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A STUDY ON SEISMIC PERFORMANCE OF FRAME SUB-ASSEMBLAGE SUBJECTED TO AXIAL LOAD VARIATION

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ABSTRACT

To investigate the seismic behavior of the lower part of an eleven-story RC frame building two 1/4-scale RC prototype frames with two stories and one span were subjected to cyclic lateral loads. Variation in axial load was the only test variable between the two frames. The specimens maintained their initial strengths, even at drift ratios of more than 6%. The tests provided measurements of beam elongation; column shortening and variation of shear force in each column. Good agreement was found between the analytical and experimental load-displacement relationships for the first story, second story and the entire frame. The analytical curvature-drift angle relationships for the frame components matched the experimental ones well. Also, the envelope curves were predicted with a good accuracy using pushover analysis.

Keywords: reinforced concrete, frame, load cell, crack, buckling, pushover analysis, damage.

1 INTRODUCTION

Many researchers [1) 2)] investigated deeply the seismic behavior of cantilever column under different type of loading in the past. However, tests data for frame structures or beam-column assemblages still not available in the same amount as for 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 beams elongation[3)].

After the completion of testing sixteen small and large-scale cantilever reinforced concrete columns under different loading histories [4) 5)], 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 highrise frame buildings. These frames were scaled to 1/4 in order to fit the loading system. The reinforced concrete frames were designed with the latest Japanese guidelines [6)]. 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, measuring beams and columns elongation and shortening had a big importance in this test program. The last target was the analytical prediction of shear force-drift at each story as well as the deformation of beams and columns.

2 EXPERIMENTAL PROGRAM

2.1 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° straight line can be seen. Load cells were inserted underneath each column base foundation as shown in Figure 2.

The cross section of the columns was 270x270 mm 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.

The horizontal load was applied through a 1000 kN jack at mid height of the third floor. A 45 mm diameter high strength bar passing through the column center was used to simulate the effect of the above stories. The bar was used to apply either compression or tension to the columns by two jacks.

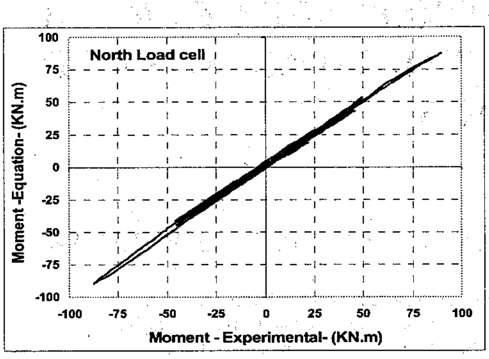


Figure 1: Moment calibration result

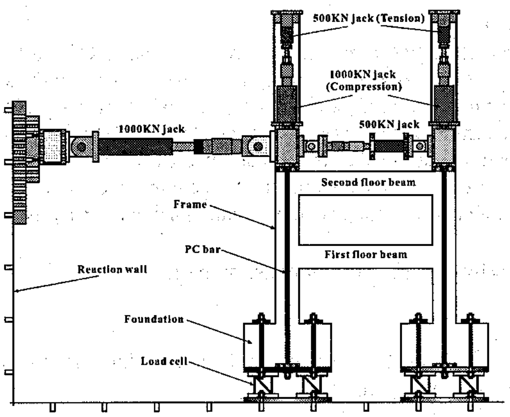


Figure 2: Test setup

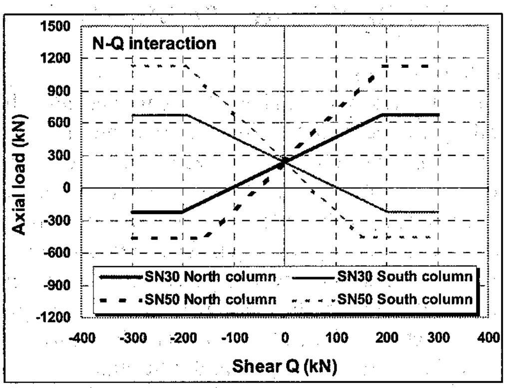


Figure 3: Variation of axial load with respect to shear force

Table 1: Materials characteristics and test variables

Frame designation	Material			Test variable -Axial load-	
	Concrete strength	Longitudinal steel	Shear rebar	Compression N/f'cAg	Tension N/f'cAg
SN30	Before 30.47MPa After 32.07 MPa	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				0.5	0.2

