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VE DAMPERS FOR UPGRADE OF NORTHRIDGE DAMAGED STEEL BUILDING

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ABSTRACT

A trial design for the upgrade modification of a 13-story building is presented. It incorporates supplemental damping to mitigate moment connection damage from earthquakes. The case study building is one that suffered damage during the Northridge earthquake. It is shown that: analytical models can reasonable correlate to the actual Northridge damage when the moment connection capacities are characterized as random variables; and, modifying the building with dampers is an effective way to reduce damage in future quakes.

KEYWORDS

Welded steel moment frames; viscoelastic dampers; random connection capacities; seismic upgrade.

INTRODUCTION

The Northridge, California earthquake of January 17, 1994 exposed deficiencies in design and construction practices for welded steel moment frame (WSMF) buildings. Numerous WSMF buildings suffered fractures in their beam-to-column connections. This was unexpected since prior to that time, WSMFs were believed to be ductile, i.e., capable of extensive yielding without loss of strength. This idea is now in question.

A consensus regarding the disposition of these damaged buildings is yet to be fully developed. Current public policy allows for their repair to pre-earthquake conditions with the recognition that they will not have the ductility presumed at the time of their original design. Such repaired WSMFs have the potential for the same brittle seismic behavior as that already experienced during the Northridge event. This paper presents a case study of an alternative upgrade modification using viscoelastic (VE) dampers that can mitigate future connection seismic damage and provide a larger seismic factor-of-safety, versus a strategy in which WSMFs are only repaired to their original condition.

CASE STUDY BUILDING

Fig. 1 depicts key features of the building. Designed in 1975, its lateral load resisting system consists of WSMFs located along each of its peripheral column rows (rows B, G, 4 and 9). Most WSMF beams and columns are W14 and W33 shapes, respectively. Corner columns are four-plate built-up box sections.

Beam-to-column connections utilize both welds and bolts. Complete joint penetration groove welds connect beam flanges to columns. Shear tab plates are welded to the columns and bolted to the beam webs. Web doubler plates and continuity plates are used to reinforce the columns. The connections are consistent with the "standard prequalified" type indicated by the Uniform Building Code.

Floor diaphragms consist of concrete filled steel decking. At the ground and plaza levels, the building has a large plan area extending beyond that shown in Fig. 1. The building is on a sloping site, and grade elevation is at the ground and plaza levels for the North and South ends of the building, respectively. This building was previously studied as part of the Phase 1 SAC Joint Venture Project (Uang *et al.*, 1995).

NORTHRIDGE PERFORMANCE

The building was located about 3 miles southwest of the 1994 Northridge earthquake epicenter (M6.8). The building has acceleration sensors located at the ground, 6th and roof levels, and the recordings were processed by CSMIP (Darragh *et al.*, 1994) (Station: Woodland Hills - Oxnard Blvd. #4, C246). The maximum recorded accelerations at the base and in-structure were 0.44g and 0.33g, respectively. Fig. 5 shows the actual North-South (NS) displacement time histories (relative to the building base). The peak drift ratio between the roof and base is about 0.006. Post-earthquake inspection revealed the following numbers of earthquake damaged connections (Bonowitz, 1995).

Table 1. Northridge Damaged Connections

Column Row	Direction	Damaged Connections	Inspected Connections Having Damage (percent)
4	NS	26	10%
9	NS	13	5
B	EW	9	3
G	EW	4	2
	Total	52	5%

Each beam flange is considered as a connection in the table, e.g., one beam framing into one column has two connections. Omitted from above are 10 type W1 damage/defects and 75 other additional connections that could be possible W1 damage/defects at worst, as determined by ultrasonic testing (for definition of W1, see Interim Guidelines, 1995). There is question as to whether these are earthquake damage or existing prior defects, and hence are assumed not caused by the earthquake.

The damaged connections are concentrated in the lower part of the building with only one found above the 7th floor level. The spatial distribution of the damage in the lower floors is random, and the damage patterns differ greatly among the WSMFs. The numbers of damaged connections vary by a factor of 2 in parallel frames, in spite of the symmetrical structural arrangement. The variability of damage in apparently identical WSMFs indicates the connection capacities may be characterized as random variables.

