COMPARISON BETWEEN REAL TIME NONLINEAR SEISMIC HYBRID AND SHAKE TABLE TESTING TECHNIQUES

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ABSTRACT :

A direct comparison is made between shake table and real time hybrid simulation testing techniques for a two-storey steel building structure model. Only the first-storey structural components were included in the hybrid test program and a Rosenbrock-W explicit integration scheme was adopted for the numerical solution. The tests were performed under seismic ground motions exhibiting various amplitude levels and frequency contents to develop first and second mode dominated responses as well as elastic and inelastic responses. Excellent correlation was obtained between the two testing techniques, indicating that real time hybrid testing method can be used to successfully reproduce both the linear and nonlinear seismic responses of ductile structural steel seismic force resisting systems.

KEYWORDS: Integration scheme, Real time hybrid testing, Rosenbrock, Shake table testing

1. INTRODUCTION

As real time hybrid testing becomes more popular among the earthquake engineering community, it is of utmost importance to evaluate the quality of the results obtained from this experimental tool using benchmark testing. The structural engineering testing laboratory of École Polytechnique de Montréal houses a high performance earthquake simulator (shake table) facility which has been in use since 1995. The researchers recently acquired the capability and the technology to perform real time hybrid dynamic testing.

In this paper, the results obtained from shake table tests carried out on the two-storey half-scale building structure are compared to real time hybrid testing results. In the hybrid tests, one storey of the structure is tested in the laboratory whilst the remainder is modelled numerically. Time integration is performed using a Rosenbrock-W based methodology. Inelastic response occurs in the form of plastic hinging at the column bases. The shake table test program involved several experiments using different excitation signals including the 1940 El Centro Imperial Valley earthquake record, a high frequency motion typical of Eastern North America (ENA), which occurred during the 1988 Saguenay earthquake, and harmonic signals. The amplitude of the excitations was varied such that both the linear and nonlinear structure responses could be investigated. In the cases where the shake table test structure exhibited a nonlinear response at both storeys, the numerical model used in the hybrid testing also included nonlinear modelling capabilities. This project represents a unique opportunity to compare the two testing techniques as the two test programs were conducted in the same laboratory environment and involved the same physical structural components.

2. SHAKE TABLE TEST PROGRAM

2.1. Test set-up

The test structure used in this study is illustrated in Figure 1. It represented a half-scale model of a two-storey steel building. At each level, the structural system consisted of a single cantilevered column rigidly anchored at its base and pin-connected at its upper end. A simplified finite element model of the structure including the masses, the columns, and the struts is presented in Figure 1a. The natural periods of vibration are also given in Figure 1a. The total height of the building was 3.0 m, i.e. 1.5 m per storey. A 3D scale representation of the test set-up is presented in Figure 1b, including the shake table, the frame supporting the seismic weight, and the strong floor of the laboratory. The columns were made of ASTM A992 W100x19 steel shapes bent about their strong axis ($A = 2470\ mm^2$, $I_x = 4.76 \times 10^6\ mm^4$, $Z_x = 103 \times 10^3\ mm^3$, $F_y = 447$ MPa, $F_u = 581$ MPa). All columns used in the test program came from the same fabrication lot and had essentially identical properties. Columns were anchored with heavy base steel plates carefully designed using a 3D nonlinear finite element model to ensure a nearly perfectly fixed end condition. The bottom storey column was anchored to the shake table, whereas the top column was anchored to the first storey mass. Each column was replaced after strong ground shaking producing significant yielding. The masses at levels 1 and 2 were equal to 7250 kg and 6500 kg, respectively. Each mass was made of two reinforced concrete blocks that slid horizontally on low friction rollers supported by the steel frame shown in Figure 1b. Figure 2a shows the entire shake table test set-up. The masses were connected to their corresponding columns by use of a pinned-pinned stiff steel strut. A load cell was mounted on each strut to measure the inter-storey shear at each level. The weight of the concrete masses was transferred to the laboratory strong floor, outside of the shake table, by an independent two-storey braced steel frame. Therefore, no P-delta effects were induced in the columns which acted as simple cantilevers.

