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STRENGTH EVALUATION OF REINFORCED CONCRETE BUILDINGS SUBJECTED TO SEISMIC MOTIONS

Jinhan Kwon
Ph.D. Candidate, The University of Texas at Austin, USA
jhkwon09@utexas.edu

Wassim M. Ghannoum
Assistant Professor, The University of Texas at Austin, USA
ghannoum@mail.utexas.edu

ABSTRACT: A four-story, full-scale reinforced concrete building was subjected on the National Research Institute for Earth Science and Disaster Prevention (NIED)/E-Defense shaking table to multi-directional ground motion records of increasing amplitude until a near collapse damage state. The test building had moment frames in one direction and rectangular shear walls in the other. The building was designed to the latest Japanese seismic standards and satisfied most U.S. seismic detailing requirements. Significant over-strengths were observed in the non-linear range of the building's behavior. Commonly prescribed strength evaluation methods were assessed in light of test results. Strain-rate effects were identified as a major source of over-strength and should be considered when estimating the seismic strength of concrete structures.

1. Introduction

A full-scale, four-story reinfroced concrete (RC) building was tested under multi-directional excitations of increasing amplitude on the National Research Institute for Earth Science and Disaster Prevention (NIED)/E-Defense shaking table in Japan ((Nagae et al., 2011a), (Nagae et al., 2011b), (Nagae et al., 2015)). The experimental program presents a unique opportunity to investigate the behavior of a complete building structural system subjected to seismic motions applied in all directions (including the vertical direction). Of particular interest was the lateral strength of the building. Strength estimates using provisions of the American Concrete Institute's (ACI) 318-14 building code (ACI Committee 318, 2014) were compared with the measured building strength. Test results showed significant over-strength from what was expected based on code strength methods. Strain-rate effects were found to play a major role in the observed over-strengths, as was also reported in prior rapid-loading and shaking table tests ((Ghannoum et al., 2012), (Ghannoum and Moehle, 2012a,b)). Test results indicate that improvements in code strength-estimation methods to account for loading-rate effects may be warranted. It is noteworthy that the strength provisions of ACI 318-14 produce similar structural strengths as the provisions of the Canadian CAN/CSA-A23.3-04 design code.

2. Test specimen details

Moment resisting frames were adopted in the longer direction of the test building (Frame Direction, Figure 1). Beams framing into columns in that direction were 300 mm wide by 600 mm deep. Rectangular shear walls coupled with frames were adopted in the shorter direction (Wall Direction). Walls were 2,500x250 mm in plan and connected to the corner columns through 300 mm by 300 mm beams. The slab thickness was 130 mm. The story height for all stories was 3 m.