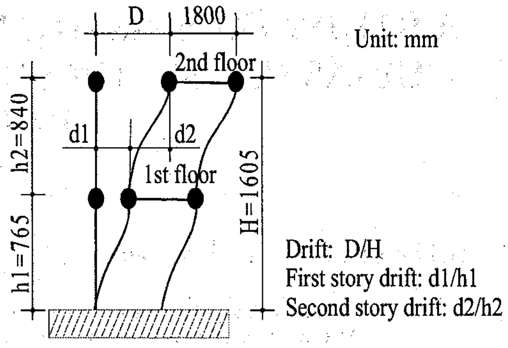


Figure 4: definition of used terms

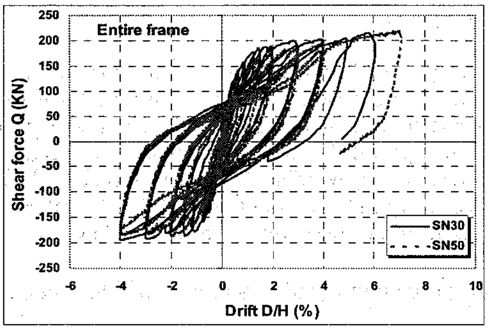


Figure 5: Load-drift relationship for the entire frames

The axial load applied to columns, N, varied linearly with respect to the applied horizontal load, Q, as shown in Figure 3. The concrete and steel characteristics as well as the test variables are shown in Table 1.

2.2 Experimental results

The experimental load-drift relationship did not show any significant difference between the two frames. This can be seen clearly through Figure 5 where load-drift curve of the entire frame is shown. The maximum drift angles reached for specimen SN30 during the test were 6.08% for the whole frame, 5.18 and 6.92% for the 1st and 2nd story, respectively. These values were 7.09%, 5.72 and 8.33%, respectively, for specimen SN50.

Using the load cells and the high strength bars data; shear force, axial load and bending moment at the columns base were determined. 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 6 shows the axial load variation in the north and south first story columns of SN50.

Columns in the 1st story of frame SN30 tended to elongate rather than shorten, especially on the south side.

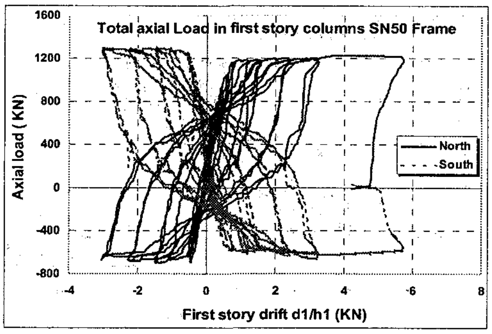
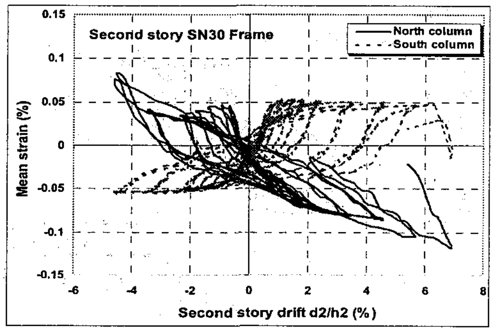
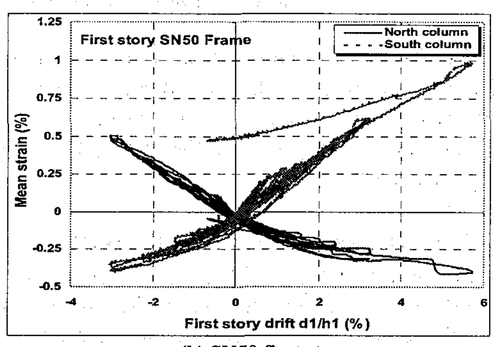


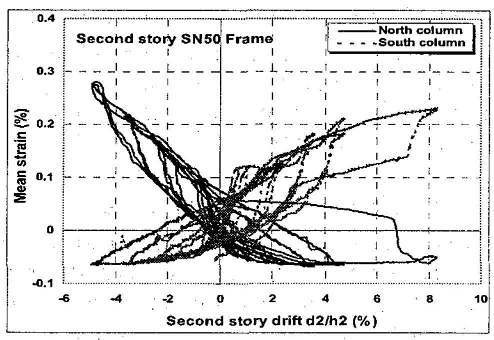
Figure 6: Axial load at the columns base SN50