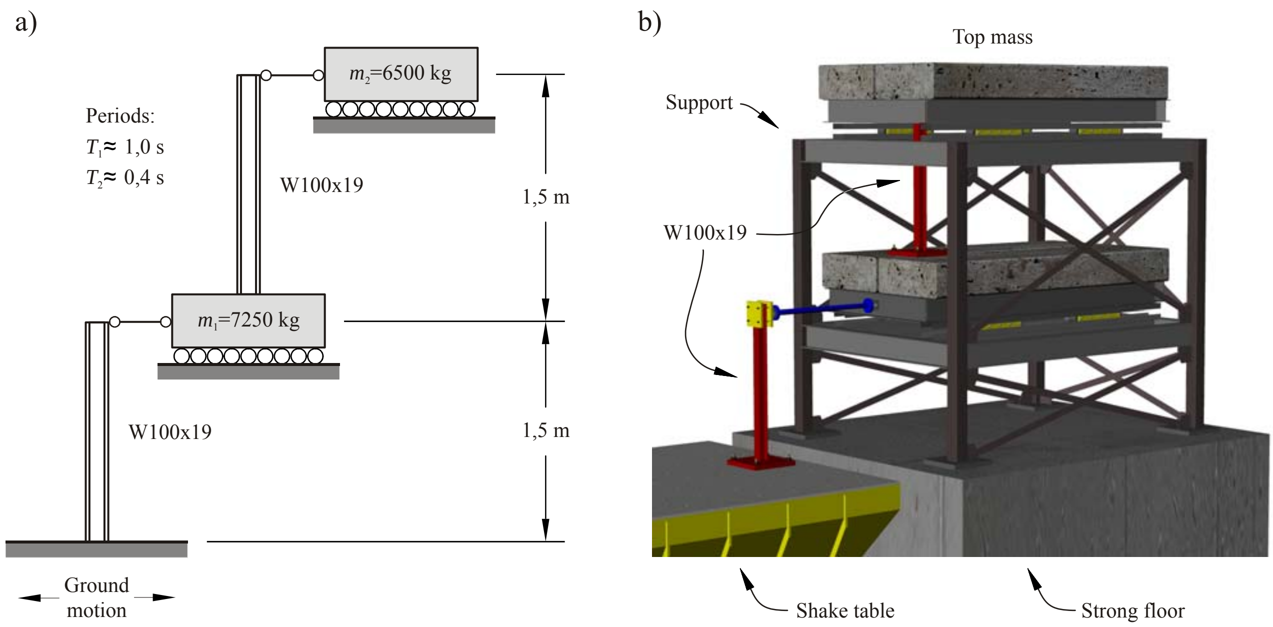

Figure 1 Two-storey building model: a) Schematic elevation of the model; b) 3D representation of the model.

2.2. Shake table test program

Three different tests were carried out on the building model using two different excitation signals. The first test was performed under the 1988 Saguenay earthquake (Chicoutimi Nord, Site 16, N124). This signal has relatively high frequency content and small displacements, which is typical for earthquakes expected in ENA. This ground motion record aimed at exciting the second mode of the structure in the linear regime. The two subsequent tests were performed under a 1940 Imperial Valley earthquake record (El Centro, S00E) selected to excite the structure in its fundamental mode of vibration. In the first of these two tests, the El Centro record was scaled to excite the structure in the linear regime. In the last test, the El Centro record was scaled to induce large inelastic excursions at the bottom storey. Details of the three tests are given in Table 2.2.1.

