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SHAKING-TABLE TEST OF A FRICTION-DAMPED FRAME STRUCTURE

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SUMMARY

This paper presents results from an international research project devoted to evaluating the seismic performance of a three-storey steel frame structure equipped with a friction-damping device (FDD) recently developed at the Technical University of Denmark. Experimental results indicate that the FDD performed very well in reducing the lateral storey drifts of the test frame. Numerical simulation of the seismic response of the primary and friction-damped frame was also conducted. This paper also compares the predictions of the displacement demand from the test results with those obtained by the capacity spectrum method. Copyright © 2004 John Wiley & Sons, Ltd.

1. INTRODUCTION

Passive energy dissipation devices have been successfully used to reduce the dynamic response of structures subjected to earthquakes. The primary reason for introducing energy dissipation devices into a building frame is to reduce the displacement and damage in the frame. Displacement reduction is archived by adding stiffness and/or energy dissipation (damping) to the building frame. Metallic yield, friction and viscoelastic energy dissipation devices typically introduce both stiffness and damping; viscous dampers will only increase the damping in a building frame. Friction-damped devices have been used as a component of these dampers because they provide high-energy dissipation potential at a relatively low cost and are easy to maintain, with some having already been implemented in buildings in different areas of the world. Several researchers have investigated the experimental testing and development of seismic design procedures for these dampers, e.g. Constantinou *et al*. (1991), Pall and Marsh (1982), Cherry and Filiatrault (1990), and design guidelines have been implemented in FEMA-273 (Building Seismic Safety Council, 1997) and FEMA-368 (Building Seismic Safety Council, 2000).

For the current study, an international team conducted an experimental research programme on a three-storey steel frame structure equipped with friction-damping devices (FDDs) at the large-scale testing facility of the National Centre for Research on Earthquake Engineering (NCREE), Taiwan. The new FDD was recently developed at the Technical University of Denmark (Mualla and Nielsen, 2000). In this program a three-storey steel moment-resisting frame braced by FDDs was tested using a shaking table. Only unilateral ground shaking was used to investigate the seismic responses of the test struc-

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ture. The performance of the FDD is discussed and numerical simulation of the seismic responses of the primary and friction-damped frame is also provided.

There are many simplified non-linear static analysis methods to estimate the maximum displacement of structures, such as the capacity spectra method, which utilizes the intersection of the capacity curve and reduced demand curve (e.g. ATC-40, Applied Technology Council, 1996; FEMA-273, Building Seismic Safety Council, 1997), and the displacement coefficient method, which utilizes a modified version of the equal displacement approximation (e.g. FEMA-273). Another objective of this paper is to compare the displacement demand obtained by the shaking-table test with those predicted by the push over analysis following the ATC-40 and FEMA-273 procedures.