BUILDING MODEL

In order to evaluate the benefits of modification with VE dampers, a model representing the pre-earthquake structure was first created. The actual Northridge earthquake response from in-structure recorded motions was used to calibrate the model. For brevity, only the building NS direction results are presented in this paper.

A planar model representing the NS direction of the building was formulated using the computer program PC-ANSR (Maison, 1992). It is composed of nonlinear beam-column elements and nonlinear rotational springs. The rotational spring was developed having special properties to capture the beam-to-column connection and column panel zone behaviors. Fig. 2 depicts the modeling at a WSMF beam-to-column location. Sample hysteretic loops in Fig. 3 illustrate key element features including: a piece-wise linear skeleton curve having multiple facets, and a beam-to-column flange fracture rule which reduces the moment capacity when a prescribed failure rotation is reached. The random process defining the failure rotations uses a normal distribution based on experimental test data found in the literature (Fig. 4). Prior to the start of a nonlinear analysis, the failure rotations are defined by sampling from the distribution. When a negative value is drawn, the failure rotation is taken as a small value with a lower bound rotation corresponding to 2/3 of the yield moment of the beam. Other building modeling features include: P-delta effects, mass and stiffness proportional damping of 3%, and lateral fixity at the plaza level where part of the building is connected to grade.

NORTHRIDGE CORRELATIVE ANALYSIS

Several trial time history analyses were performed to calibrate the model to the actual building recorded response. A model having good fit has a fundamental period of 3.5 sec, and a failure rotation distribution mean and standard deviation of 0.005 rad and 0.004 rad, respectively. This results with about 20% of the connections having essentially no plastic rotation capacity, i.e., fracture near 2/3 of the beam yield moment value. Since the connections have their capacities set in a random manner, multiple nonlinear time history analyses (simulations) were performed to obtain statistics on the numbers of damaged connections.

As shown in Fig. 5, the typical analysis displacement time history is in good agreement with the actual building motions. Fig. 6(a) shows the distribution of damaged connections over the height of the building from ten simulations. The average number and concentration of damaged connections in the lower part of the building are consistent with the actual observed damage.

VE DAMPER UPGRADE

A trial damper design was performed with the objective to provide 20% supplemental damping to the building. Fig. 7 shows an elevation view of the damper arrangement in column rows 5, 6, 7 and 8. The dampers are located at the apex of inverted-V strut pairs. The dampers are made with slabs of 3M Company material type ISD-110 sandwiched between steel plates. Interstory displacements cause shearing of the VE material which dissipates energy. The general design methodology used is that described by Maison and Kasai, 1994.

Additional frames having VE dampers were added to the analytical model described above. The dampers were modeled using nonlinear damper elements that explicitly account for VE material frequency, temperature and strain dependence (Kasai *et al.*, 1993).

Ten repeat Northridge earthquake simulations were performed. Figs. 8 and 9 show the results from typical simulations for both the original building structure (No Dampers) and the upgraded structure with damper modifications (With Dampers). Note that the dampers also add to the building effective lateral stiffness as indicated by shortening of the apparent response period. The damper upgrade leads to reductions in peak deflections (-12% at the roof), story drifts (-43% at the 6th floor), and plastic rotations (-58% at the 6th floor). The rotations shown in Fig. 9 are the peak plastic rotation including both the beam-to-column connection and column panel zone plastic rotations.

Fig. 6(b) shows the distribution of damaged connections over the height of the building from the ten simulations. The upgraded structure has on average, only about one-fourth the number of damaged connections versus the original structure.

SEVERE EARTHQUAKE RESPONSE

Additional analyses were performed using a near-source acceleration record to evaluate the benefits of damper use under violent ground shaking conditions. The 1978 Tabas, Iran earthquake (M7.4) record was used. It is one of the largest earthquakes having a near-source accelogram recorded within 2 miles of the epicenter. Fig. 10 shows the response spectra.