Those three tests were repeated to verify reproducibility and examine the influence of other effects. For instance, in some of the tests, a slender diagonal tension-only bracing member made of a 19.1 mm x 3.2 mm

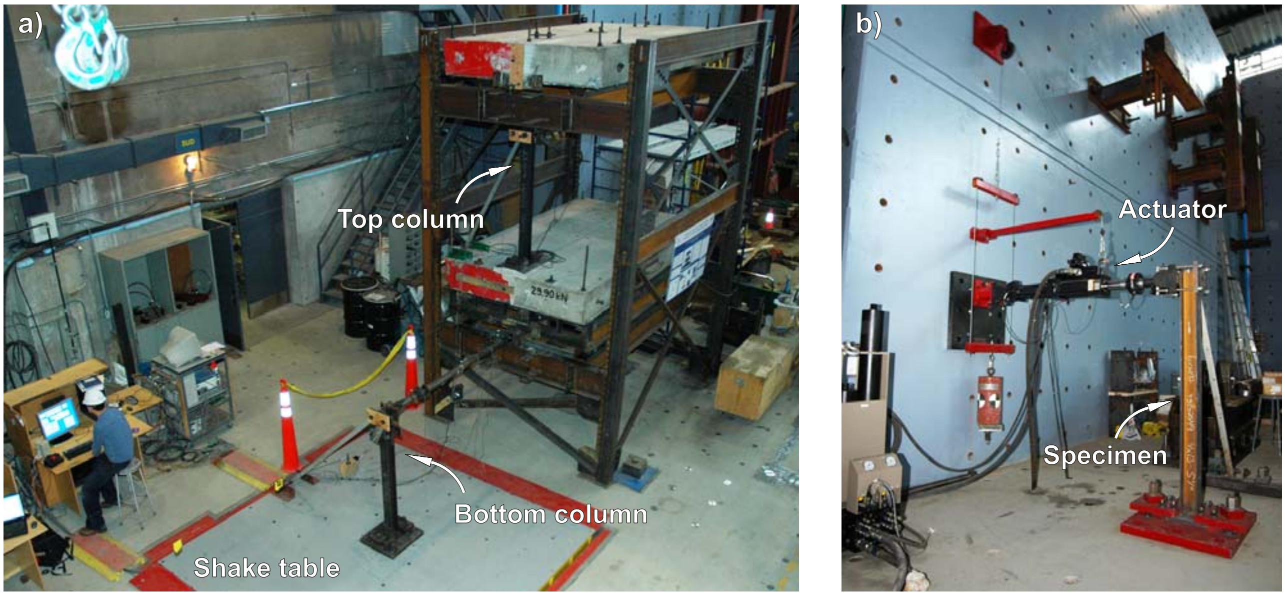

Figure 2 Test set-up for: a) Shake table tests (full structure); b) Hybrid tests (first-storey column only).

Table 2.2.1 Test program used for shake table testing and hybrid testing.

Test	Type	Record	Scale factor	PGA (g)	Frequency content
#1	Linear	1988 Chicoutimi	20.0	2.62	High
#2	Linear	1940 El Centro	0.5	0.174	Low
#3	Nonlinear	1940 El Centro	1.0	0.348	Low

steel bar was added at the first storey. The brace buckled at a very low load level and, hence, was only acting when stretched in tension. This led to an unsymmetrical system with highly variable stiffness during the ground motions, depending whether the brace was active or not during the applied ground motions. In this paper, only the tests with the bare columns are presented and discussed.

3. HYBRID TESTING

3.1. Test set-up

Figure 2b shows the physical model considered in the hybrid test program. The physical structure only included the first-storey column whilst the remainder of the building was modeled numerically. The column was anchored to the strong floor of the laboratory and horizontal displacements were applied through a high performance hydraulic actuator. A counter weight system was used to maintain the actuator in the horizontal position during the tests, avoiding axial loads to be induced in the column. As shown in Figure 2b, this counter weight device included a steel cable, a strut, and two pulleys. The hydraulic actuator used had a 100 kN capacity and a ±127 mm dynamic stroke. The 227 l/min two-stage servo-valve of the actuator was driven using a real time MTS® structural PID control system connected to a real time PC via Scramnet® shared memory. The integration scheme was implemented using MathWorks's Simulink® and MathWorks's XPC target®. An average group delay of 18 milliseconds was determined from the transfer function of the actuator-control system. Delay between the command and feedback in displacement of the actuator was compensated by use of a 3rd order FIR filter (Finite Impulse Response) that was specifically developed for this application to obtain constant group delay compensation.

