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SEISMIC RESPONSE AND SYSTEM IDENTIFICATION OF A ONE-TENTH SCALE MODEL OF A SEVEN-STORY RC FRAME-WALL BUILDING SUBJACTED TO MAJOR EARTHQUAKES

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ABSTRACT

A one-tenth scale "replica" model of the full-scale prototype seven-story RC frame-wall building tested at the Large-Size Structures Laboratory, BRI, Tsukuba, Japan, was designed and tested on the 4m×4m shaking table in Tongji University, Shanghai, China. An ultimate strength model with additional artificial mass was chosen to investigate the damage development and failure mechanism of such structures under major earthquakes, while the input seismic waves complied with that of the full-scale test structure. The degree of correlation between the experimental responses and those of both one-fifth scale model tested in Berkeley and the full-scale model tested in Japan is assessed. Based on the experimental results, a story-based simplified model, namely modified shear-bending beam model, is put forward to analytically predict overall seismic response of RC structures of a kind. The simplified model is then incorporated in a system identification procedure to determine characteristic parameters of the structure in inelastic range. The predominant factors which induce discrepancies between experimental and analytical results are therefore figured out. Efficiency of the modelling methodology in predicting seismic responses of such structures are verified.

KEYWORDS

Seismic response; system identification; RC frame-wall structures; one-tenth scale model; shaking table tests; modified shear-bending beam model; simplex method; inelastic range.

INTRODUCTION

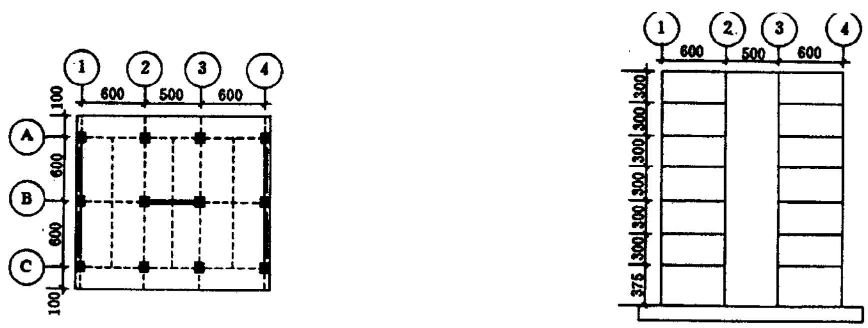
In the early nineteen eighties, a full-scale RC seven story frame-wall building was tested at Building Research Institute, Tsukuba, Japan, as part of U. S.-Japan coorperative Research Program Utilizing Large Scale Testing Facilities (Kaminosono et al., 1984). Besides, a series of supporting experimental investigations related to the seven-story building, such as earthquake simulator tests of reduced one-fifth scale "replica" model at the University of California, Berkeley (Aktan and Bertero, 1984), were conducted for a more comprehensive study on seismic behavior of such structures in order to improve the states of the art and practice of earthquake-resistant design and construction. On the basis of these previous research work, a reduced 1/10th-scale "replica" model of the 7-story prototype test structure was fabricated and tested on the

4m × 4m shaking table in Tongji University, Shanghai, China. The main objectives of this investigation were: ①to observe seismic behavior, damage/failure mechanism development and ultimate limit state responses of RC frame-wall structures under seismic excitations; ② to discuss the degree of correlation obtained between the experimental responses of the full-scale, 1/5th and 1/10th scale models; ③ to discern the predominant factors causing discrepancies between these experimental and analytical results.

EXPERIMENTAL INVESTIGATION

Design and Fabrication of 1/10th-Scale Model

The layout of the "replica" 1/10-scale model, which followed all the geometric similitude requirements of the prototype building, is shown in Fig. 1. The particular model scale of 1 to 10 chosen for the model test specimen was dictated by the capacities of the shaking table in Tongji University, Shanghai (Size of the table is 4m × 4m, its maximum bearing capacity is 150kN). An ultimate strength model with additional artificial mass was selected to enable the study of serviceability, ultimate limit state response characteristics and damage/failure mechanism of the conceptually designed and well-constructed prototype frame-wall structure, as the mass density similitude requirement in the directly true replica model was exceptionally difficult to satisfy employing common microconcrete (Zhang, 1994).



