated column hence, the importance of such frame testing.



Figure 8: Comparison of axial strain of cantilever and first story SN50 columns

First and second floor beams were severely damaged especially near the beam -column joint. Using the displacement gauges attached directly to the beams, beam's length changes were evaluated. Best fit for the envelope curves of each beam was computed and compared to Reference [8]. In this reference and based on the mathematical expression proposed by Fenwick and Magget [9] and Restrepo [10], the beam elongation was reported to be 2 to 5% of the beam depth per plastic hinge. Using the clear beam length, and taking into account two plastic hinges per beam, the mean strain in this test frame was found to be

between 0.71 to 1.76%. The best fit for the second floor beam of frame SN50, shown in Figure 10, gave the maximum mean strain of 1.59% at 6% drift, which is within the range given by Reference [8].



Figure 9: Experimental normalized moment



Figure 10: Second floor beam elongation of SN30

All the best fits, mean strain-drift relationships, had a linear equation passing through the origin with a form of y = ax. The "*a*" coefficients for frame SN30 were 0.130 and 0.246 for the first and second floor beam respectively. These values were 0.129 and 0.225 for frame SN50 that are nearly the same as those for SN30. Taking an average of the above coefficients, Eq. 1 and Eq. 2 can be used to evaluate the beam mean strain,  $\mathcal{E}$ , at the first and second floor respectively:

$$\varepsilon = 0.129(D/H)$$
  
Eq. 1  
 $\varepsilon = 0.236(D/H)$ 

Eq. 2

where *D* and *H* are respectively the top frame displacement and the total frame height as shown in Figure 3 (a). It is obvious that the beam elongation will amplify the column bending moment demand on one side of the frame and will reduce it for the other side, due to the increase in the  $P - \delta$  effect and horizontal displacement.

#### ANALYTICAL RESULTS

## **IDARC** results

#### Frame load-story drift relation

Analysis was carried out using the nonlinear IDARC [11] program. Both frames were modeled as a lumped mass. In IDARC program the moment-curvature analysis is carried out on the cross-section using a fiber model. The incremental curvature that is applied to the section is continued until the specified ultimate compressive strain in the concrete or the specified ultimate strength of one of the rebar is reached. Figure 11 shows a sample of the results consisting of the shear force-drift of the entire SN30 and SN50 frames respectively. The analytical load-story drift results, showed a higher stiffness than the experimental one at low cyclic loading. However, maximum peak and no drop of horizontal load capacity were predicted with a good accuracy.



Figure 11: Frame drift-shear force relationship

#### Column curvature-story drift relation

The IDARC program includes a spread plasticity formulation. The formulation can capture the change in the plastic length under a single or double curvature conditions. The penetration length is updated at each step in the analysis as a function of the instantaneous moment diagram in the element, but the penetration length is never allowed to become smaller than the previous maximum. In other words, the program uses a variable plastic hinge length. As shown in Figure 12, a good agreement was found in term of curvature-story drift relationship for all columns. The experimental curvature was computed using displacement gauges readings placed at  $D_c/2$  from the column base with  $D_c$  is the column depth. Contribution of upper part of the column to the frame top displacement was found to be negligible.

#### Beam curvature-story drift relation

Good agreement between the experiment and the predicted curvature-story drift relationship was also found for frame SN30 beams using a plastic hinge length equal half of the beam height. For the first floor beam of specimen SN50, and as shown in Figure 15 (b) for the crack spreading in that beam, the experimental curvature was computed using the strain gauges reading placed at  $H_b$  where  $H_b$  is the beam height. For the second floor beam this value was set to  $0.5H_b$ . Figure 13 shows a comparison between the analytical and the experimental results of the first floor beams for both frames.