3.2. Integration scheme

The numerical integration scheme chosen to perform the hybrid tests is a recently developed two-stage Rosenbrock-W variant for real time dynamic substructuring and pseudo-dynamic testing (Lamarche et al., 2008a). In state space formulation, the first time derivative of the state vector $\mathbf{y}_k = [\mathbf{u}_k \ \dot{\mathbf{u}}_k]^{\mathrm{T}}$ at pseudo time k is given by:

$$\mathbf{f}_k = \dot{\mathbf{y}}_k = \begin{bmatrix} \dot{\mathbf{u}}_k \\ \mathbf{M}^{-1}(\mathbf{p}_k - \mathbf{r}_{\mathrm{n}}(\mathbf{u}_k, \dot{\mathbf{u}}_k) - \mathbf{r}_{\mathrm{e}}(\mathbf{u}_k, \dot{\mathbf{u}}_k)) \end{bmatrix} \tag{3.2.1}$$

where $\mathbf{r}_{\mathrm{n}}(\mathbf{u}_k, \dot{\mathbf{u}}_k)$ and $\mathbf{r}_{\mathrm{e}}(\mathbf{u}_k, \dot{\mathbf{u}}_k)$ are respectively the numerical and experimental restoring force vectors, $\mathbf{p}_k$ is the external force vector, and $\mathbf{u}_k$ and $\dot{\mathbf{u}}_k$ are respectively the displacement and velocity vectors. Before initiating the numerical integration process, the method requires an initial estimate of the Jacobian matrix **W:**

$$\mathbf{W} \approx \left.\frac{\partial \mathbf{f}}{\partial \mathbf{y}}\right|_{t=0} = \begin{bmatrix} 0 & 1 \\ \mathbf{M}^{-1}\mathbf{K}_0 & \mathbf{M}^{-1}\mathbf{C}_0 \end{bmatrix} \tag{3.2.2}$$

where $\mathbf{M}$ is the mass matrix, and $\mathbf{K}_0$ and $\mathbf{C}_0$ are the initial stiffness and initial damping matrix estimates, respectively. By making the substitution:

$$\mathbf{X} = [\mathbf{I} - \Delta t \gamma \mathbf{W}]^{-1} \tag{3.2.3}$$

where γ is a parameter of the method, the first stage leading to the displacements to be imposed to the test specimens at time $\mathrm{t} + \Delta \mathrm{t} / 2$ are computed from:

$$\mathbf{k}_1 = \mathbf{X}\mathbf{f}_k \tag{3.2.4}$$

$$\mathbf{y}_{k+1/2} = \mathbf{y}_k + \frac{\Delta t}{2}\mathbf{k}_1 \tag{3.2.5}$$

Once the updated displacements are applied to the test specimen, the experimental restoring force vector $\mathbf{r}_{\mathrm{e}}(\mathbf{u}_k, \dot{\mathbf{u}}_k)$ is fed back to assemble the vector $\mathbf{f}_{k+1/2}$. Similarly, the second stage that leads to the displacements at $t + \Delta t$ are computed from:

$$\mathbf{k}_2 = \mathbf{X}(\mathbf{f}_{k+1/2} - \Delta t \gamma \mathbf{W}\mathbf{k}_1) \tag{3.2.6}$$

$$\mathbf{y}_{k+1} = \mathbf{y}_k + \Delta t \mathbf{k}_2 \tag{3.2.7}$$

The scheme can also be formulated to solve the 2^{nd} order differential equation in its classical format. This alternative formulation is presented by Lamarche et al. (2008a), together with a detailed procedure for its implementation. In the hybrid tests performed during the course of this project, $\gamma = 1/2$ was used. With $\gamma = 1/2$, the method is proven to be non dissipative and unconditionally stable. Furthermore, the method is explicit in both displacement and velocity. The basic operations involved at each time step when performing the numerical integration are (see Figure 3):

1) The numerical integration process is performed.
2) The updated displacement is imposed by the hydraulic actuator.
3) The resulting force is fed back in the numerical integration scheme.
4) Step 1 is repeated.