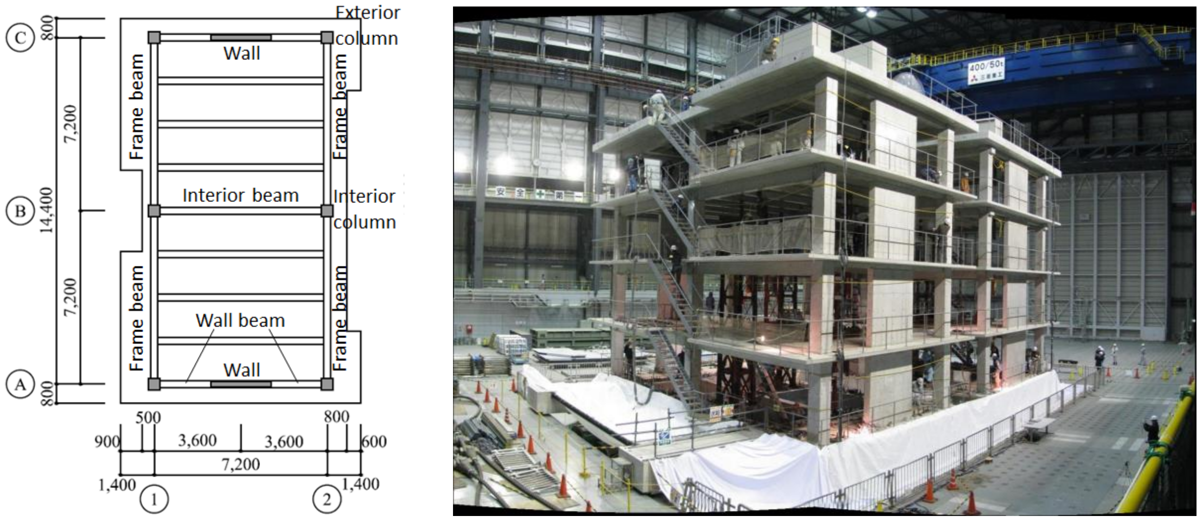


Figure 1 – Test building floor plan and picture on the E-Defense shaking table

The test building satisfied all Japanese (MLIT 2007, AIJ 2010) provisions for regions of high seismicity. The building was designed for an equivalent lateral force corresponding to 20% of the building weight. Columns and beams in the Frame Direction were reinforced with D22 longitudinal bars and D10 hoops satisfying most ACI 318-14 seismic provisions for Special Moment Resisting Frames (Nagae et al., 2015). Beams in the Wall Direction were reinforced with D19 bars and D10 hoops satisfying most ACI 318-14 seismic provisions for Special Moment Resisting Frames. The shear walls at axes A and C had the same amount of longitudinal reinforcement but using a different spacing of transverse reinforcement and boundary transverse reinforcement (Figure 2). Additional details about building reinforcement details and design can be found in Nagae et al. (2015).

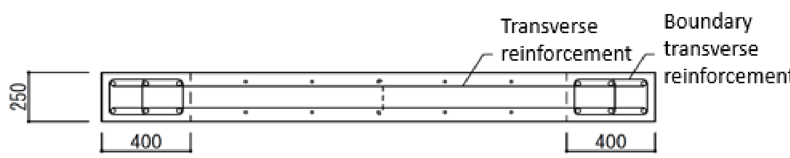
		1st story	Upper stories
Width x Depth		2,500 x 250	
Boundary longitudinal reinforcement		2 x 6-D19	
Transverse reinforcement	(A)	D10@125	D10@125
	(C)	D10@200	D10@200
Boundary transverse reinforcement	(A)	D10@80	D10@100
	(C)	D10@100	D10@100
Section			

Figure 2 – Dimensions and reinforcement details of shear walls; see (Nagae et al., 2015) for additional reinforcement details

The measured compressive strengths and moduli of elasticity of the concrete are presented in Table 1. Table 2 lists the measured yield and tensile strengths, as well as the measured modulus of elasticity of the reinforcing bars. The specified yield strength and the specified modulus of elasticity of the reinforcing bars were 345 N/mm^2 and 200 KN/mm^2, respectively. Gravity axial loads imparted on the columns were relatively low and ranged from 0.9% to 7.5% of column gross axial capacity. Wall axial loads were also relatively low and ranged from 0.2% to 1.0% of wall gross axial capacity. In all, 235 strain gauges were installed on column, beam, and wall reinforcing bars to measure bar strains.