(a) SN30 Second story



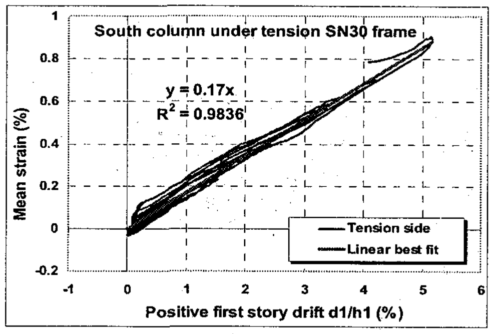
(b) SN50 first story



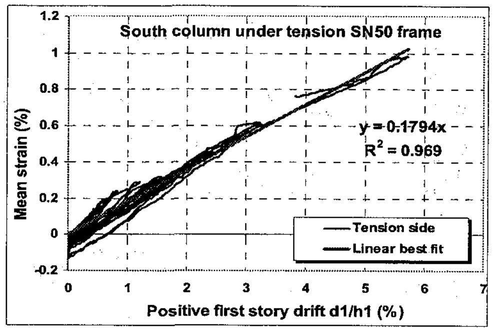
(d) SN50 Second story

Figure 7: Columns shortening -elongation positive-

The second-story columns showed nearly the same amount of shortening and elongation as illustrated in Figure 7 (a).



(a) SN30 frame



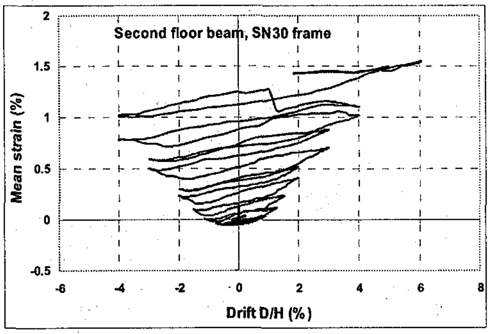
(b) SN50 frame

Figure 8: South columns under tension

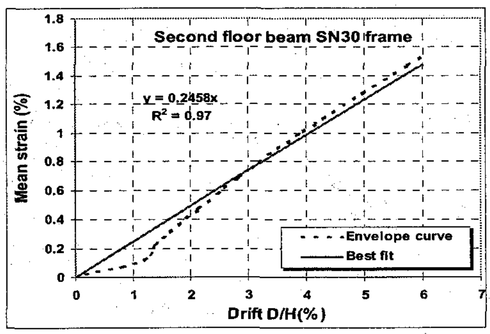
In contrast, all columns in frame SN50, first and second stories, elongated more than shortened as shown in Figure 7 (b) and (c).

Under tensile axial loading, the first story column's mean strains of frame SN30 and SN50 showed the same variation as shown in Figure 8. In both frame the columns elongation followed the almost identical linear equation regardless the amount of applied axial load.

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 [7]. In this reference and based on the mathematical expression proposed by Fenwick and Magget [8] and Restrepo [9], the beam elongation have been observed to be 2 to 5% of the beam depth per plastic hinge. Using the clear beam length, the mean strain was found to be between 0.71 to 1.76%. The best fit for the second floor beam of frame SN50, shown in Figure 9, gave the maximum mean strain of 1.59% at 6% drift, which is within the range given by reference [7]. All the best fits, mean strain-drift relationships, had a linear equation passing through the origin with a form $y = ax$.

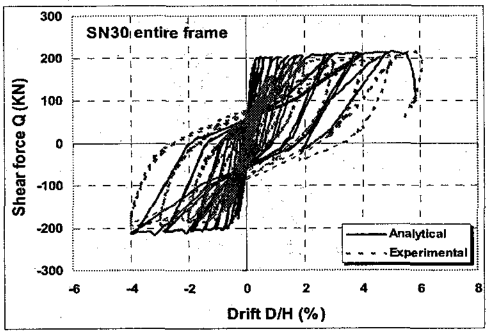


(a) Second floor beam elongation history

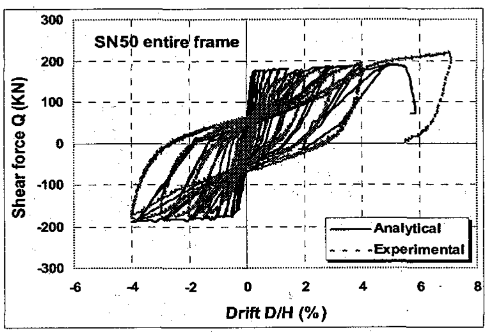


(b) Envelope curve and best fit

Figure 9: Second floor beam elongation of SN30



(a) SN30



(b) SN50

Figure 10: Frame drift-shear force relationship

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, the following equations can be used to evaluate the beam mean strain, ε, at the first and second floor respectively:

$$\varepsilon = 0.129(D/H) \quad (1)$$

$$\varepsilon = 0.236(D/H) \quad (2)$$

where D *and* H were introduced in Figure 4. 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.