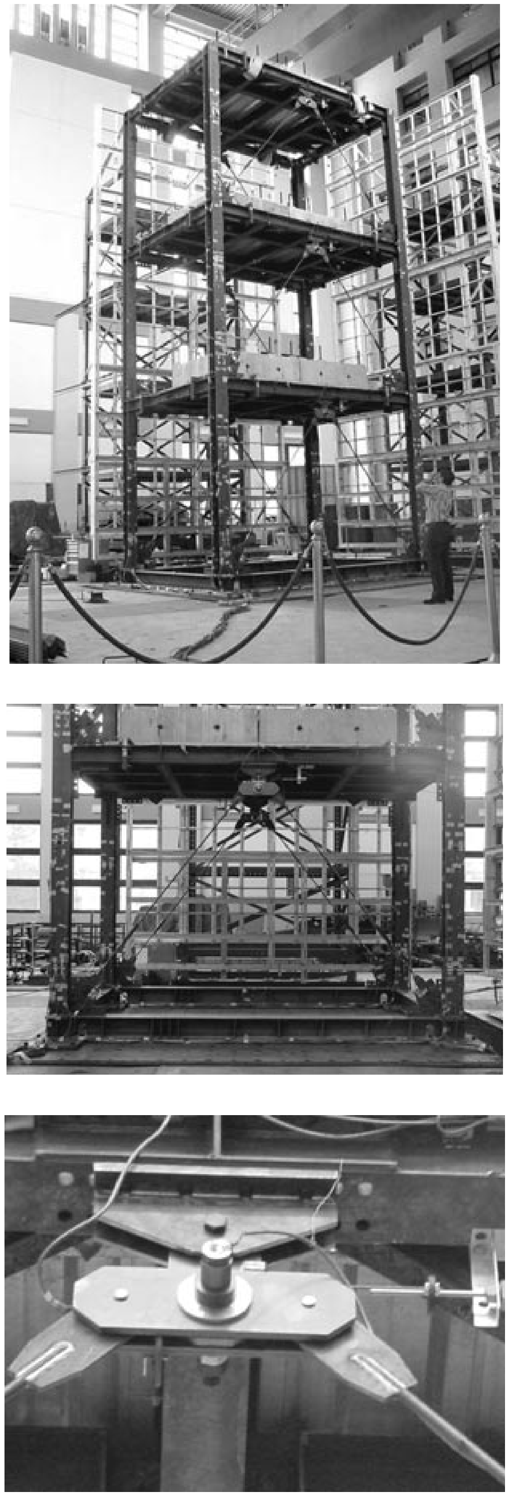
2. TEST PROGRAMME AND TEST RESULTS

The test structure was a three-storey steel moment-resisting frame with 3.0 m storey height and 4.5 m bay in the direction of shaking, as shown in Figure 1, and the total weight of the whole frame was about 34 tons. Two FDDs were installed at each storey. The damper unit attached to the girder and instrumented for the test is also shown in Figure 1(a). The details and the mechanism of the FDD are shown in Figure 2(a) and 2(b), respectively. The damper consists of three steel plates, rotating against each other around a pre-stressed bolt, which presses the plates together. Between these steel plates there are two circular friction pad discs, which provide dry friction lubrication in the unit, ensuring stable friction force of the movements. A friction pad disc with friction coefficient 0.4 was used in this programme.

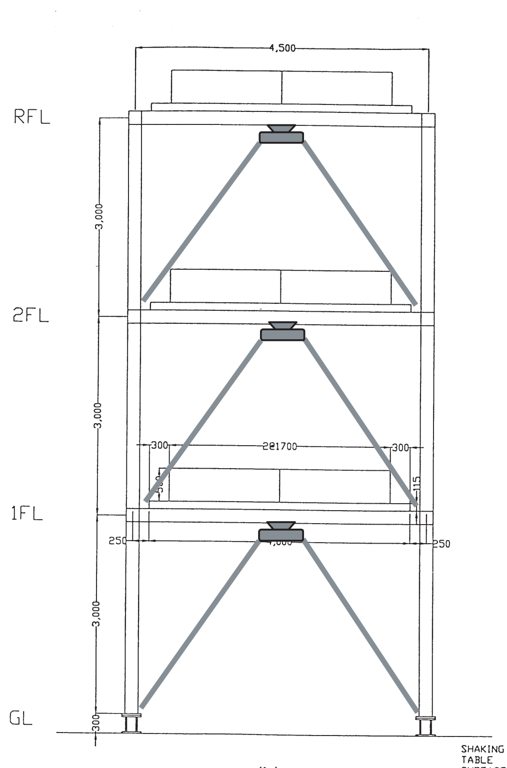
The central plates connected the damper device to the girder of the frame structure by a hinge, and the two sides were connected to the bracing system. In this test, inverted V-bracing was used. The bracing system consisted of pre-tensioned bar members in order to avoid buckling. The bracing bars were pin-connected at both ends to the damper and to the beam–column joint. The magnitude of pre-stress applied to the bolt and to the bracing bar was according to earthquake type and peak ground acceleration (PGA) level. In general, higher PGA needs higher pre-stress for both the bolt and the bracing bars. Table 1 shows the pre-stress applied to the bolts and bracing bars used in the test for input of Kobe earthquake with PGA = $0{\cdot}18\,g$. In order to have a constant friction force, several disc spring washers were used. In addition, hardened washers were placed between these spring washers and steel plates to prevent any marks on the steel plate due to the disc springs when they were in compression.