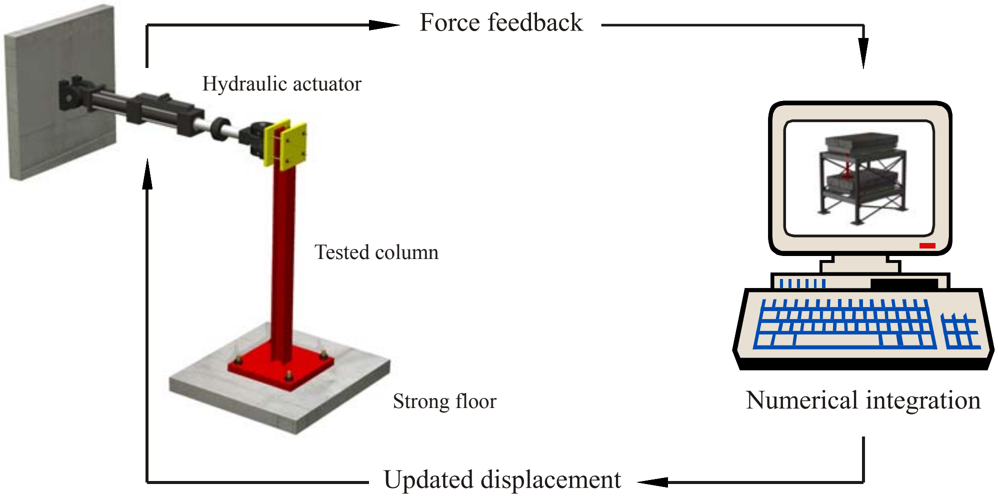

Figure 3 Basic operations involved when performing the hybrid test.

3.3. Numerical model and physical substructure

The numerical model comprised two dynamic degrees-of-freedom. The stiffness of the 1st storey was not modeled numerically because it was taken into account physically in the laboratory. The inelastic flexural behaviour of the steel column at the 2nd storey was modelled using the modified Giuffré-Menegotto-Pinto hysteretic model proposed by Fillipou et al. (1983). This hysteretic model was proven to correctly represent the behaviour of steel columns with residual stress patterns (Lamarche and Tremblay, 2008b). In the model, the following parameters based on the nomenclature employed in Mazzoni et al. (2006) were used: $F_y = 36$ kN (storey shear at yield), E = 0.7 kN/mm (lateral stiffness of the column), b = 5 %, $R_0 = 15$, $cR_1 = 0.8$, $cR_2 = 0.15$, $a_1 = a_3 = 2.0$ %, $a_2 = a_4 = 1.0$. In Figure 4a, the measured and predicted hysteretic flexural responses of the column are compared. To compute the numerical hysteretic response, a displacement time history measured during a hybrid test was used as an input. The numerical model presented in Figure 4a is shown to be in excellent agreement with the test data. In Figure 4a, V_1 and Δ_1 correspond to the inter-storey shear and inter-storey displacement at the first storey, respectively.

Rayleigh damping was used with $\xi = 1\%$ of critical damping in both modes based on harmonic test results near the two resonant frequencies of the structure, i.e. $f_1 \approx 1.0$ Hz and $f_2 \approx 2.5$ Hz. The friction induced by the rollers supporting the concrete masses was measured and taken into account in the numerical model. The beams of the braced frame supporting the rollers were found to have a slight geometrical inclination. This defect was also included in the numerical model. These combined effects were modeled with the inverted parallelogram hysteretic law shown in Figure 4b. In Figure 4b, a comparison between a quasi-static cyclic test and the numerical model is presented for the 1st storey. Parameters F_1 and u_1 respectively correspond to the frictional force, and the total displacement at the first storey. The inverted parallelogram hysteretic law had different amplitude levels in the positive and negative directions. The model was calibrated using a least square curve fitting method. Similar correlation was obtained for the second level.