One simulation each was performed using the original building model (No Dampers) and the upgraded model with damper modifications (With Dampers). Figs. 11 and 12 shows the results. The displacement time histories for the no damper case shows much softening due to fractured connections, as evidenced by the apparent period lengthening of the response beyond 12 sec during the earthquake. The peak building drift ratio between the ground and roof levels is 0.017. Like the Northridge analyses presented above, the damper upgrade leads to reductions in peak deflections (-29% at the roof), story drifts (-26% at the 6th floor), and plastic rotations (-61% at the 6th floor).

Fig. 13 depicts the damage over the building height. With no dampers, the building suffers damage to 26% of its connections, whereas the building modified with dampers has 15% connection damage. For this case, damper use reduces connection damage by about one-half.

CONCLUSIONS

1. Building damage from the Northridge earthquake can be reasonably simulated using analytical models that define the connection rotation capacities as random variables.

2. Many of the study building's connections experienced damage from moment demands that were less than the yield moment of the beams. Reasonable damage correlation was achieved by analytical models having about 20% of the connections with capacities near 2/3 the yield moment of the beams.

3. Upgrading WSMF buildings with supplemental damping devices can lead to reduced damage in future quakes. The reduction in connection damage can be significant as indicated by the case study building results summarized below.

Table 2. Moment Connection Damage

Earthquake	No Dampers (number)	With Dampers (number)	Percentage Reduction In Connection Damage
1994 Northridge (M6.8)	24.0	6.3	-74%
1978 Tabas (M7.4)	73	41	-44%

ACKNOWLEDGMENTS

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REFERENCES

Bonowitz, D. (1995). Revised Damage Results, *Private Communication*, July 10.

Darragh, R., T. Cao, V. Graizer, A. Shakal and M. Huang (1994). *Los Angeles Code-Instrumented Building Records From the Northridge, California Earthquake of January 17 1994: Processed Release No. 1*, Report No. OSMS 94-17, California Strong Motion Instrumentation Program, Sacramento, California.

Interim Guidelines: Evaluation, Repair, Modification and Design of Welded Steel Moment Frame Structures (1995). Federal Emergency Management Agency FEMA 267.

Kasai, K., J.A. Munshi, M. Lau and B.F. Maison (1993). Viscoelastic Damper Hysteretic Model: Theory, Experiment, and Application, *Seminar on Seismic Isolation, Passive Energy Dissipation, and Active Control*, Applied Technology Council, ATC-17-1, San Francisco, March.

Maison, B.F. (1992). *PC-ANSR: A Computer Program For Nonlinear Structural Analysis*, Available from National Information Service For Earthquake Engineering - NISEE/Computer Applications, University of California, Berkeley.

Maison, B.F. and K. Kasai (1994). Case Study of Building Retrofit Using Viscoelastic Dampers, *Proceedings Fifth U.S. National Conference on Earthquake Engineering*, Chicago, Earthquake Engineering Research Institute.

Uang, C.M., Q.S. Yu, A. Sadre, D. Bonowitz and N. Youssef (1995). *Performance of a 13-Story Steel Moment-Resisting Frame Damaged in the 1994 Northridge Earthquake*, Report No. SSRP-95/04 (draft), Structural Systems Research, University of California, San Diego, California.