Three records were used as the input ground motion for the shaking-table test: the 1940 Imperial Valley–El Centro (EW), 1995 Kobe (Takatori EW) and 1999 Chi-Chi (TCU052 EW) earthquake. Various PGA levels of each record were employed to perform the unilateral excitation, and in total 14 tests were conducted in this programme. Figure 3 compares time histories of the first-floor storey drift between the friction-damped frame and primary frame by inputting the El Centro, Kobe and Chi-Chi earthquake records, respectively. The input PGA for each earthquake record is also shown in this figure. As observed from these figures, the friction damping system can effectively reduce the storey drift. Table 2 compares the maximum storey drift of the friction-damped frame and primary frame under various PGA with the input of different earthquake records. From this table, it can be found that the friction damping system can effectively reduce the storey drift in different PGA levels, but the reduction of storey drift decreases as the PGA level increases. Based on the 14 test results, the seismic performance and durability of the FDD can be summarized as follows:

1. The rigorous full-scale testing at the shaking-table facility of NCREE proved the excellent capacity of the damping system to significantly reduce the building vibrations, as shown in the tables.
2. After all 14 tests no damage occurred to the dampers, bracing bars, frame members or connections.



(a)



(b)

Figure 1. (a) Photos of the test frame, damper attached to the girder, and instruments. (b) Elevation view of the test steel frame

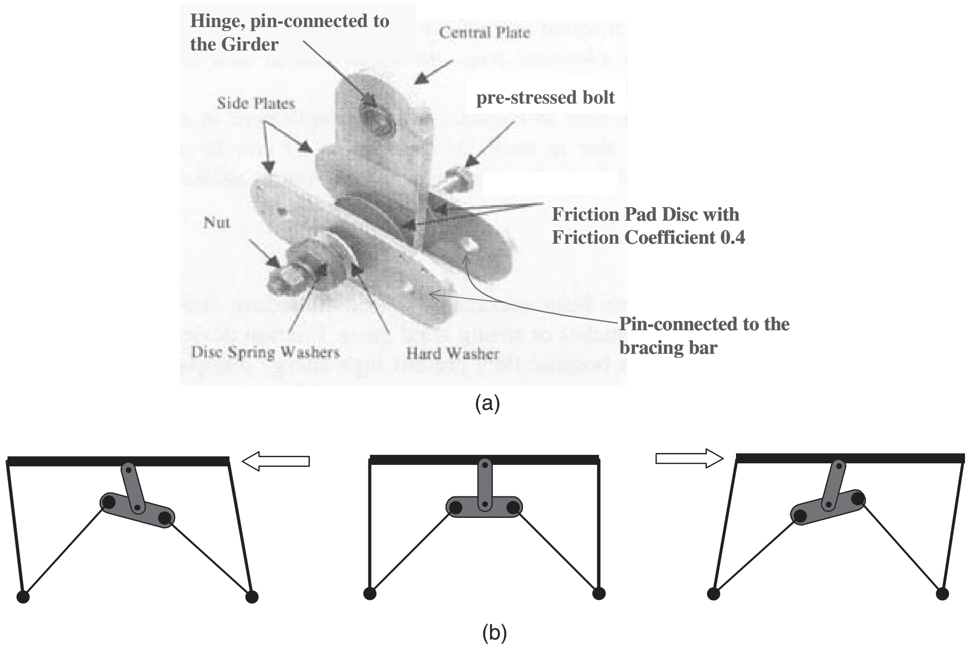


Figure 2. (a) Details of the friction damper device (Mualla and Nielsen, 2000). (b) Mechanism of the friction damper device

Table 1. Pre-stress in bracing bars and bolt

	Left bracing bar (kN)	Right bracing bar (kN)	Pre-stressed bolt (kN)	Yielding friction moment (kN m)
1st storey	15·9	14·6	37·8	1·510
2nd storey	16·1	10·7	49·5	1·586
3rd storey	10·1	10·5	25·6	1·016

3. The friction pad material also showed very good performance, and no scratches were observed on the sliding interfaces.
4. The mass of the dampers and the bracing system was about 0·15 tons—just 0·4% of the total test steel frame mass—and is expected to be much less in actual buildings.
5. The dampers remain in excellent condition and can be used for protection against future earthquakes.