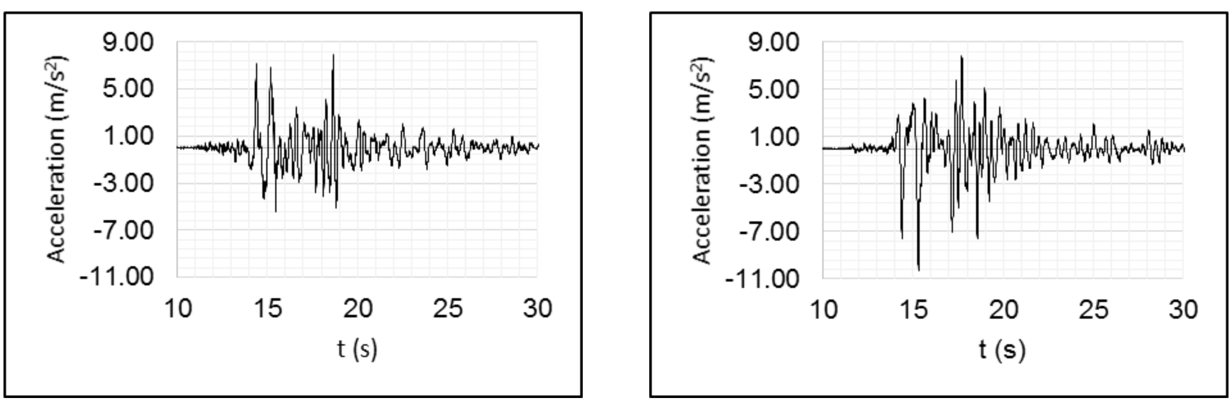
Table 1 – Material properties of concrete

	Measured compressive strength of concrete, f'_c (N/mm^2)	Measured modulus of elasticity of concrete, E_c (KN/mm^2)
4th story and roof	41.0	30.5
3rd story and 4th floor	30.2	30.3
2nd story and 3rd floor	39.2	32.8
1st story and 2nd floor	39.6	32.9

Table 2 – Material properties of reinforcement

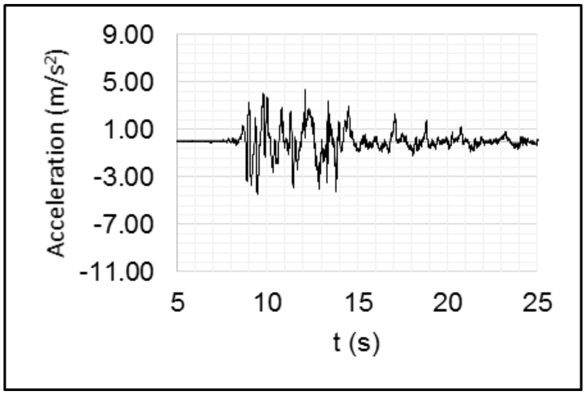
	Measured yield strengths of reinforcement, f'_y (N/mm^2)	Measured ultimate strengths of reinforcement, f'_t (N/mm^2)	Measured modulus of elasticity of reinforcement, E_c (KN/mm^2)
D10	388	513	191
D19	380	563	195
D22	370	555	209

The building was subjected to a series of ground motions of increasing amplitude until lateral story drift ratios exceeded 0.04 (near collapse damage state). All components of the JMA-Kobe (Figure 3) and the JR-Takatori motions (Figure 4) recorded during the Hyogoken-Nanbu earthquake were applied to the test structure. The structure was first subjected to the JMA Kobe motion scaled to 10%, followed by the same motion scaled to 25%, 50%, and 100%. The structure was then subjected of the Takatori motion scaled to 40% and 60%. The structure sustained limited flexural yielding during the JMA Kobe 50% motion and significant inelastic deformations and damage during the JMA Kobe 100% motion.

