Nonlinear Dynamic Analysis of Reinforced Concrete Planar Structures

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1. Introduction

The objective of this study was to develop a solution algorithm for the nonlinear dynamic analysis of reinforced concrete planar structures subject to general in-plane loadings. To assess the applicability of the solution algorithm, obtained response histories are compared with test results on a shear wall subject to an earthquake loading.

2. Solution Procedure for Dynamic Analysis

To represent the nonlinear behavior of a reinforced concrete effectively, simple but reliable hysteretic rules defining the cyclic stress-strain relations of concrete and steel are adopted in this study [1].

The governing equation for a dynamic analysis of a structure is given by

$$\mathbf{M}\ddot{\mathbf{U}} + \mathbf{C}\dot{\mathbf{U}} + \mathbf{KU} = \mathbf{P} \quad (1)$$

where **M**, **C**, and **K** are the mass, damping, and stiffness matrices, respectively; **P** is the external load vector; and $\mathbf{U}$, $\dot{\mathbf{U}}$, $\ddot{\mathbf{U}}$ are the displacement, velocity, and acceleration vectors, respectively. The damping matrix **C** is defined as a linear combination of the mass and stiffness matrices (Rayleigh damping).

If a nonlinear MDOF system is only subject to a ground acceleration $\ddot{u}_g(t)$, the external load vector is

$$\mathbf{P} = -\mathbf{M1}\ddot{u}_g(t) \quad (2)$$

where **1** is a vector of order N with each element equal to unity, and N is the number of total DOFs.

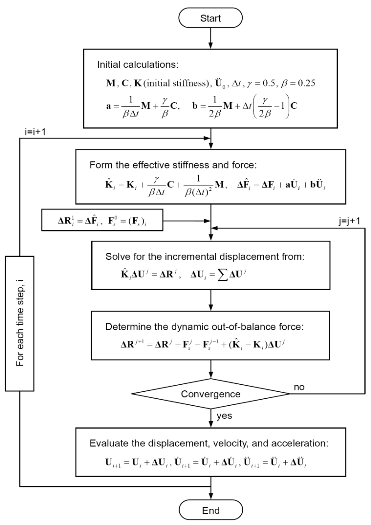


Figure 1. Outline of solution algorithm for nonlinear dynamic analysis

Nonlinear dynamic analysis is carried out with the modified Newton-Raphson iteration scheme in conjunction with the unconditionally stable Newmark's method. The algorithm based on Newmark's method for solving the equation of motion is given in the flow chart (see Figure 1). More details can be found elsewhere [2].

3. Numerical Application

The proposed material model and analysis procedure for simulating the dynamic behavior is demonstrated by their application to a three story reinforced concrete shear wall under a series of simulated earthquake motions tested by Hsu [3]. The total height of the specimen D-4 was 1473.2mm with 457.2mm as three equal story heights [3]. A depth of 457.2mm with a thickness of 25.4mm was constructed through out the section. The geometry and cross section of the selected specimen, designated as D-4, are presented in Figure 2. Number 8 wires (diameter=25.4mm) were used as reinforcement both vertically and horizontally. The material data for the shear wall is given in Table 1.

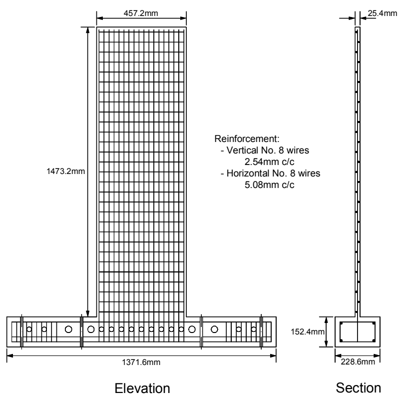


Figure 2. Dimensions and reinforcement details of specimen D-4

Table 1. Material properties of specimen D-4

Test structure	Concrete			Reinforcement			
	E_c	f_c'	f_t	E_s	f_y	ρ (%)	
						Vertical	Horizontal
D-4	26,200	32.5	2.4	200,000	369	8.0	4.0

The finite element idealization of the wall is also shown in Figure 3. The structure is modeled with 150 four-node elements, and nodes at the base of the bottom foundation are fully restrained against horizontal and vertical translations.