3. NUMERICAL SIMULATION

The computer program DRAIN-2DX (Prakash and Powell, 1994) was adopted in this study for dynamic response simulation of the primary and friction-damped frames. In the analytical model of the frames, girders and columns are represented by line elements with a 2% strain hardening bi-linear model, the bracing bars were modelled by a truss element with pre-stress, and the FDDs were modelled by non-linear rigid-plastic rotational springs. The natural period of the three steel frames iden-

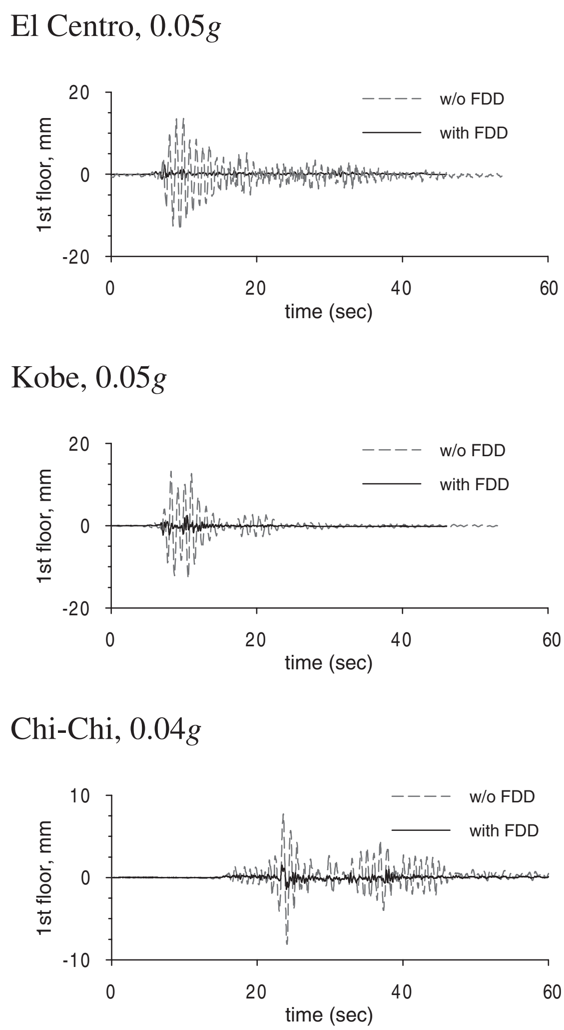


Figure 3. Comparison of storey drift between the primary and the damped frame for input of El Centro, Kobe and Chi-Chi earthquakes

tified through the system identification method ARX model is 0·912 s for the first mode and 0·287 s for the second mode, respectively, and the natural periods calculated by the proposed numerical simulation method are 0·923 s for the first mode and 0·292 s for the second mode. These results show good agreement with the results obtained by system identification.

Figure 4 compares the simulated and measured storey displacement of the primary frame excited by the El Centro earthquake, with peak ground acceleration 0·05 g, and the inherent damping ratio is taken as 0·02 for the primary frame structure. Figure 5 compares the simulated and measured storey drift of the friction-damped frame by input of the Kobe (PGA = 0·18 g) earthquake and the Chi-Chi (PGA = 0·15 g) earthquake, respectively. Those figures also show good agreement between the simulated responses of the friction-damped frame and the measured responses.

Table 2. Comparison of storey drift between primary and damped frame

Earthquake	PGA (g)	Max. storey drift, with damper (mm)	Max. storey drift, w/o damper (mm)	Reduction (%)
El Centro, USA, 1940	0·36	18·96	38·5[a]	50·7
	0·26	15·06	32·9[a]	54·5
	0·05	2·31	13·63	83·1
Kobe, Japan, 1995	0·18	8·67	32·3[a]	73·2
	0·12	5·87	24·2[a]	75·8
	0·05	2·50	13·11	80·9
Chi-Chi, Taiwan, 1999	0·15	13·50	47·0[a]	71·3
	0·12	10·69	39·7[a]	73·1
	0·04	1·52	8·10	81·2

[a] Results obtained by numerical simulation.