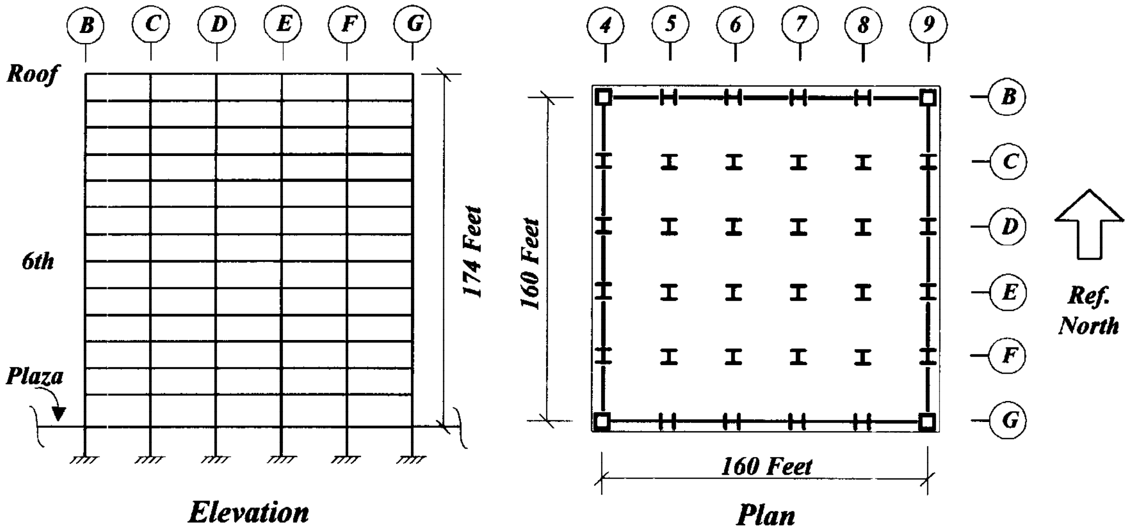

Fig. 1. Study building

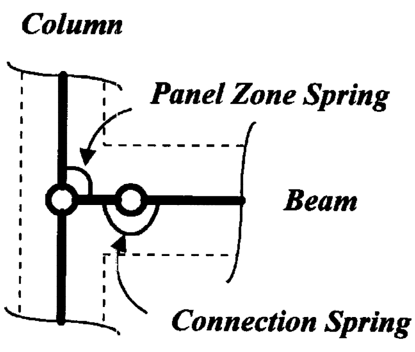

Fig. 2. Beam-to-column modeling

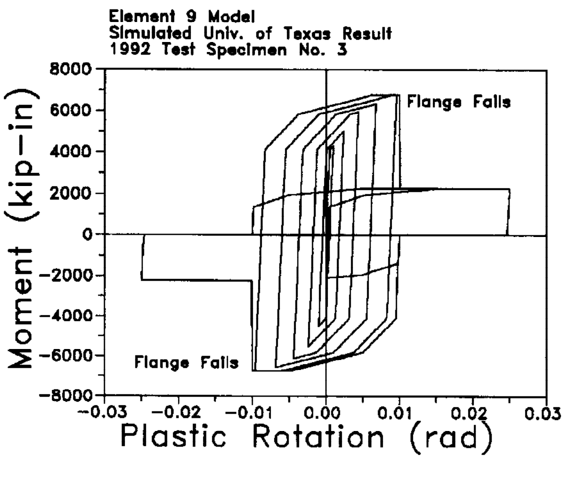

Fig. 3. Rotational spring hysteretic model

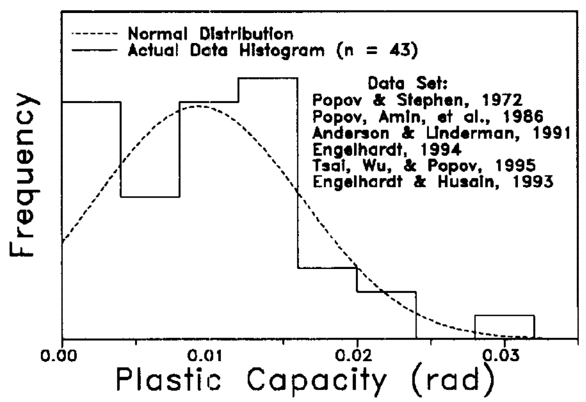

Fig. 4. Ultimate plastic rotations from tests

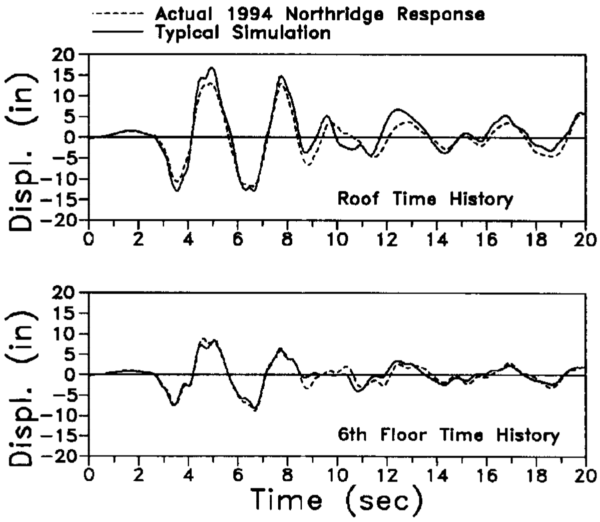

Fig. 5. Northridge time history results

DISTRIBUTION OF DAMAGE FROM 10 SIMULATIONS
1994 NORTHRIDGE, NO DAMPERS

		ROW B	ROW C		ROW D		ROW E		ROW F		ROW G	TOT/FLR
		R	L	R	L	R	L	R	L	R	L	
ROOF	TOP FLG	-	-	-	-	-	-	-	-	-	-	-
	BOT FLG	-	-	-	-	-	-	-	-	-	-	
FL 12	TOP FLG	-	-	-	-	-	-	-	-	-	-	-
	BOT FLG	-	-	-	-	-	-	-	-	-	-	
FL 11	TOP FLG	-	-	-	1	-	1	-	-	-	-	2
	BOT FLG	-	-	-	-	-	-	-	-	-	-	