(a) Plan view

(b) Longitudinal frame B

Fig. 1 Test specimen

Geometric Similitude. This was rigorously satisfied except the floor slab thickness was 15mm instead of 12mm, considering feasibility of reinforcement detailing in the location of beam-column-slab joints and capacity of the slabs to support additional masses augmented by means of lead ballast weights on the floors.

Similitude in Materials. Microconcrete and galvanized iron wires were used to model the full-scale concrete and reinforcement in the 1/10th scale model. Mix proportions of microconcrete, mechanical properties of both microconcrete and galvanized iron wires were discussed in detail elsewhere (Zhang, 1994). It is noted that in the 1/10th scale model, the compressive strength and elastic modulus were lower by 20% and 30% than those of full-scale concrete, employing uniaxial compression loading on standard prisms and cubes (100mm × 100mm × 300mm and 70mm × 70mm × 70mm), respectively. Reinforcement of model components was determined in accordance with equivalent ultimate strength principle (Zhang, 1994).

Mass and Gravity Similitude. The mass density similitude requirement was satisfied by distributing uniformly lead ballast masses on each floor slab.

Loading Inputs Similitude. The same seismic waves as that of the full-scale structure, i. e. , modified

Miyagiken-Oki Record (NS, 1978), Taft Record (EW, 1952) and Takachioki Hachinohe Record (EW, 1968), were used as input excitations. But the time axis scales were compressed by $\sqrt{10}$.

Fabrication of 1/10th scale model. It was undertaken in National Disaster-Prevention Key Lab. of Civil Engineering. Similar procedures as the full scale structure were employed to construct the model.

Instrumentation and Test Program

In the loading direction of the model, accelerometers were placed on each floor to measure horizontal accelerations at floor levels; and perpendicular to the loading direction, accelerations of the first, fourth and seventh floor levels were also recorded. Deflections of both top of the model and the table itself were measured. Test program is shown in Table 1. Nine excitations were undertaken. White noises were inputed to diagnose the first and second vibrating frequencies both along the loading direction X and its perpendicular direction Y respectively before and after real seismic excitations.

Table 1. Test excitation program for 1/10th-scale model

Test No.	Input Waves	Code Name	Peak Amplitude(g)	
			X—direction	Y—direction
1	White Noise	W80	0.080	0.080
2	Miyagiken-Oki Record (NS, 1978)	MO90	0.091	
3	Miyagiken-Oki Record (NS, 1978)	MO107	0.107	
4	Miyagiken-Oki Record (NS, 1978)	MO240	0.240	
5	Taft Record (EW, 1952)	TF378	0.378	
6	Taft Record (EW, 1952)	TF90	0.091	
7	Takachioki Hachinohe Record (EW, 1968)	TH370	0.370	
8	Takachioki Hachinohe Record (EW, 1968)	TH90	0.091	
9	White Noise	W80	0.080	0.080

Experimental Results

Initial Dynamic Behavior. The first and second frequencies along the loading direction X-axis were 7.1Hz and 27.5Hz respectively. Comparing with the counterparts of the full scale structure being 7.05Hz and 25.1Hz respectively (after increasing the measured frequencies of the full scale structure by multiplying them by $\sqrt{S_l}=\sqrt{10}$, so that it could be compared with that of the 1/10th scale model), only slight differences by 0.67% and 9.44% were observed, indicating that the initial dynamic characteristics of the models were similar before the shaking table excitations were begun.