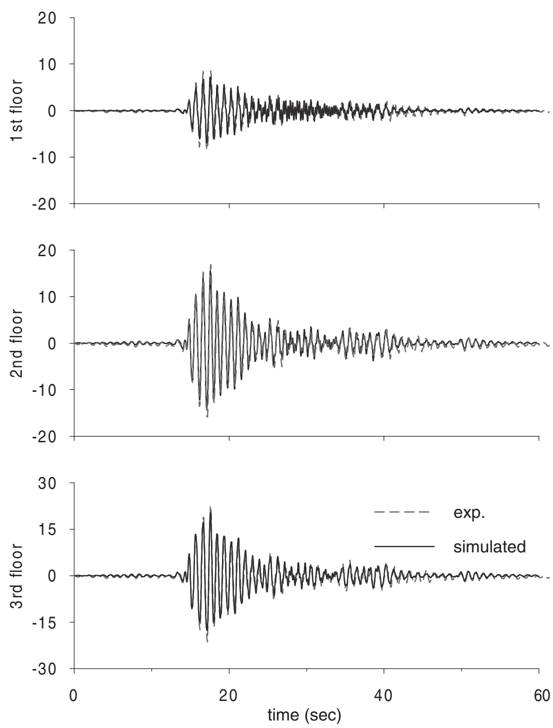


Figure 4. Comparison of the simulated and measured storey displacement of the primary frame by input of El Centro earthquake (PGA = 0·04 g)

4. CAPACITY SPECTRUM ANALYSIS AND COMPARISON WITH TEST RESULTS

Another objective of this paper was to compare the displacement demand predicted by the test results with that obtained by the pushover analysis procedure. For a building, static pushover analysis is used to find the capacity curve and convert it to the capacity spectrum by following the procedure defined

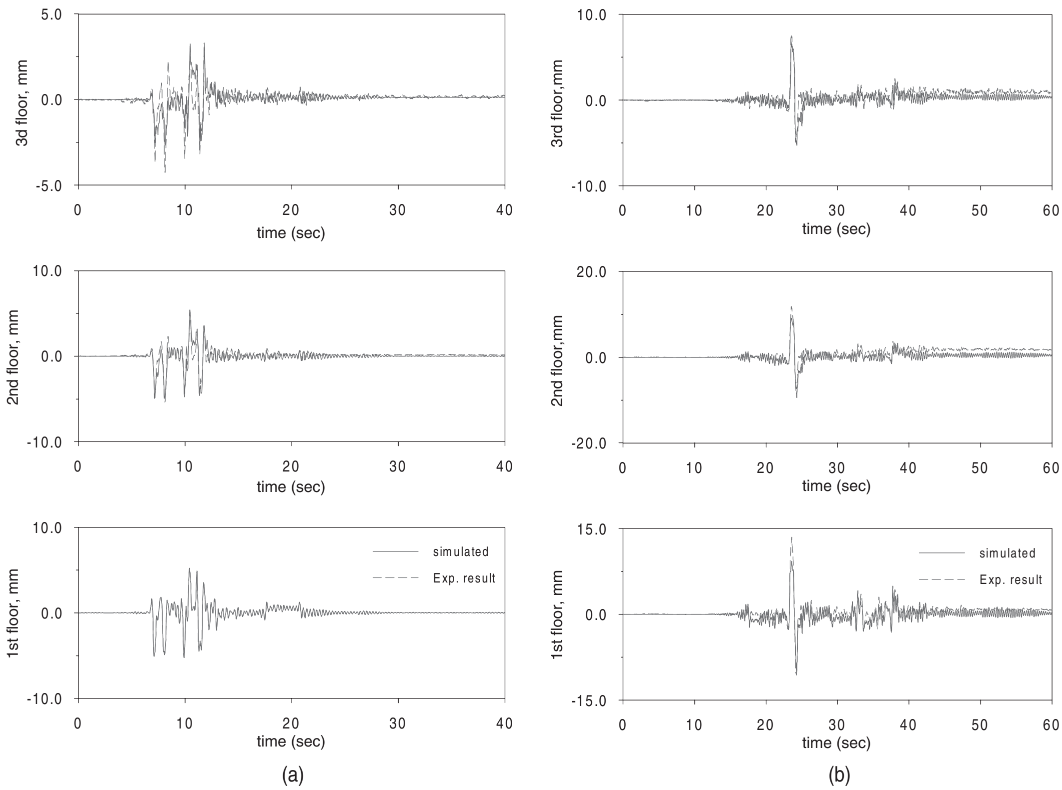


Figure 5. (a) Comparison of the simulated and measured storey drift of the damped frame by input of Kobe earthquake (PGA = 0·18 g). (b) Comparison of the simulated and measured storey drift of the damped frame by input of Chi-Chi earthquake (PGA = 0·15 g)

