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Dynamic behavior of a shear wall : comparison between tests and analyses

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ABSTRACT

Presented in this paper are the results of a practical analytical study to assess the effectiveness of various industry standard finite element codes and simple approaches at predicting the elastic properties, dynamic properties, and inelastic seismic response of reinforced concrete (RC) shear walls. The study involves the comparison of experimental shake-table responses obtained by the Nuclear Power Engineering Corporation of Japan with results generated by inelastic push-over and dynamic analyses. Ultimate strength predictions based on ACI design criteria were also evaluated.

INTRODUCTION

Reinforced concrete shear walls are often the primary seismic-resisting elements of many safety related structures within nuclear power plants. Therefore, the accurate analytical representation of these shear wall elements is critical in the evaluation of element design loads, in the study of member ultimate capacity, as well as in the generation of in-structure response spectra necessary for equipment evaluation.

At present, a common and powerful analytical tool available for determination of shear wall seismic performance is the finite element method (FEM). Many different analysis approaches can be used to provide reasonable estimations of the elastic properties, the ultimate strength, and the inelastic response of RC shear walls. These analytical approaches range in sophistication from the relatively simple quasi-static, push-over analysis to the more complex time-history analysis. However, there has been only limited accumulation of accepted dynamic experimental response data to validate the accuracy of these various analytical approaches when shear wall response is dominated by high levels of material inelasticity.

The Nuclear Power Engineering Corporation (NUPEC) of Japan has conducted a series of well controlled dynamic response tests on large-scale, RC shear wall specimens [1]. These tests provide valuable experimental data on the dynamic behavior of these elements up to their ultimate state.

The intent of the analyses performed herein was to follow standard industrial practice for estimating the overall seismic behavior of the shear wall elements, rather than to study local effects. This was done in order to determine if the use of readily available methodologies will allow practicing engineers to accurately capture the dynamic response of this type of structural element. Results of this study indicate that simple methods of analysis can provide reasonable predictions of the overall inelastic behavior of a reinforced concrete shear wall. Also, the results show that the simplified techniques commonly used in standard industry practice can predict the initial elastic properties of this type of structural element.

TEST DESCRIPTION

The NUPEC experimental test program involved the dynamic testing of two identical reinforced concrete shear wall specimens. Dynamic testing of these walls involved subjecting each specimen to five sequentially increased excitation levels on a shake table. A single horizontal acceleration record was scaled at each level of excitation to obtain the dynamic response characteristics of the shear wall ranging from the elastic to the ultimate state. In the tests, the specimens were subjected to excitations in the horizontal direction along the axis of the shear wall. The maximum acceleration values for each input level are given in Table 1.

The two identical specimens were tested for the same excitations to ensure the reproducibility of the dynamic tests (U-1 and U-2). The specimens consisted of an "I" shaped wall system, as viewed in plan, with top and base slabs. Figure 1 shows the overall geometry and dimensions of the specimens. Table 2 summarizes the reported reinforcing steel ratios and properties.

Normal concrete with aggregate having a nominal maximum size of 10 mm was used for the web and flange walls. The results of the concrete material tests are summarized in Table 3. It should be noted that the concrete elastic modulus estimated using the ACI Equation 8.5.1 [2], $E_c = 57{,}000\ (f'_c)^{1/2}$ (psi), is 25,000 MPa, about 9% greater than the measured value.

A concrete block was added to the top slab of the specimens to simulate the gravity load on the shear wall. The total weight placed upon each specimen was approximately 1,197 KN. This weight is the primary source of inertial load acting on the wall.

A comparison between the dynamic responses measured during the two tests indicates that both specimens behaved in a nearly identical manner, implying repeatability of the testing. Based on these results, only the experimental responses of specimen U-1 were used in the comparisons with the analytical responses reported in this study.

ANALYTICAL PROGRAM

Two different inelastic analysis approaches were undertaken to estimate the seismic response of the RC shear walls, namely: (1) a quasi-static push-over analysis and (2) a dynamic time-history analysis. The analytical program discussed herein is summarized in Table 4.

Both types of analyses were performed on a two-dimensional mathematical representation of the specimen U-1. The actual geometric dimensions, as given in Figure 1, were used in the model, except the flange walls which were only partially modeled. An effective flange width (b_{eff}), equivalent to 24% of the flange wall length, was as computed using the formulation recommended by Paulay and Priestly [3]. This reduced flange width, was used to approximate the partial engagement of those flanges in the longitudinal response of the main shear wall. The material properties for the steel reinforcing bars and the concrete were approximated by the corresponding stress-strain curves shown in Figures 2(a) and 2(b), respectively. Material properties not reported by NUPEC were assumed as noted in Tables 2 and 3.