Seismic Responses of the Model. During 7 subsequent tests, the model was excited to progressively higher levels of damage until a complete flexural failure occurred at the base of the central wall. A brief summary of the test results is provided in Table 2 in which 4 significant selected tests, namely, 2MO90, 4MO240, 5TF378, 7TH370, are listed to represent typical responses of the model both in serviceability and ultimate limit state phases. Corresponding top-floor relative displacement time histories and the base shear-top floor relative displacement hysteresis relations are illustrated in Fig. 2, Fig. 3, separately.

Table 2. Main maximum responses of 1/10th-scale model

Test Signal	Max. Displ. (%H)	Max. Base Shear (%W)	Max. Top Acceleration (g)
2MO90	0.117	19.4	0.40
4MO240	0.224	30.4	0.68
5TF378	0.399	35.3	1.04
7TH370	1.200	42.1	1.09

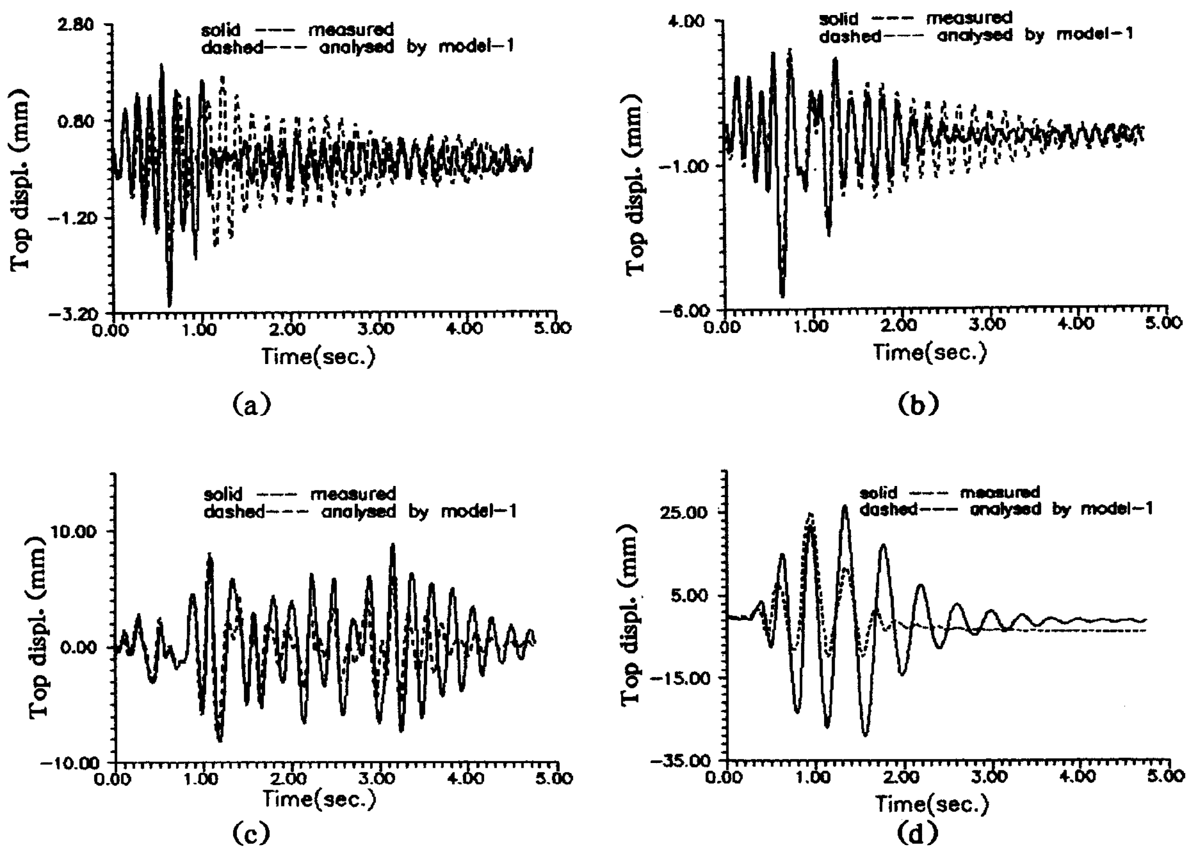
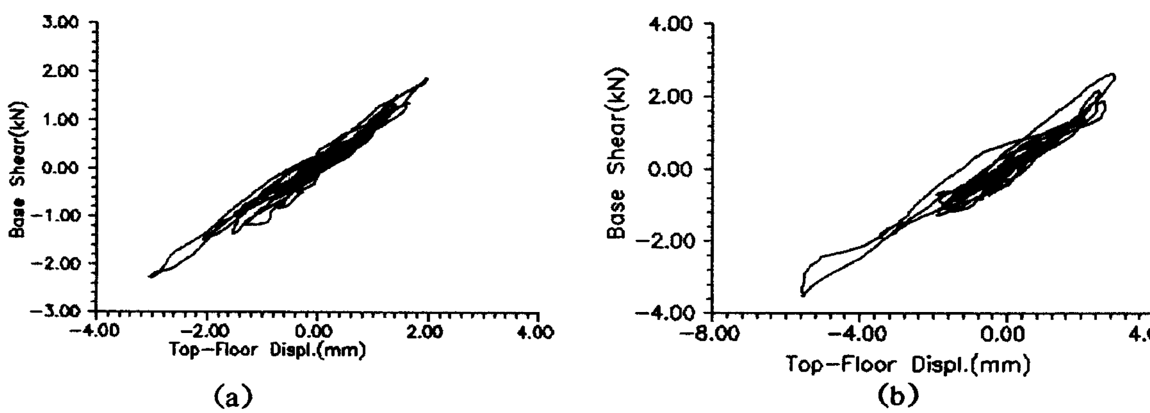
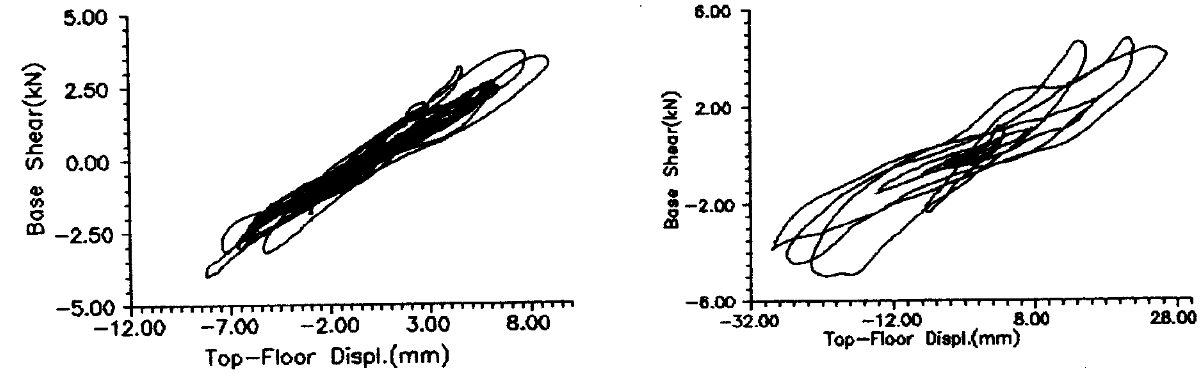