by FEMA-273 and ATC-40. The capacity spectrum is assumed to be unique, irrespective of the earthquake ground motion input. The capacity is used to demonstrate the seismic capacity of a building structure; its ordinate is usually the base shear V and its abscissa is usually the roof displacement Δ of the building structure. Through the non-linear pushover analysis, the relationship between the base shear V and the roof displacement Δ can be established, and the pushover curve converted to a capacity spectrum by the following equations:

$$S_a = \frac{V}{W\alpha_1} \tag{1}$$

$$S_d = \frac{\Delta}{PF_1\phi_{1,1}} \tag{2}$$

where PF_1 is the modal participation factor for the first mode, α_1 is the modal mass coefficient for the first mode, $\phi_{1,1}$ is the modal amplitude at roof storey of the first mode, W is the seismically effective weight of the frame structure, S_a is the spectral acceleration and S_d is the spectral displacement.

The effective hysteretic damping of the damped frame structure must be calculated in order to construct the demand curve. For the friction-damped structure, the effective hysteretic damping will be dependent on the level of deformation in the framing system. The effective hysteretic damping can be calculated as follows:

$$\beta_{eff} = \frac{W_D}{4\pi W_k} \tag{3}$$

where W_D is the energy dissipated by the primary frame and the FDD in one complete cycle of motion. The term W_k is the strain energy stored in the primary frame at displacements equal to those used to estimate W_D. In the pushover analysis, lateral forces are applied at each storey, resulting in corresponding displacements. The strain energy can be estimated as

$$W_k = \frac{1}{2}\sum_{i=1}^{N} F_i \delta_i \tag{4}$$

where F_i is the applied storey lateral force; the distribution of the storey lateral force used herein is proportional to the storey weight multiplied by the first mode shape amplitude of each storey; and δ_i is the resulting displacement at each storey.

Next, the earthquake demand per procedure defined by the ATC-40 method was obtained by linear spectrum analyses for each earthquake record and for various damping ratios of single degree of freedom structures. The capacity spectrum is super-imposed with the demand spectrum of the selected earthquake record with varying viscous damping ratios. The structure roof displacement is expected to be uniquely defined at an intersection point at which the estimated viscous damping of the demand spectrum is equal to the effective hysteretic damping estimated by Equation (3) of the friction-damped structure. From the procedure, the capacity spectrum of the three-storey steel frame and the demand spectrum of the El Centro earthquake at PGA = 0·36 g is shown in Figure 6, predicting that the roof displacement is about 56 mm. In this figure, the solid line represents the capacity spectrum of the primary frame and the dashed line represents the capacity spectrum of the friction-damped frame. The difference in spectrum acceleration of these two capacity spectra indicates the damping force provided by the FDDs. Table 3 summarizes the comparison of the spectral displacement obtained by the shaking-

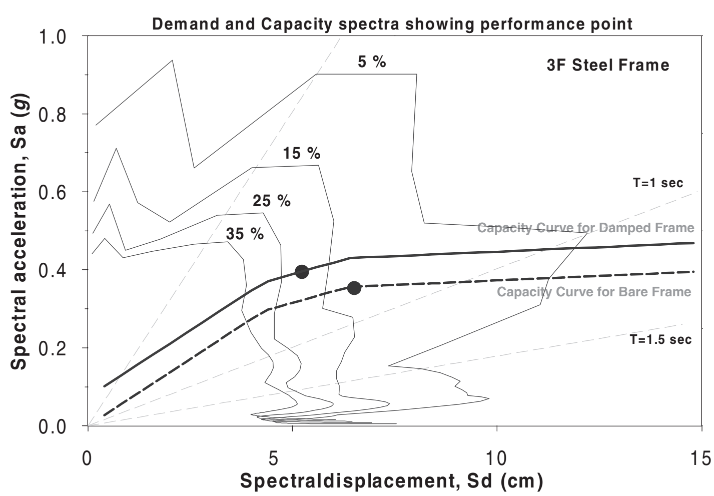


Figure 6. Capacity spectra of the steel frames and demand spectra of the El Centro earthquake (PGA = 0·36 g)