3 ANALYTICAL RESULTS

3.1 IDARC Results

3.1.1 Frame load-story drift relation

The analytical results using the nonlinear IDARC program [10)] showed a higher stiffness than the experimental one at low cyclic loading.

However, maximum and no drop of horizontal load capacity were predicted with a good accuracy. Figure 10 shows a sample of the results consisting of the shear force-drift of the entire SN30 and SN50 frames respectively.

3.1.2 Column curvature-story drift relation

As shown in Figure 11, 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 $D_c/2$ where D_c is the column depth. The program IDARC includes a spread plasticity formulation to capture the variation of the section flexibility, and combine them to determine the element stiffness matrix.

3.1.3 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 14 for the crack spreading in that beam, the experiment curvature was computed using the strain gauges reading placed at H_b where H_b is the beam height.

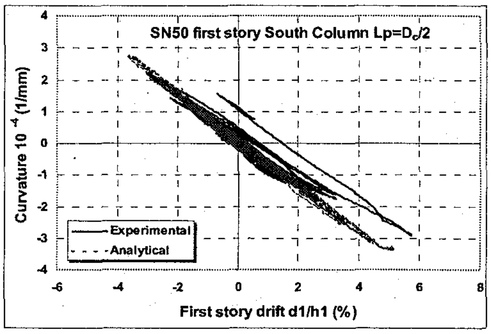


Figure 11: South column curvature-story drift relationships SN50

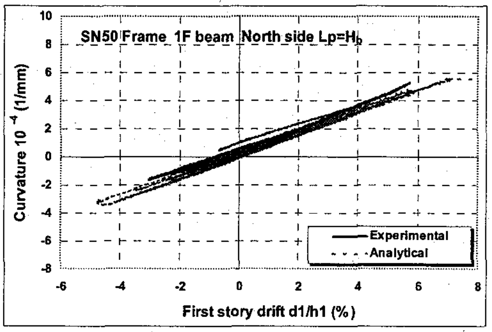


Figure 12: North side first floor beam curvature-story drift relationship SN50

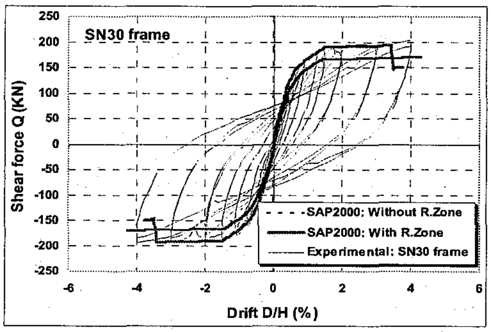


Figure 13: Pushover analysis for SN30

For the second floor beam this value was set to $0.5H_b$. Figure 12 shows a comparison between the analytical and the experimental results of the first floor beam of SN50 frame.

3.2 SAP2000 Pushover analysis

Pushover analysis using SAP2000 [11)] was carried out to predict the envelope curve of the shear force-drift relationship. Plastic hinges characteristics such as yielding moments; yielding rotation and initial stiffness were evaluated for each column and beam using the equations recommended by the Japanese design guidelines [6)].

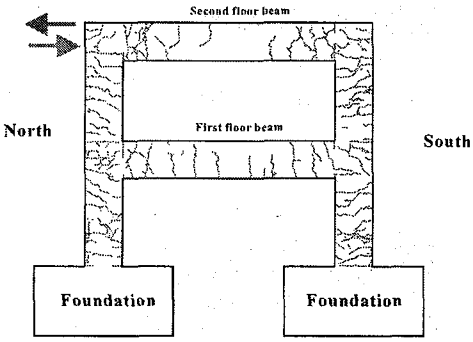


Figure 14: Observed damage for SN50

The analytical results, agreed with the experimental results quite well. As an example Figure 13 shows a comparison between the experimental and the analytical SAP200 results for SN30 frame. Inserting rigid zones at the edge of the columns and beams improved considerably the prediction.

4 OBSERVED DAMAGE

Figure 14 shows the crack pattern at 2% rotation 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.