In the shake table tests, the steel frame carrying the masses was supported on the strong floor of the laboratory and the 1st storey column of the specimen was anchored to the shake table. Hence, a total displacement finite element formulation had to be used in the numerical model, rather that the more common relative displacement formulation. In this multiple support formulation, the displacement and acceleration outputs of the shake table were used to properly model the frictional forces and adequately compare the shake table and hybrid test results. The time step used in the integration process was $\Delta t = 1/512$ s.

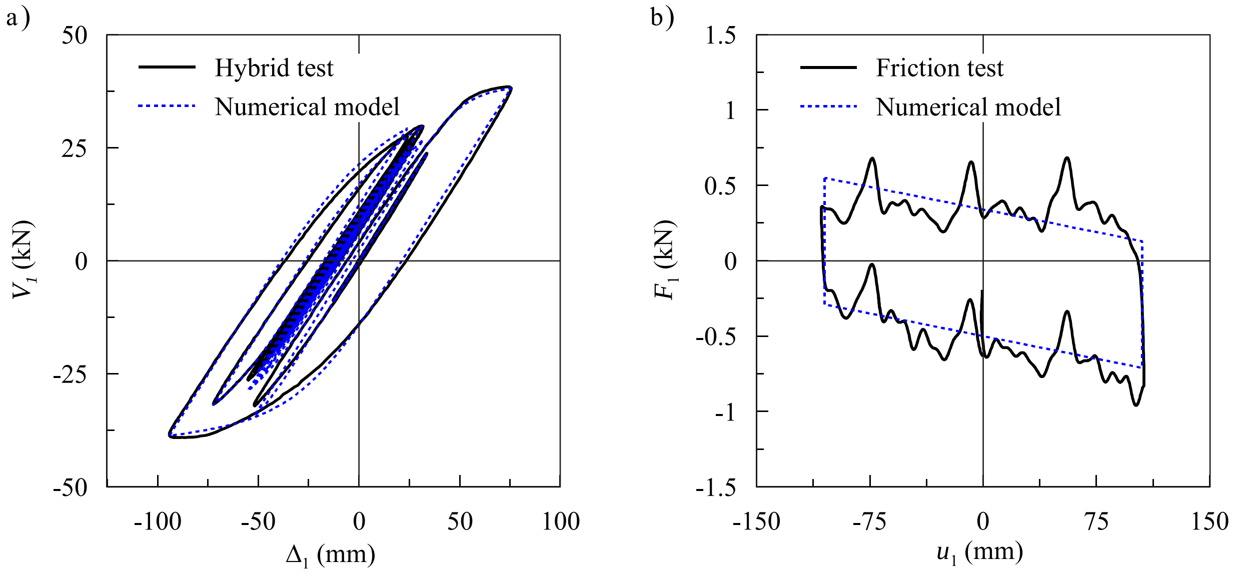

Figure 4 a) Inter-storey shear: hybrid test data *vs* numerical model; b) Friction test: data *vs* numerical model.

As explained in section 3.2, an initial estimate of the Jacobian matrix **W** is needed at the beginning of the numerical integration process. The matrix estimates used to construct **W** from Eq. 3.2.2 are:

$$\mathbf{M} = \begin{bmatrix} 7250 & 0 \\ 0 & 6500 \end{bmatrix} \text{kg}, \quad \mathbf{C}_0 = \begin{bmatrix} 1924 & -637 \\ -637 & 1220 \end{bmatrix} \text{kg/s}, \quad \mathbf{K}_0 = \begin{bmatrix} 1400 & -700 \\ -700 & 700 \end{bmatrix} \times 10^3 \,\text{kg/s}^2 \qquad (3.3.1)$$

3.4. Hybrid test program

During the hybrid test program, the same three excitation signals described in Table 2.2.1 were used. The sequence of testing was kept the same as in the shake test program to replicate the same strain demand history and, thereby, reproduce the same strain hardening effects.