FL 10	TOP FLG	-	-	-	-	-	-	-	-	-	1	1
	BOT FLG	-	-	-	-	-	-	-	-	-	-	
FL 9	TOP FLG	2	1	1	1	-	-	3	-	1	3	15
	BOT FLG	-	-	-	-	-	-	-	-	-	3	
FL 8	TOP FLG	1	1	1	3	-	4	2	4	1	1	26
	BOT FLG	1	-	1	-	-	1	1	-	4	-	
FL 7	TOP FLG	4	4	1	3	1	4	1	5	-	6	44
	BOT FLG	3	1	1	-	2	-	4	-	3	1	
FL 6	TOP FLG	2	4	-	3	1	3	2	2	1	4	35
	BOT FLG	4	-	2	-	2	-	1	-	2	2	
FL 5	TOP FLG	-	2	1	4	1	5	5	3	1	1	34
	BOT FLG	1	-	1	-	3	1	1	1	1	2	
FL 4	TOP FLG	4	2	2	2	2	1	1	3	2	1	24
	BOT FLG	1	-	-	-	-	-	-	-	1	2	
FL 3	TOP FLG	1	1	2	3	-	1	1	3	1	4	25
	BOT FLG	2	2	-	-	1	-	-	1	-	2	
FL 2	TOP FLG	1	4	1	2	-	3	3	3	5	-	33
	BOT FLG	1	1	-	-	-	-	1	2	4	2	
FL 1	TOP FLG	1	-	-	-	-	-	-	-	-	-	1
	BOT FLG	-	-	-	-	-	-	-	-	-	-	
PLAZA	TOP FLG	-	-	-	-	-	-	-	-	-	-	-
	BOT FLG	-	-	-	-	-	-	-	-	-	-	