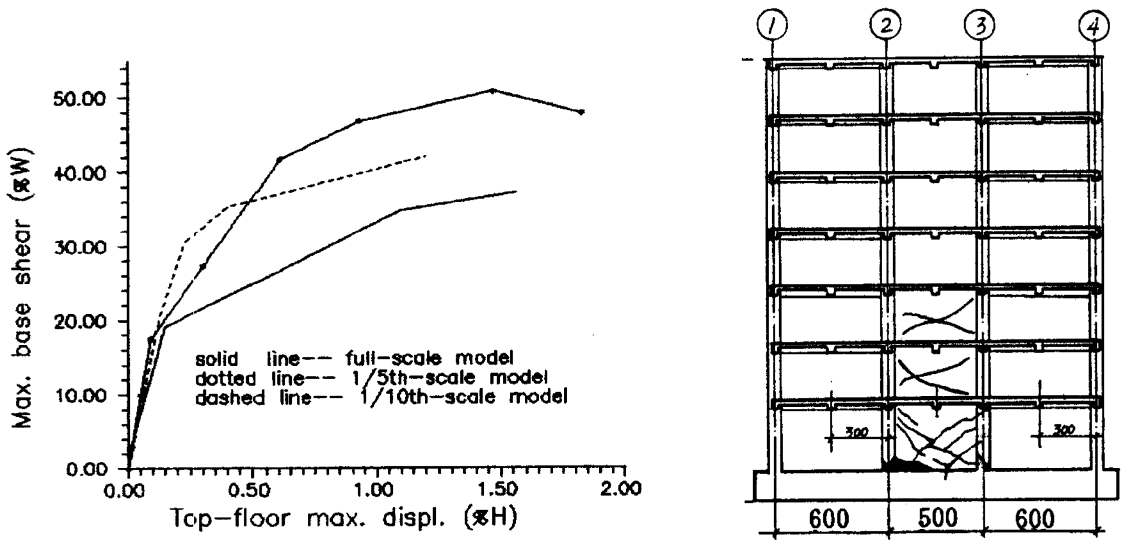
Fig. 2. Time histories—top floor relative displacements





(c) (d)

Fig. 3. Base shear—top floor relative displacement hysteresis relations



(a)Response Envelopes (b)Final Crack Pattern

Fig. 4. Response envelope and final crack pattern

Correlation Study of Experimental Results

Maximum Base Shear—Top Floor Relative Displacement Response Envelope. Comparison of the envelopes for the full-,1/5th-,and 1/10th-scale models in Fig. 4(a) leads to the following observations: ① The initial average stiffness characteristics of both the 1/10th- and full-scale models are more similar than that of 1/5th-scale model, despite the differences between the 1/10th- and full-scale model in the stress-strain constitutive relations of constituent model materials. ②The maximum base shear of the 1/10th-scale model when yielding occurred is 33% greater than that of the full-scale model. It may contribute mainly to over-reinforcement of model columns by 17%. ③The ultimate base shear of the 1/10th-scale model is 15.1% greater than that of the full-scale model when ultimate limit state is reached. It is believed that lower strain-hardening slope of model reinforcement induces such a discrepancy besides influences of size effect, loading rate and loading history, strain gradient and so forth(Zhang, 1994).

Comparison of Crack Patterns. The final crack pattern of the 1/10th-scale model is shown in Fig. 4(b). Path of damage development was much similar to that of both 1/5th- and full-scale models, but the final cracks distribution took much after the 1/5th-scale model, i. e. , cracks were less and more wide than that of the full-scale model, and concentrated in the lower wall panel and its edge columns. Like the 1/5th-scale model,a concentrated horizontal crack formed at the wall-foundation interface after the 5TF378 test. The crack extended completely to edge columns and wall panel,causing rocking of the central wall panel during the subsequent 7TH370 test. It is believed that the difference in crack patterns was caused by the differences in :①concrete tensile strength,② average strain gradient,and③ bonding characteristics between model concrete and reinforcement.