4. COMPARISON BETWEEN TESTING TECHNIQUES

A comparison in the time domain between the shake table and the hybrid test results is presented in Figure 5. For each test, the time histories involving the inter-storey shear, V, is plotted for both levels. In addition, the corresponding hysteretic responses are also plotted as a function of the inter-storey displacements, Δ. Figure 5a shows the responses to the 1988 Saguenay earthquake record scaled to a Peak Ground Acceleration (PGA) of 2.62 g where the response is dominated by the second mode of the structure. In both tests, the structure remained mostly elastic, except for a slight inelastic excursion observed in the first storey during the shake table test, as depicted graphically on the corresponding hysteretic curve.

In Figure 5b, the responses to the 1940 El Centro record scaled to a PGA = 0.174 g are presented. The hysteretic plots indicate that the columns in both storeys remained within the linear range. Again, the displacement time histories obtained from both testing techniques are in excellent agreement.

The test results obtained for the unscaled 1940 El Centro record (PGA = 0.348 g) are depicted in Figure 5c. In these tests, the first-storey column experienced significant inelastic response. Good agreement is generally achi-

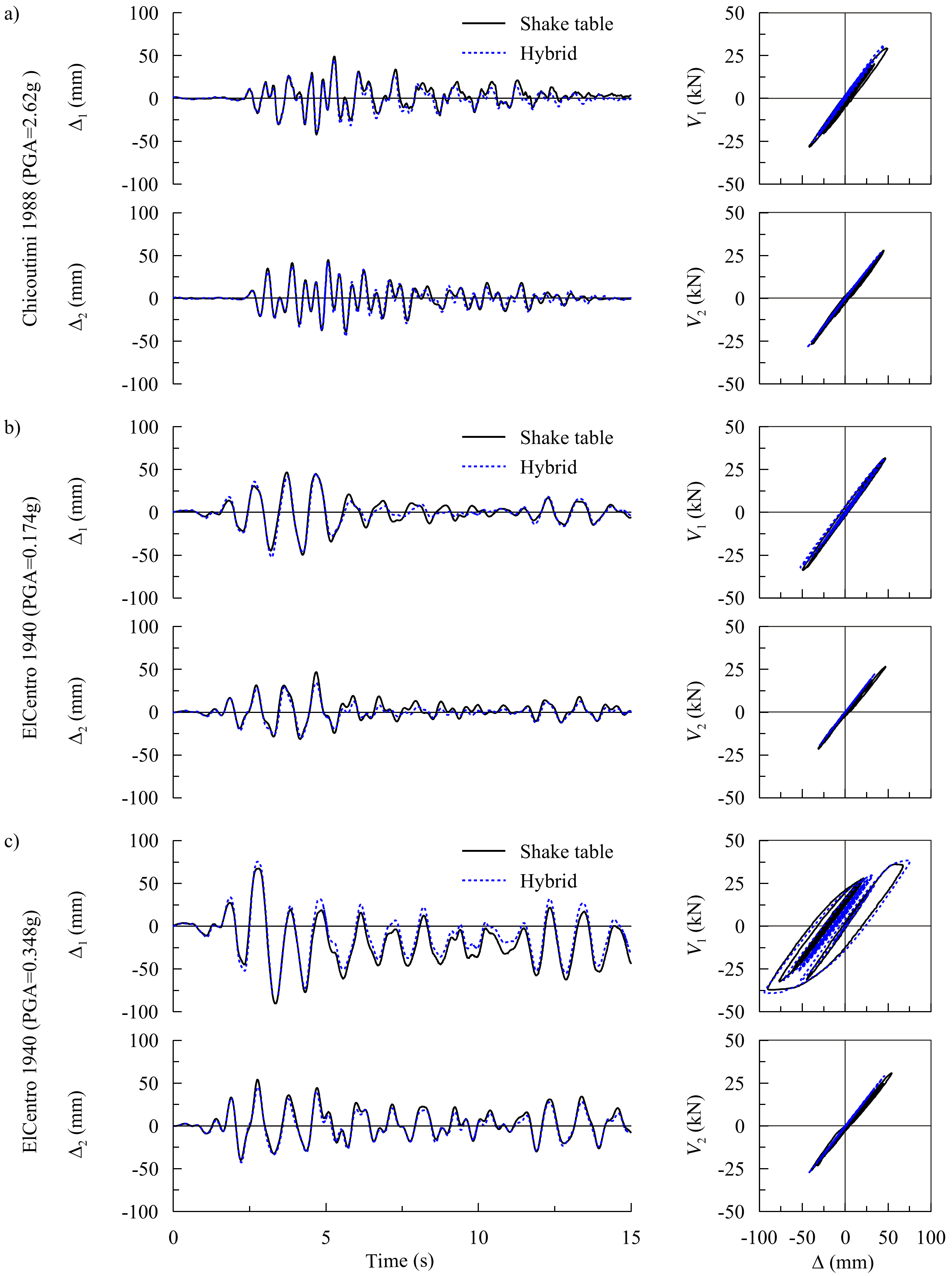

Figure 5 Comparisons between the shake table and hybrid test results: a) 1988 Saguenay (PGA=2.62g); b) 1940 El Centro (PGA=0.174g); and c) 1940 El Centro (PGA=0.348).

eved between the two testing methods. However, a shift gradually developed between t = 2 s and 5 s between the two first-storey displacement responses. This has been attributed to the difference in the hysteretic responses of the first-storey physical column specimens used in the two tests. As shown, the column in the hybrid test exhibits less strain hardening than the equivalent column in the shake table test, which likely resulted in higher displacement predicted by the hybrid test technique. This confirms that the nonlinear response of structural systems can be sensitive to slight variations in the hysteretic characteristics of the material or structural components that are expected to yield during an earthquake. Such variation and related uncertainties must be accounted for in numerical or physical assessment of the seismic performance of structural systems.