\- = NO DAMAGE IN ANY SIMULATION
2 = DAMAGED IN TWO SIMULATIONS, ETC.
L = LEFT SIDE OF COLUMN
R = RIGHT SIDE OF COLUMN

AVE. NUMBER PER SIMULATION = 24.000
STANDARD DEVIATION, SIGMA = 4.320
COE. OF VARIATION, COV = 0.180

(a) no dampers

DISTRIBUTION OF DAMAGE FROM 10 SIMULATIONS
1994 NORTHRIDGE, WITH DAMPERS

		ROW B	ROW C		ROW D		ROW E		ROW F		ROW G	TOT/FLR
		R	L	R	L	R	L	R	L	R	L	
ROOF	TOP FLG	-	-	-	-	-	-	-	-	-	-	-
	BOT FLG	-	-	-	-	-	-	-	-	-	-	
FL 12	TOP FLG	-	-	-	-	-	-	-	-	-	-	-
	BOT FLG	-	-	-	-	-	-	-	-	-	-	
FL 11	TOP FLG	-	-	-	-	-	-	-	-	-	-	-
	BOT FLG	-	-	-	-	-	-	-	-	-	-	
FL 10	TOP FLG	-	-	-	-	-	-	-	-	-	-	-
	BOT FLG	-	-	-	-	-	-	-	-	-	-	
FL 9	TOP FLG	-	-	-	-	-	-	-	-	-	4	4
	BOT FLG	-	-	-	-	-	-	-	-	-	-	
FL 8	TOP FLG	-	-	-	1	-	-	-	1	-	2	4
	BOT FLG	-	-	-	-	-	-	-	-	-	-	
FL 7	TOP FLG	-	1	-	1	-	3	-	1	-	2	9
	BOT FLG	1	-	-	-	-	-	-	-	-	-	
FL 6	TOP FLG	-	-	-	2	-	1	-	2	-	3	9
	BOT FLG	1	-	-	-	-	-	-	-	-	-	
FL 5	TOP FLG	1	-	-	1	1	-	-	-	-	2	5
	BOT FLG	-	-	-	-	-	-	-	-	-	-	
FL 4	TOP FLG	1	2	-	2	-	1	-	2	-	3	18
	BOT FLG	3	-	1	-	1	-	2	-	-	-	
FL 3	TOP FLG	-	3	-	-	-	1	-	-	-	-	7
	BOT FLG	1	-	1	-	1	-	-	-	-	-	
FL 2	TOP FLG	1	-	-	-	-	2	-	1	-	2	7
	BOT FLG	-	-	-	-	-	-	1	-	-	-	
FL 1	TOP FLG	-	-	-	-	-	-	-	-	-	-	-
	BOT FLG	-	-	-	-	-	-	-	-	-	-	
PLAZA	TOP FLG	-	-	-	-	-	-	-	-	-	-	-
	BOT FLG	-	-	-	-	-	-	-	-	-	-	

\- = NO DAMAGE IN ANY SIMULATION
2 = DAMAGED IN TWO SIMULATIONS, ETC.
L = LEFT SIDE OF COLUMN
R = RIGHT SIDE OF COLUMN

AVE. NUMBER PER SIMULATION = 6.300
STANDARD DEVIATION, SIGMA = 2.163
COE. OF VARIATION, COV = 0.343

(b) with dampers

Fig. 6. Simulated connection damage (Northridge)

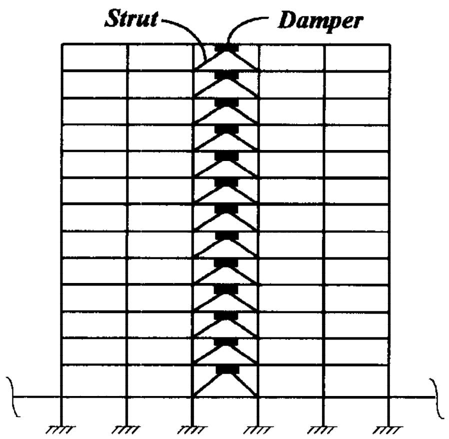

Fig. 7. Damper arrangement

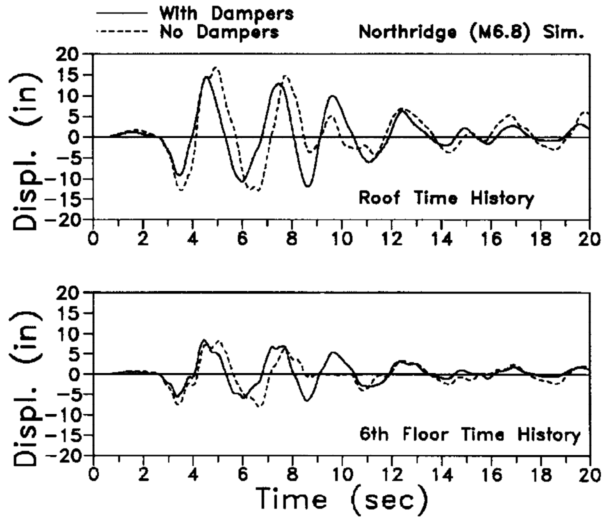

Fig. 8. Effect of dampers (Northridge)