ANALYTICAL RESULTS

In modelling RC frame-wall buildings for inelastic seismic response analysis,the so called"Shear-bending beam model"has been used extensively(Sheng and Zhang,1981),because of its efficiency in predicting globe responses of RC structures and its economy both in computation time and memory. On the basis of previous studies,a refined and more reliable model,namely modified shear-bending beam model,was formulated for regular RC frame-wall structures(Zhang,1994). In this model,an original three dimensional frame-wall structures was replaced by planar integrated walls and integrated frames along its principle axes,which were simplified as equivalent shear-bending beam element and shear-beam element in parallel respectively. The floor system was assumed to be infinitely regid in its own plane,therefore transverse hinged infinite-axial-stiffness links were introduced to connect walls with frames to account for their interaction under earthquakes. Masses of the structure were lumped at each floor level.

The hysteretic model of the equivalent shear-beam element was the Takeda model,representing story shear —story relative displacement relation. For shear-bending beam element,modified Takeda model accounting for pinching behavior was employed to describe inelastic behavior of sectional moment(M)-curvature(ϕ) relation at the two ends of the element in which concentrated inelastic springs locate. Two significant modifications in the modelling methodology comparing with previous studies are:(1) Effect of shear stiffness degradation on seismic response of the integrated walls is approximately accounted for,except that influence of varying axial stresses on both stiffness and yielding strength is considered. (2)Systematic methods for pragmatic evaluation of characteristic parameters of the above two hysteretic models are proposed in terms of monolithic and cyclic loading experimental results(Zhang,1994).

Analytical results of the full-scale 7-story structure using the modified shear-bending beam model,which is abbreviated as model-1. when inputs are 2MO90,4MO240,5TF378 and 7TH370 are shown in dashed lines in Fig. 2,respectively. Other results were shown and discussed in detail elsewhere(Zhang,1994).

It is recognized from the above analytical results that:①Inelastic seismic response analysis incorporating the proposed modified shear-bending beam model is of acceptable accuracy in predicting globle maximum responses of RC frame-wall structures subjected to seismic excitations;②Time histories of globle responses can also be approximately simulated in some degree both in serviceability and ultimate limit state ranges;③ Employing the proposed model in simplified seismic response analysis can reduce both computation time and occupied memory in orders of magnitude,paving the way for its practical application in seismic design.

SYSTEM IDENTIFICATION

In order to gain insight into the seismic responses of a RC frame-wall structure in inelastic range to detect the factors causing discrepancies between experimental and analytical results,a system identification procedure was ultilized to appraise characteristic parameters as its inelastic mathematical model was formulated qualitatively and statically quantitatively. Measured data of two runs of shaking table tests,namely 2MO90 and 5TF378, while the test model was believed typically in elastic and inelastic ranges respectively, were employed for characteristic parameters identification for hysteretic models (Zhang, 1994). The latter case are discussed here in detail.

Identification Procedure

Formulation of Inelastic Mathematical Model. The analytical model of the test structure is simplified as the aforementioned modified shear-bending beam model, in which the hysteretic model for the shear-bending beam element is represented by modified Takeda model and that for the equivalent shear-beam element is the Takeda model. Characteristic parameters of these hysteretic models are tentatively evaluated.

Criterion Function. The criterion function used here is an integral mean squared error function which includes errors in time histories of accelerations acquired both experimentally and analytically. Assuming that the initial vector of parameters to be identified is $\{\beta\}^{(0)}$, while $\ddot{X}(\beta, t)$, $\ddot{Y}(t)$ are response qualities both calculated from the model using $\{\beta\}^{(0)}$ and measured from the test to the same excitation, then the error function can be expressed as

$$J(\beta, T) = \frac{1}{T}\int_0^T (\ddot{X}(\beta, t) - \ddot{Y}(t))^2 dt, \tag{1}$$

in which T is an interval of integration considered. The simplex method is chosen to adjust systematically the vector of parameters $\{\beta\}^{(i)}$ until the value of error function is minimized.