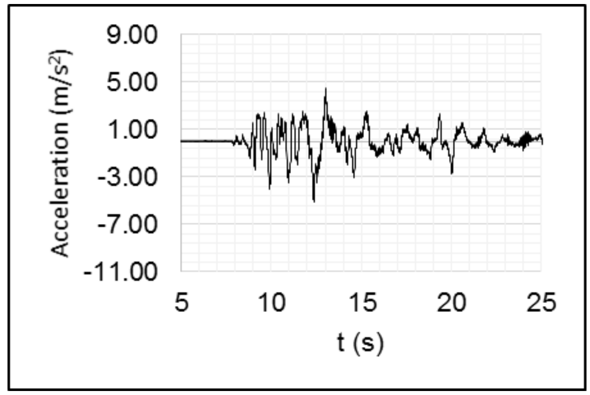


(a) Frame Direction (b) Wall Direction

Figure 3 – Acceleration histories of recorded JMA-Kobe 100% motions



(a) Frame Direction



(b) Wall Direction

Figure 4 – Acceleration histories of recorded JR-Takatori 60% motions

3. Strength Evaluation

A three-dimensional analytical model of the test building was constructed to evaluate the strength of the full structural system. Columns, beams, and walls were modeled as line elements with effective elastic stiffnesses. The effective stiffnesses and effective flange widths of the beams were calculated using the provisions of the standard Seismic Evaluation and Retrofit of Existing Buildings (ASCE/SEI 41-13). The effective stiffness of columns and beams was taken as 30% of the gross flexural stiffness. Wall effective flexural stiffness was taken as 50% of the gross stiffness. The effective flange widths of frame beams and wall beams were 2,380 mm and 1,240 mm, respectively. Rotational springs with bi-linear moment versus rotation relations were introduced at the ends of all elements to simulate flexural yielding of the members. The response of the springs was rigid-plastic capped at a member's flexural strength.

ACI 318-14 specifies that the probable moment strength of concrete members (M_p) be evaluated using a steel yield strength equal to 1.25 f_y (with f_y being the specified yield strength of the longitudinal reinforcing bars). This provision is intended to provide an estimate of ultimate moment strength accounting for the typically higher than specified yield strength of bars and strain hardening that may occur at large inelastic deformations. The reinforcing bars within the effective flange width were included when calculating the moment strength of beams. The probable moment strengths (M_p) of all members were used in the rotational springs to estimate the building lateral strength in accordance with ACI 318-14.

The peak strain rates applied to longitudinal bars in the columns, beams, and walls were obtained from strain gauge measurements for the JMA Kobe 100% motion (Table 3). As can be seen in Table 3, the strain rates in the boundary longitudinal bars were relatively high and highest in the walls. Reinforcing bars loaded at high strain rates can see significant increases in their yield and tensile strengths (Malvar, 1998). Bar strength increases are given by Malvar (1998) as a function of strain rates through dynamic increase factors (DIF) that are multiplied by the quasi-statically obtained yield or tensile strengths. Malver (1998) noted that the yield strength of bars increases more significantly than the tensile strength at the same strain rate. Table 3 presents the DIF for the yield (f_y) and tensile (f_u) strengths of longitudinal bars in various members in the building. Since a range of peak strain rates were extracted from strain data for each member type of the building, a range of DIF was obtained as seen in Table 3.

Table 3 – Properties for the calculation of nominal moments with strain-rate effects

	Peak Strain Rates (μstrain/s)	**DIF (f_y)**	**DIF (f_u)**
Shear walls	5.7×10^6 ~ 12.7×10^6	1.50 ~ 1.55	1.22 ~ 1.24
Exterior columns	7.8×10^3 ~ 1.8×10^5	1.18 ~ 1.35	1.05 ~ 1.09
Interior columns	1.1×10^3 ~ 1.1×10^6	1.20 ~ 1.56	1.03 ~ 1.14

Frame beams	$6.0x10^3$ ~ $1.0x10^6$	1.17 ~ 1.42	1.03 ~ 1.11
Wall beams	$1.1x10^4$ ~ $8.0x10^6$	1.20 ~ 1.52	1.09 ~ 1.23

An equivalent rectangular concrete stress block approach (Whitney, 1937) was used to calculate the nominal moment (M_n) and the probable moment (M_p) strengths of members in accordance with ACI 318-14 provisions. Moment strengths including strain rate effects ($M_{u,strain}$) were obtained using moment-curvature analyses (Figure 5). The elastic perfectly plastic relation that matched the maximum moment from the analyses was implemented in the rotational springs of the building model. Core concrete material properties were adjsuted in the moment curvature analyses to account for the confining effects of transvese reinfrocement in accordance with recommendation by Mander et. al., 1988. For the moment curvature analyses, Dynamic Increase Factors for the yield strength (DIF (f_y)) of 1.55 and 1.2 were adopted for walls and other members, respectively. For the ultimate strength of longitudinal reinforcement, DIF (f_u) of 1.24 and 1.09 were adopted for walls and other members, respectively. Table 4 lists the ratio of the moment strengths obtained including strain-rate effects ($M_{u,strain}$) to the probable moment strength (M_p).