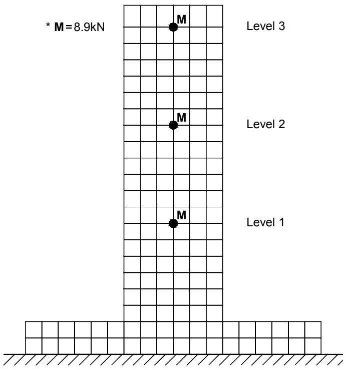


Figure 3. Finite element idealization of specimen D-4

Approximately 8.9kN added mass was installed at each floor level to simulate the dead load of a real structure. Steel transverse walls were also inserted to assure a proper bracing in the transverse direction. The base accelerations of the El Centro earthquake record (N-S component of the El Centro earthquake of May 18, 1940), with a maximum amplitude of 1.05g and time axis being compressed by a factor of five, were discretize at a constant interval, ΔT=0.004sec (see Figure 4), and were applied to the shear wall in horizontal direction parallel to its plane [3]. Typically a 5% damping ratio is recommended for reinforced concrete structures, and this value is adopted for the first and second modes to derive a Rayleigh damping matrix.

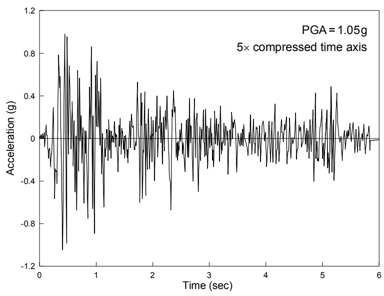
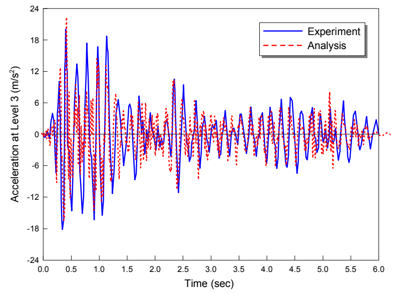
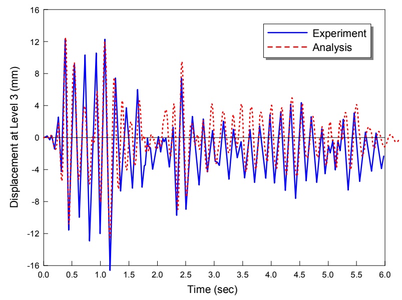


Figure 4. Base accelerations (N-S component, El Centro 1940)

As shown in Figure 5, the hysteretic responses of the shear wall at level 3 (see Figure 3) have been traced for 6 seconds of the earthquake. Flexural and diagonal shear cracks develop fully followed by yielding of longitudinal reinforcements at T=0.4sec. The computed damage is concentrated at the bottom of each floor, similar to the actual observed failure location. From a general observation that the displacement response history is very smooth at all levels, it is thought that the displacement hysteresis is dominated by the first mode component. It can be seen in Figure 5 that the analysis shows a fair agreement with the experiment in terms of a maximum response, post-peak vibration character and residual displacement of specimen D-4. Some discrepancy might be caused by ignoring the strain rate effect in the stress-strain relation of the concrete. The proposed model can be improved further by providing a strain rate option in the stress-strain curve.



(a) Acceleration at level 3



(b) Displacement at level 3

Figure 5. Response histories of specimen D-4

4. Conclusion

In this study, a dynamic solution algorithm was incorporated into the developed nonlinear finite element program for the analysis of reinforced concrete planar structures. The developed numerical algorithm was validated through a comparison of the obtained numerical results with experimental data of a orthogonally reinforced concrete shear wall.

ACKNOWLEDGEMENT

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