Even though the spacing of the shear rebar was 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, the 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 as illustrated in Figure 15. Damage was also predicted analytically using Park et al. cumulative damage index [12)]. The damage model consists of a simple linear combination of normalized deformation and energy absorption. Figure 16 shows the damage progress at beams of frame SN50, as an example. According to Park et al's damage classification, first floor beams sustained a minor damage. Whereas, the second floor beams suffered a severe damage.

Figure 15: Damage to second floor beam of SN50

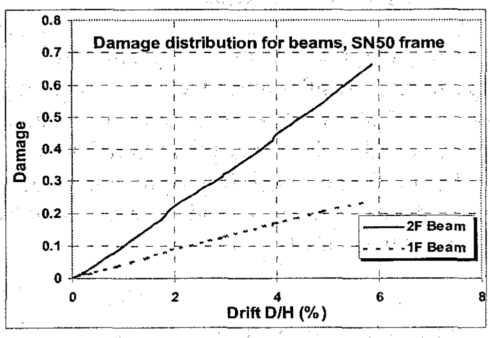


Figure 16: Computed damage for SN50 beams

The first story columns suffered no damage while; the second story columns suffered a minor damage. These classifications were consistent with the observed damage.

5 CONCLUSIONS

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

- Slight difference was found between the two frames in term of load-drift relationship either for stories or entire frame.
- No serious damage was observed for the first and second floor columns.
- Even though the stirrup's spacing at beams was six times the longitudinal bar diameter, buckling of longitudinal reinforcement was observed.
- Beam elongation affect the input moment to columns due to the increasing/decreasing in the $P-\delta$ effect and horizontal displacement.
- Behavior of the entire frame as well as its components, columns and beams, was well simulated using the nonlinear IDARC program.
- Pushover analysis predicted quite well the shear force-drift envelope curve of the cyclic loadings.
- Park et al. damage index reflect the observed damage for beams and columns.

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REFERENCES

1) H. Thomsen and W. Wallace, "Lateral load behavior of RC columns constructed using high-strength materials", ACI structural journal/September-October 1994 (in English).
2) Ang Beng Ghee, M. J. N. Priestley, and T Pauly, "Seismic shear strength of circular reinforced concrete columns", ACI structural journal/January-February 1989 (in English).
3) Norton J. A., King A. B., Bull D. K., Chapman H. E., McVerry G. H., Larkin T. J., Spring K. C., " Northridge Earthquake Reconnaissance Report", Bulletin of the New Zealand National Society for Earthquake Engineering Vol.27, No.4, 1994.
4) Bechtoula, H., Arai, Y., Kono, S., Watanabe, F., "Damage Assessment of RC Columns Under Large Axial and Lateral Loadings" Symposium on a design procedure of RC structures based on inelastic deformation -Use of confined concrete-, November 30, 2001, pp263-270 (in English).
5) Kono, S., Bechtoula, H., Kaku, T., Watanabe, F., "Damage assessment of RC columns subjected to axial load and bi-directional bending" Proceeding of the Japan concrete institute Vol.24, No2, 2002, PP235-240 (in Japanese).
6) Design guidelines for earthquake resistant reinforced concrete buildings based on inelastic concept, Architecture Institute of Japan, AIJ 1999 (in Japanese).
7) J. Matthews, D. Bull, and J. Mander, "Investigating the load paths of floor diaphragm forces during severe damaging earthquakes", Proc. of combined concrete society and ready mix, TR24, Rotorua, Oct.2001: 122-31.
8) Fenwick R.C and Megget L.M, "Elongation and load deflection characteristics of reinforced concrete members containing plastic hinges", March 1993, Bulletin of the New Zealand National Society for Earthquake Engineering, Vol. 26, No. 1, pp 28-41
9) Restrepo J.I, Park R and Buchanan A, "Seismic behavior of connections between precast concrete elements", 1993, Research Report 93-3, Department of Civil Engineering, University of Canterbury, Christchurch, New Zealand.
10) Valles, R., E., Reinhorn, A., M., Kunnath, S., K., Li, C., Madan, A., "IDARC2D Version 4.0: A computer program for the inelastic damage analysis of buildings" Technical report NCEER-96-0010 (in English).
11) Sap2000, Integrated Finite Elements Analysis and Design of Structures.
12) Young-Ji Park and Alfredo H.S. Ang, "Mechanistic seismic damage model for reinforced concrete", Journal of structural engineering, ASCE, vol. 111, No. 4, April 1985, pp. 722-739 (in English).