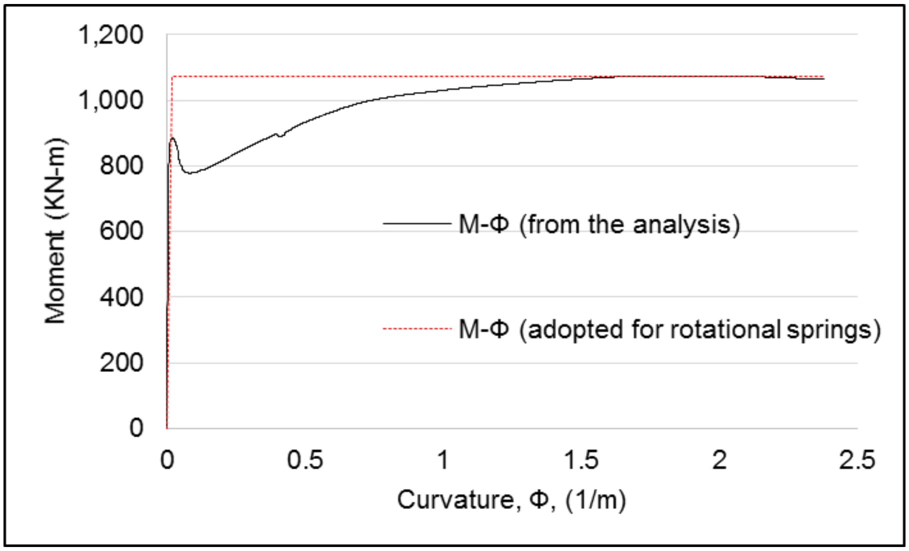


Figure 5 – Sample moment versus curvature relation for the second- floor frame beams

Table 4 – Ratio of moment strengths accounting for strain-rate effects to probable moment strengths evaluated using 1.25f_y

	$M_{u,strain}$ / M_p
Shear walls	1.39 ~ 1.40
Exterior columns	1.22 ~ 1.38
Interior columns	1.20 ~ 1.27
Frame beams	1.22 ~ 1.32
Wall beams	1.06 ~ 1.24

4. Strength comparison between analyses and experiments

Pushover analyses were conducted on the three dimensional analytical model described above. Two vertical distributions of lateral forces were used to push the building model. A vertical distribution of lateral forces defined by ASCE 7-10 (approximate inverted triangular distribution), as well as a uniform vertical distribution were used. Figure 6 compares experimental and analytical building overturning moments at the base versus roof drift ratio in the Wall Direction. The roof drift ratio is the lateral drift of the roof divided by the full height of the building measured from the top of the foundation. When nominal moment strengths (M_n) were used in the rotational springs, the maximum overturning moment was up to 49% lower than the peak experimental overturning moment. When probable moment strengths(M_p) were used in the rotational springs, the maximum overturning moment was 24% lower than the peak experimental overturning moment if the uniform vertical distribution of lateral loads was applied to the model. The maximum overturning moment corresponding to M_p was 29% lower if the inverted triangular vertical distribution of lateral loads was applied. When strain-rate effects were taken into account, however, maximum overturning moments derived analytically were less than 4% different from peak experiemtnal values for both pushover force distributions considered (Figure 6). Similar results were observed in the building Frame Direction.