Table 3. Spectral displacement predicted by FEM 273 and by shaking-table test

Earthquake	PGA (g)	Shaking-table test (mm)	FEM 273 method (mm)
El Centro, USA, 1940	0·36	48·4	52·0
Kobe, Japan, 1995	0·18	22·5	14·0
Chi-Chi, Taiwan, 1999	0·15	32·3	24·0

table test and predicted by the pushover analysis procedure. It can be seen that the spectral displacements obtained by pushover analysis are very close to the test results for the input of the El Centro earthquake, but are about 40% and 25% underestimated for input of the Kobe and the Chi-Chi earthquakes, respectively.

5. CONCLUSIONS

An FDD equipped in a three-storey braced steel frame was tested at the National Centre for Research on Earthquake Engineering. The seismic response was evaluated using the shaking-table facility, simulating earthquake ground motions of far-field and near-fault types from the 1940 Imperial Valley–El Centro earthquake (USA), the 1995 Kobe earthquake (Japan) and the 1999 Chi-Chi earthquake (Taiwan). All tests confirmed the remarkable efficiency of the damping system in reducing lateral displacements and storey drifts of the test frame.

Numerical simulation of the seismic response of the test frames was also conducted, showing good agreement with the test results, which is valuable for design of an actual building and parameter study of the FDD. The predicted seismic displacement of the friction-damped frame by FEMA 273 methodology is compared with the test results, and shows good agreement for input of the El Centro earthquake, but underestimates the displacement responses for input of the Kobe and the Chi-Chi earthquakes. These two earthquake records belong to the near-fault type earthquake, which is much more destructive than the far-field earthquake. Thus it would be valuable to study how to improve the capacity spectrum method to accurately predict the structure responses when subjected to near-fault ground motions.

REFERENCES

Applied Technology Council. 1996. *Seismic Evaluation and Retrofit of Concrete Buildings*, Vol. 1, ATC-40. ATC: Redwood City, CA.

Building Seismic Safety Council. 1997. *NEHRP Guidelines and Commentary for the Seismic Rehabilitation of Buildings: FEMA 273–274*. BSSC: Washington, DC.

Building Seismic Safety Council. 2000. *NEHRP Recommended Provisions for Seismic Regulations for New Buildings and Other Structures: FEMA 368*. BSSC: Washington, DC.

Cherry S, Filiatrault A. 1990. Seismic Design Spectra for Friction Damped Structures. *Journal of the Structural Division, ASCE* **16**(ST5).

Constantinou MC, Reinhorn AM, Mokha AS, Watson R. 1991. 'Displacement control device for base isolated bridges,' *Earthquake Spectra* **7**(2): 179–200.

Mualla I, Nielsen LO. 2000. Parameters influencing the behavior of a new friction damper device. In *Papers in Structural Engineering and Materials*, Technical University of Denmark; 49–58.

Pall AS, Marsh C. 1982. Responses of friction damped braced frames. *Journal of the Structural Division, ASCE* **108**: 1313–1323.

Prakash V, Powell GH. 1994. *DRAIN-2DX: Base Program User Guide, Version 1.10*. A computer program distributed by NISEE/Computer applications, Department of Civil Engineering, University of California, Berkeley, CA.

NOMENCLATURE

F_i = applied story lateral force
PF_1 = modal participation factor for the first mode
S_a = spectral acceleration
S_d = spectral displacement
V = base shear of the frame structure
W = seismically effective weight of the frame structure
W_D = energy dissipated by the structure in one complete cycle of motion
W_k = strain energy stored in the structure
α_1 = modal mass coefficient for the first mode
$\phi_{1,1}$ = modal amplitude at roof storey of the first mode
δ_i = displacement at each story
Δ = roof displacement of the frame structure