The ultimate shear capacity of the model was first calculated using the ACI design criteria [2]. The ACI ultimate capacity (equation 21-7) was computed to be 680 KN. The results of the finite element analyses are presented normalized respect to this ACI ultimate capacity.

For comparison purposes, the theoretical elastic shear stiffness was calculated using only the dimensions of the wall web and the uncracked concrete elastic properties (see Ghali and Naville equation 16-2 [4]).

Push-Over Analyses

The push-over analysis approach is a simple and commonly used method of estimating the ultimate capacity, stiffness at various load levels, and overall response before and beyond yield. This type of analysis is less computationally intensive and requires significantly less modeling effort than a truly non-linear, time-history analysis.

Three computer finite element codes were used to perform the push-over analyses: IDARC2D [5], FEM-I [6], and ADINA [7].

The first push-over analysis was performed with IDARC2D, a finite element code for the inelastic analysis of planar reinforced concrete frame systems. In this code, the entire shear wall is represented as a single two-dimensional shear element. Tri-linear moment-curvature (M-Φ) relationships define the inelastic behavior of the shear element and are generated internally by the program using a generalized fiber model implementing material non-linearity. Figure 3 shows the comparison between the experimental dynamic response and the IDARC2D push-over response for the various excitation levels. The IDARC2D results compare well with the experimental response within the elastic region. Also, the theoretical elastic shear stiffness compare with the initial experimental stiffness. At the load corresponding to the ACI ultimate capacity, IDARC2D indicates that inelastic behavior was initiated with the yielding of the flange reinforcement. At imposed levels of displacement beyond that of first yield, the IDARC2D model tended to overestimate the lateral stiffness of the wall as yielding spread into the cross-section of the element.

The second push-over analysis was performed with the finite element code FEM-I. This is a finite element code for the inelastic in-plane static analysis of reinforced concrete building components. The model of the wall contained 160 four-node plane-stress elements. A smeared model was assumed to represent the steel reinforcement with a bi-linear stress-strain relationship. The concrete was represented by a material model with combined features that included tension stiffening, a compression softening model, a strain softening, and a tensile cracking model [6]. Figure 4 shows the comparison between the experimental dynamic response and the FEM-I push-over response for the various excitation levels. The FEM-I results compare well with the experimental response within the elastic region. At approximately the load corresponding to the ACI ultimate capacity, FEM-I indicates that inelastic behavior was initiated with the yielding of the flange reinforcement. At imposed levels of displacement beyond that of first yield, the FEM-I the inelastic response parallels that measured experimentally, although the shear capacity is over-predicted by nearly 20%.

The third push-over analysis was performed with the finite element code ADINA using the internal nonlinear concrete model. The model of the wall contained 315 four-node plane-stress concrete elements and 294 inelastic truss elements to model every other pair of vertical and horizontal reinforcing bars. Figure 5 shows the comparison between the experimental dynamic response and the ADINA push-over response for the various excitation levels. As before, the ADINA results compare well with the experimental response within the elastic region and at the beginning of the beyond first yield region.

Figure 6 shows the comparison of the responses of the three push-over analyses and the experimental responses. This figure clearly indicates that all three models are able to accurately predict the occurrence of the initial yielding and the associated reduction of shear stiffness.

Figure 8 shows the results of a series of push-over analyses performed to demonstrate the effect on the inelastic response of considering different flange width contribution to the model.

Dynamic Analysis

A dynamic analysis was performed with program IDARC2D for the same model used in the push-over analysis to include the hysteretic behavior of the model. Figure 8 shows the comparison between the analytical and experimental results. The analytical model is in a close approximation with the test in the initial elastic range. As the degree of degradation increases, the analytical response become softer than the experimental response. This analysis was also used to determine the analytical frequencies and damping ratios at the initial condition and after the initial 3 tests. Table 5 shows the comparison between the analytical and experimental values.