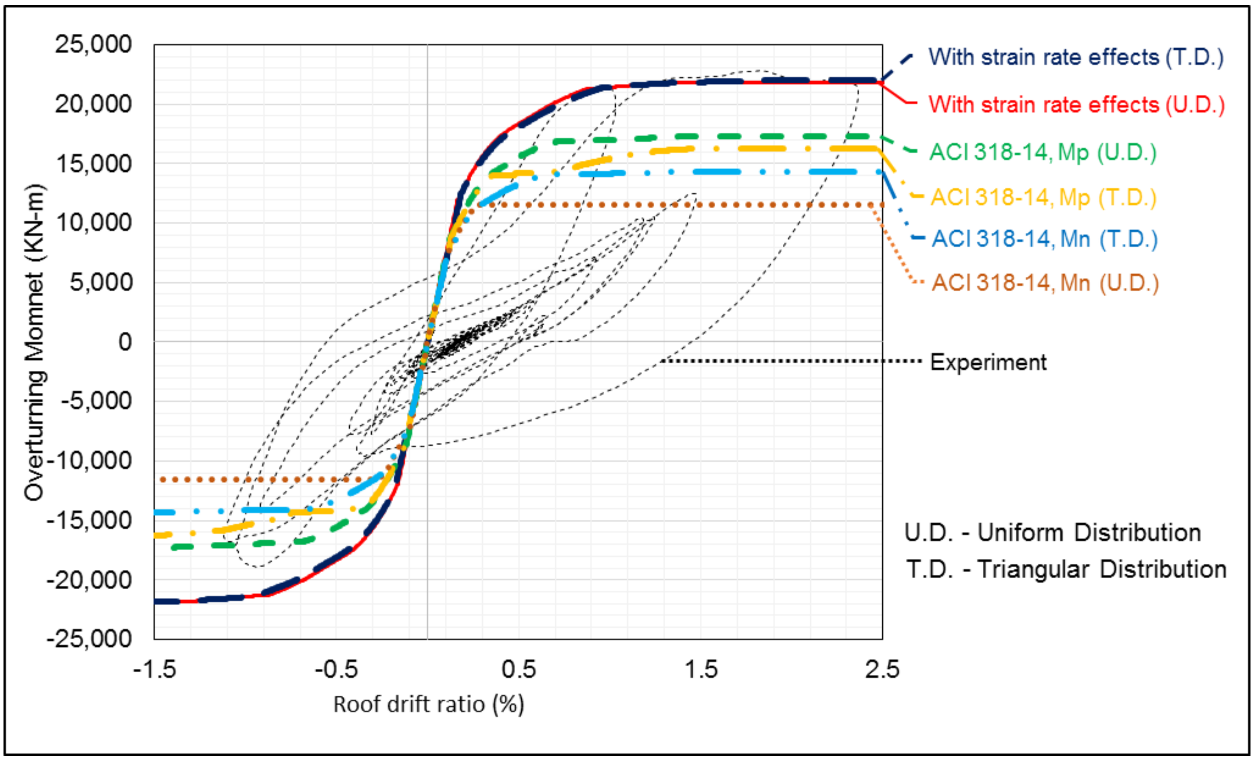


Figure 6 – Roof drift ratio (%) versus base overturning moment (KN-m) in the Wall Direction

5. Summary and Conclusions

A full-scale, four-story reinforced concrete building was tested under multi-directional seismic excitations of increasing amplitude on the NIED/E-Defense shaking table in Japan. The building was designed according to the latest Japanese seismic provisions and satisfied most of the U.S. seismic standards of practice. Experimental results showed significant over-strengths beyond the elastic range from what was expected, while accounting for strength increases in the longitudinal bars due to strain rates produced building overturning moment strength estimates within 4% of experimental values. The probable moment strengths evaluated using ACI 318-14 provisions were up to 40% lower than moment strengths evaluated including steel strength gains due to high strain rates. The discrepancy in moment strength was largest for shear walls where strain rates in the longitudinal bars were highest. Commonly used moment strength evaluation methods defined in ACI 318-14 may need to be adjusted to account for load-rate effects for reinforced concrete buildings subjected to seismic motions.

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