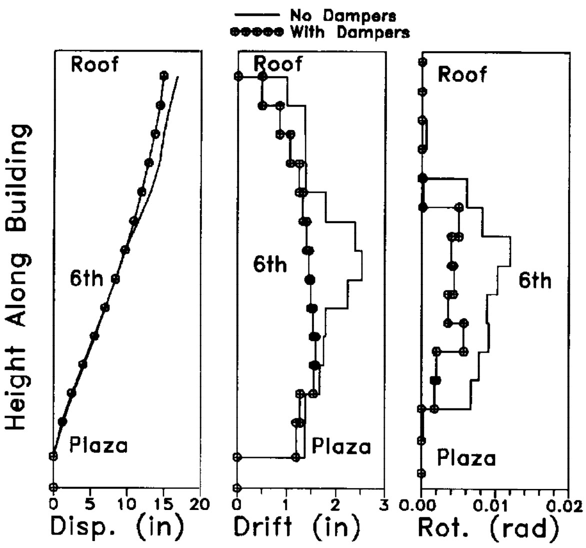

Fig. 9. Peak response envelopes (Northridge)

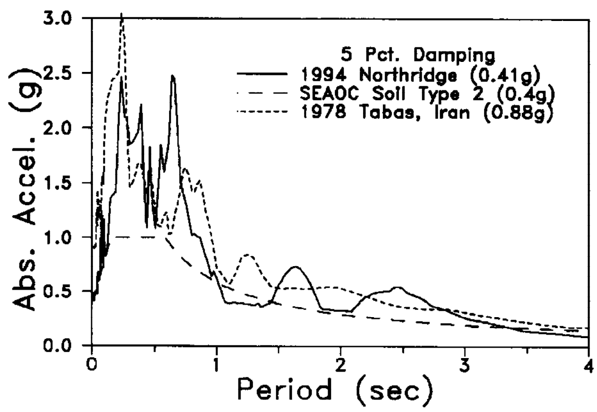

Fig. 10. Response spectra

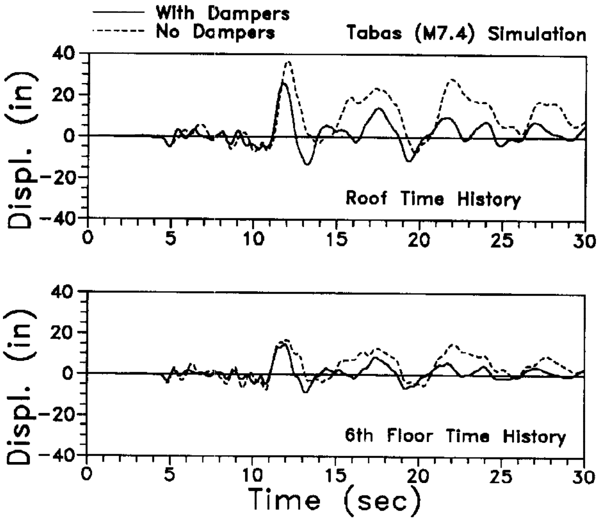

Fig. 11. Effect of dampers (Tabas)

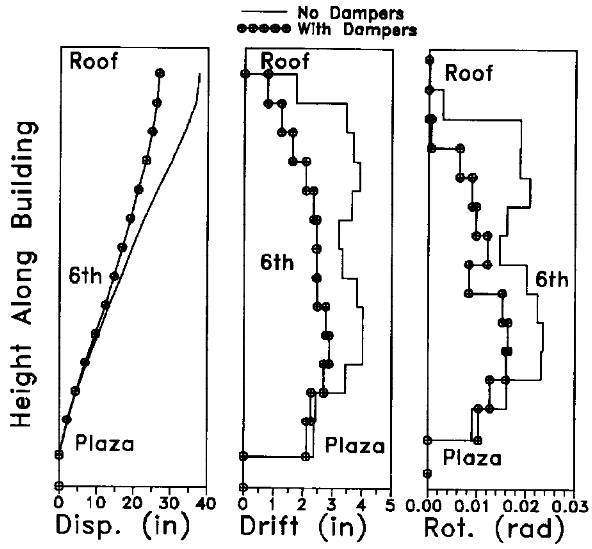

Fig. 12. Peak response envelopes (Tabas)