CONCLUSIONS

The results of this limited study suggest the following:

(a) A simple static push-over analysis accurately estimate the dynamic shear response of a shear wall at the first yield and initial beyond first yield ranges, and could reasonably well estimate (over predicting) the shear response at larger degradation stages.
(b) Initial shear stiffness calculations give accurate values for linear dynamic analysis.
(c) ACI ultimate capacity prediction coincides with the elastic limit of a shear wall and is conservative respect to the actual ultimate capacity.
(d) Simple nonlinear dynamic analysis with program IDARC2D allows a quick and accurate calculation of the damping ratio of a shear wall structure at an specified excitation level for its use in equivalent linear analysis.
(e) The recommendation given in Reference 3 for flange reduction provides a good estimate of the contribution of connected orthogonal walls to the stiffness of a wall in its longitudinal direction.

Table 1: PEAK INPUT ACCELERATION

Case	Amax (g)
Run-1	0.054
Run-2	0.114
Run-2'	0.308
Run-3	0.357
Run 4	0.585
Run-5	1.247

Table 2: Reported and Assumed Steel Reinforcing Bar Material Properties

Web Horizontal Steel Ratio, $\rho_{w,h}$	0.0121
Web Vertical Steel Ratio, $\rho_{w,v}$	0.0126
Flange Horizontal Steel Ratio, $\rho_{f,h}$	0.0091
Flange Vertical Steel Ratio, $\rho_{f,v}$	0.0044
Reported Yield Strength, F_y	383.6 MPa
Reported Ultimate Strength, F_u	485.6 MPa
Reported Elastic Modulus, E_s	184428 MPa
Reported Elongation	29.1%
Assumed Strain Hardening Modulus, E_{sh}	3070 MPa
Assumed Strain at Initial Hardening, ε_{sh}	0.005

Table 3: Reported and Assumed Concrete Material Properties

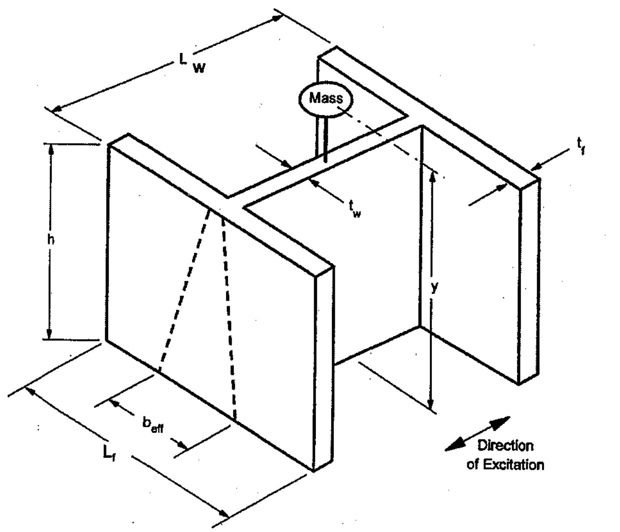
Measured Peak Compressive Strength, f'_c	28.65 MPa
Measured Uncracked Elastic Modulus, E_c	22955 MPa
Assumed Ultimate Compressive Strength, f'_u	22.91 MPa
Assumed Ultimate Tensile Stress, f_t	2.66 MPa
Assumed Strain at Peak Comp. Stress, ε_{cp}	0.002
Assumed Stain at Ultimate Comp. Stress, ε_{cu}	0.004

Table 4: Analytical Program

Analysis Method	Description of Finite Element Method Model		
Quasi-Static Push-Over	Lumped Shearing Model IDARC2D (1 Element)	2-D Plane Stress Model FEM-I (160 Elements)	2-D Solid Element Model ADINA (609 Elements)
Dynamic Time-History	Lumped Shearing Model IDARC2D (1 Element)	-	-

Table 5: Dynamic Properties of the IDARC2D Model

	Experiment Frequency (Hz)	Experiment Equivalent Damping Ratio (%)	IDARC2D Frequency (Hz)	IDARC2D Estimated Damping Ratio (%)
Before Run 1	13.2	1.1	13.09	1.2
After Run 2'	11.3	2.5	10.97	2.7
After Run 3	9.0	3.0	8.70	3.4
After Run 4	7.7	4.0		



General Wall Dimensions	SI
Overall Web Length, L_w	3,100 mm
Web Thickness, t_w	75 mm
Overall Flange Length, L_f	2,980 mm
Flange Thickness, t_f	100 mm
Wall Height, h	2,020 mm
Concrete Cover	18 mm
Wall Base to Mass Centroid Distance, y	2,400 mm