Establishment of Identified Parameters. Nine parameters are chosen as the predominant indentified factors: ①The second stiffness (after cracking) D_2, the third stiffness D_3 (after yielding), story cracking and yielding relative displacement δ_{cr}, δ_y of the shear-beam element representing the integrated frames; ②The second stiffness EI_2, the third stiffness EI_3, cracking and yielding curvatures ϕ_{cr}, ϕ_y of the shear-bending beam element; ③Damping ratio ξ. As the above-mentioned parameters can be tentatively evaluated by static loading experiments, the identified parameter β_i can be described as ratio of its real value Q_r to the tentative value Q_t, which is expressed as

$$\beta_i = \frac{\text{real value}, Q_r}{\text{tentative value}, Q_t}, \qquad (i = 1, \cdots\cdots, 9), \tag{2}$$

thus the vecter of identified parameters can written as

$$\{\beta\} = \{\beta_1, \beta_2, \beta_3, \beta_4, \beta_5, \beta_6, \beta_7, \beta_8, \beta_9\}^T, \tag{3}$$

Evaluation of Initial values. On the basis of the tentative values of the analytical model, a process of trial and error is used to determine initial identified values. The initial vector of parameters $\{\beta\}^{(0)}$ thus is expressed as

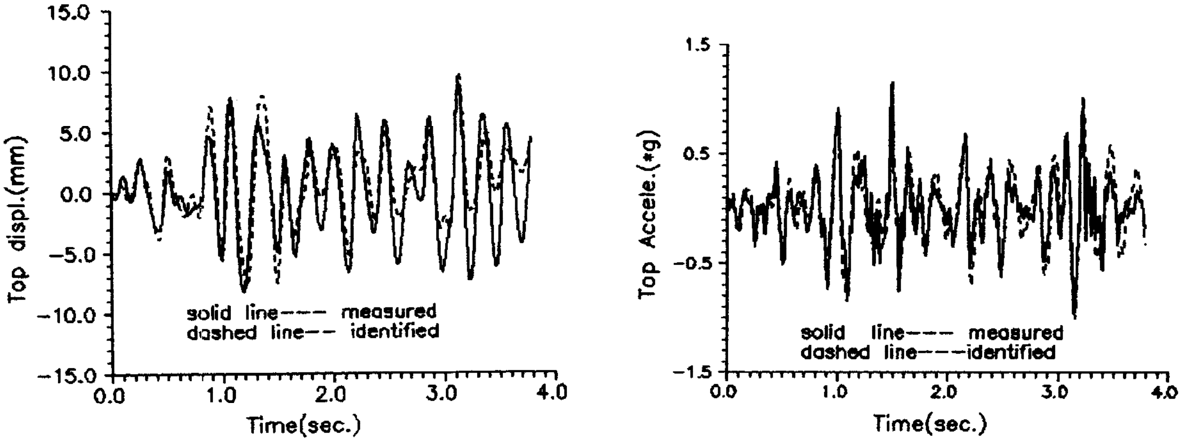
$$\{\beta\}^{(0)} = \{0.8, 0.8, 0.8, 0.8, 0.8, 0.8, 0.8, 0.8, 0.5\}^T. \tag{4}$$

Identification Results

The optimal set of parameters after minimizing the error function utilizing simplex method is:

$$\{\beta\} = \{0.65, 0.65, 0.85, 0.65, 0.85, 0.65, 0.85, 0.85, 0.55\}^T. \tag{5}$$

The identified time histories of the top-floor displacement and acceleration are shown in Fig. 5.



(a)Top-floor relative displ..　　　　(b)Top-floor acceleration.

Fig. 5. Identified results

CONCLUSIONS

The study in this paper concludes: ①The one-tenth scale model was successful in predicting overall seismic responses of the prototype test structure; ②Results of the identification procedure show that the proposed modified shear-bending beam model is efficient and reliable in predicting overall seismic responses of RC frame-wall structures and is readily used in seismic design; ③Among various factors affecting inelastic seismic response of RC frame-wall structures, stiffness degradation and its hysteretic rules are the most predominant, while the others are of less dominance.

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