Figure 13: Beam curvature-story drift relationship

## SAP2000 pushover analysis

To predict the envelope curve of the shear force-drift relationship pushover analysis was carried out using SAP2000 program [12]. This software possesses a pre-processor and post-processor that makes it very easy to deal with. Columns and beams were modeled with a beam element that exists in the library of the program. Plastic hinges where introduced at each element ends having the characteristics recommended by the Japanese design guidelines [7] that are not given here due to space and size limitation. Figure 14 (a) shows the shape and the different parameters of the tri-linear model. As an example, Figure 14 (b) shows a comparison between the experimental and the analytical SAP2000 results for SN30 frame. The analytical results agreed with the experimental results quite well. Inserting rigid zones at the edge of the columns and beams improved considerably the prediction. In Table 3 a comparison, in term of horizontal peak load Q, between the SAP2000 predictions with and without the rigid zones and the experimental results are compared.

Frame identification	Q-analysis (Witho	/Q-experimental ut rigid zone)	Q-analysis/Q-experimental (With rigid zone)					
	Positive	Negative	Positive	Negative				
SN30	0.846	0.877	0.963	0.998				
SN50	0.838	0.862	0.897	0.923				

Table 3: Comparison between SAP2000 and test results



#### **OBSERVED DAMAGE**

Figure 15 shows the crack pattern at 2% drift angle for both frames. More crack can be seen on the SN50 columns than those of frame SN30. No buckling or severe concrete crushing was found for any column. At the end of the test, the outside concrete cover located at the base of the first floor column either in the north or south side of the frame was found to be 26 cm for SN50 and 15 cm for SN30 as shown in Figure 16. Even though the spacing of the shear rebar was as much as 80 mm, six times the longitudinal bar diameter, buckling of the longitudinal reinforcement of the second floor beam ends were observed for both frames. Concrete of the lower part of the south side of the second floor beam crushed due to high compression, and its length was found to be 10 cm for frame SN30 and 20 cm for frame SN50. The same crushing was found at the upper part of the north side of the second floor beam with 10 cm length for both frames due also to high compression force. Damage was also predicted analytically using Park et al.'s cumulative damage index [13]. The damage model consists of a simple linear combination of normalized deformation and energy absorption. Figure 17 shows the damage progress at beams and columns of frame SN50. According to Park et al's damage classification, first floor beams sustained a minor damage. Whereas, the second floor beams suffered a severe damage. The first story columns suffered no damage while; the second story columns suffered a minor damage. These classifications were consistent with the observed damage.







Figure 16: Damage at the south column bases



Figure 17: Computed damage for SN50 using park and et al's model

## CONCLUSIONS

To understand the damage progression and the behavior of the lower part of a mid high-rise building, two 1/4 scale reinforced concrete frames were tested with different axial load variation. The main conclusions from this experimental program can be summarized as follows:

- Slight difference was found between the two frames in term of load-drift relationship either for stories or entire frame. Effect of axial load on frame seismic performance was not seen clearly during the test.
- No serious damage was observed for the first and second floor columns due to the high shear reinforcement ratio and by consequence high confinement.
- Even though the stirrup's spacing at beams was six times the longitudinal bar diameter, buckling of longitudinal reinforcement was observed. After buckling of these reinforcements, concrete was severely damaged since it carried the entire compression force.
- ★ Beam elongated as much as 1.5% of the clear beam length. This phenomenon affects the input moment to columns due to the increasing/decreasing in the  $P-\delta$  effect and horizontal displacement.
- Different behavior was found while comparing the axial strain and bending moment of first story column to that of an isolated cantilever column. More investigations are needed to confirm or deny the possibility of predicting the entire frame behavior through a simple isolated test on its components.
- In general, behavior of the entire frame as well as its components, columns and beams, was well simulated using the nonlinear IDARC program. However, analytical stiffness at low cyclic loading was higher than the experimental one.

- Pushover analysis predicted with a good accuracy the shear force-drift envelope curve of the cyclic loadings using the tri-linear model for plastic hinge region recommended by the Japanese guidelines.
- Park et al's damage index reflects the observed damage for beams and columns quite well.

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