Figure 1: NUPEC Dynamic test Specimen Geometry

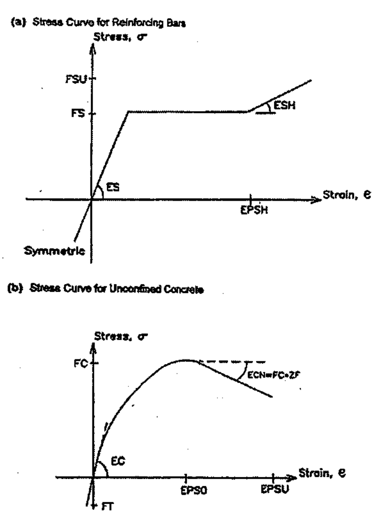


Figure 2: Material Stress-Strain Relationships

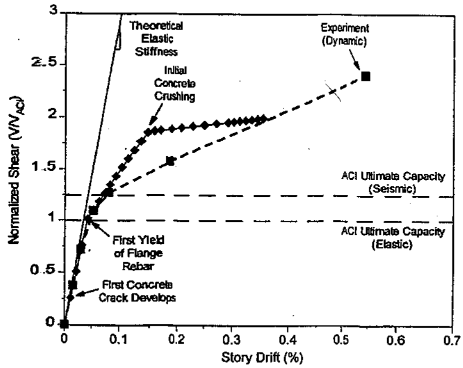


Figure 3: Comparison Between IDARC2D Quasi-Static Push-Over Model and Dynamic Experimental Response

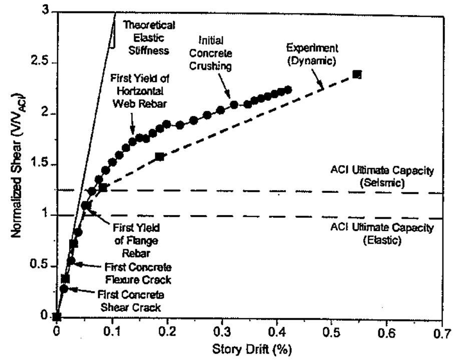


Figure 4: Comparison Between FEM-I Quasi-Static Push-Over Model and Dynamic Experimental Response

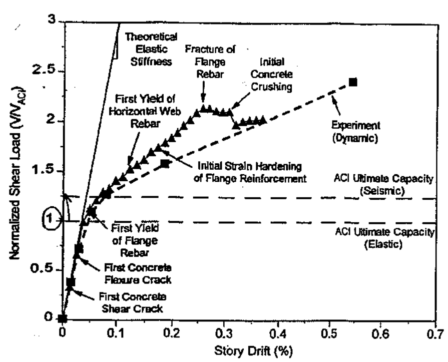


Figure 5: Comparison Between ADINA Quasi-Static Push-Over Model and Dynamic Experimental Response

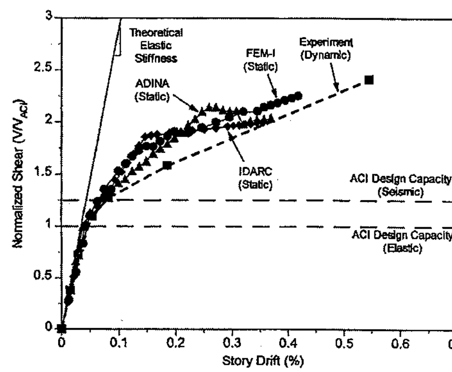


Figure 6: Comparison Between FEM Quasi-Static Push-Over Models and Dynamic Experimental Response

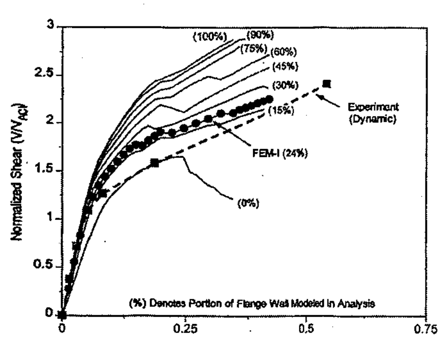


Figure 7: Comparison Between FEM-I Quasi-Static Push-Over Model with Various Flange Wall Lengths and Dynamic Experimental Response

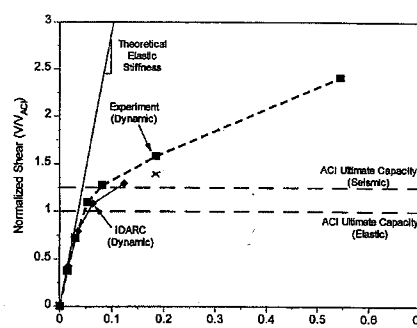


Figure 8: Comparison Between IDARC2D Time-History Model and Dynamic Experimental Response

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