5. CONCLUSIONS

A study was conducted to directly compare the results obtained from shake table and real time hybrid simulation testing techniques for a half-scale model of a two-storey steel building structure. Only the first-storey structural components were included in the hybrid test program. The tests were performed under seismic ground motions exhibiting various amplitude levels and frequency contents. This permitted to examine both first and second mode dominated responses as well as both elastic and inelastic responses. The cyclic inelastic flexural responses could be well predicted using a modified Giuffré-Menegotto-Pinto hysteretic model. Excellent correlation was obtained between the two testing techniques, proving the capability of the Rosenbrock-W method for real time dynamic substructuring to adequately predict the linear and nonlinear seismic responses of ductile structural steel seismic force resisting systems. The tests confirmed that the inelastic seismic response of structures can be sensitive to small changes in the hysteretic characteristics of the yielding structural components. Such variation is unavoidable in typical civil engineering structures and must be accounted for in seismic performance assessment.

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REFERENCES

Fillipou, C.C., Popov, E.P., and Bertero, V.V. (1983). Effects of bond deterioration on hysteretic behavior of reinforced concrete joints. Report No. UCB/EERC-83/19, Earthquake Engineering Research Center, University of California, Berkeley, CA.

Mazzoni, S., McKenna, F., Scott, M.H., Fenves, G.L. (2006). Open System for Earthquake Engineering Simulation, User Command-Language Manual. Pacific Earthquake Engineering Research Center, University of California, Berkeley, CA.

Lamarche, C.-P., Bonelli, A., Bursi, O., and Tremblay, R. (2008a). A Rosenbrock-W method for real time sub-structuring and PSD testing. *Journal of Earthquake Engineering and Structural Dynamics* (submitted).

Lamarche, C.-P. and Tremblay, R. (2008b). Accounting for Residual Stresses in the Seismic Stability of Nonlinear Beam-Column Elements with Cross-Section Fiber Discretization. Proc. 2008 SSRC Annual Stability Conference, Nahsville, TN, 59-78.