DISTRIBUTION OF DAMAGE FROM 1 SIMULATION
1978 TABAS, NO DAMPERS

		ROW B	ROW C		ROW D		ROW E		ROW F		ROW G	TOT/FLR
		R	L	R	L	R	L	R	L	R	L	
ROOF	TOP FLG	-	-	-	-	-	-	-	-	-	-	-
	BOT FLG	-	-	-	-	-	-	-	-	-	-	
FL 12	TOP FLG	-	-	-	-	-	-	-	-	-	-	-
	BOT FLG	-	-	-	-	-	-	-	-	-	-	
FL 11	TOP FLG	-	D	-	-	D	-	-	D	-	D	7
	BOT FLG	D	-	D	-	-	-	-	-	D	-	
FL 10	TOP FLG	-	D	D	D	-	D	-	-	-	D	7
	BOT FLG	-	-	D	-	-	-	D	-	-	-	
FL 9	TOP FLG	-	-	-	D	-	D	-	D	-	D	9
	BOT FLG	D	-	D	D	D	-	-	-	D	-	
FL 8	TOP FLG	-	-	-	-	-	-	-	-	-	D	3
	BOT FLG	D	-	-	-	-	-	-	-	D	-	
FL 7	TOP FLG	-	-	-	D	D	-	D	D	-	-	6
	BOT FLG	-	-	D	-	D	-	-	-	-	-	
FL 6	TOP FLG	-	-	-	D	-	D	-	D	-	-	6
	BOT FLG	D	-	-	-	D	-	-	-	D	-	
FL 5	TOP FLG	-	-	-	-	-	D	-	D	-	D	6
	BOT FLG	D	-	-	-	-	-	D	-	D	-	
FL 4	TOP FLG	-	D	-	-	-	D	-	D	-	-	8
	BOT FLG	D	-	D	-	D	-	D	-	D	-	
FL 3	TOP FLG	-	D	-	D	D	D	-	D	-	D	10
	BOT FLG	D	-	D	-	D	-	-	-	D	-	
FL 2	TOP FLG	-	-	-	D	D	D	-	D	-	D	8
	BOT FLG	D	-	-	-	-	-	D	-	D	-	
FL 1	TOP FLG	D	-	-	-	D	-	D	-	-	-	3
	BOT FLG	-	-	-	-	-	-	-	-	-	-	
PLAZA	TOP FLG	-	-	-	-	-	-	-	-	-	-	-
	BOT FLG	-	-	-	-	-	-	-	-	-	-	

- = NO DAMAGE
D = DAMAGED
L = LEFT SIDE OF COLUMN
R = RIGHT SIDE OF COLUMN

TOTAL NUMBER = 73.000

(a) no dampers

DISTRIBUTION OF DAMAGE FROM 1 SIMULATION
1978 TABAS, WITH DAMPERS

		ROW B	ROW C		ROW D		ROW E		ROW F		ROW G	TOT/FLR
		R	L	R	L	R	L	R	L	R	L	
ROOF	TOP FLG	-	-	-	-	-	-	-	-	-	-	-
	BOT FLG	-	-	-	-	-	-	-	-	-	-	
FL 12	TOP FLG	-	-	-	-	-	-	-	-	-	-	-
	BOT FLG	-	-	-	-	-	-	-	-	-	-	
FL 11	TOP FLG	-	-	-	-	-	-	-	-	-	-	-
	BOT FLG	-	-	-	-	-	-	-	-	-	-	
FL 10	TOP FLG	-	-	-	-	-	-	-	D	-	D	3
	BOT FLG	D	-	-	-	-	-	-	-	-	-	
FL 9	TOP FLG	-	D	-	-	-	-	-	D	-	-	3
	BOT FLG	-	-	-	-	D	-	-	-	-	-	
FL 8	TOP FLG	-	-	-	-	-	-	-	D	-	D	4
	BOT FLG	-	-	-	-	-	-	D	-	D	-	
FL 7	TOP FLG	-	-	-	-	-	D	-	-	-	D	3
	BOT FLG	-	-	-	-	-	-	D	-	-	-	
FL 6	TOP FLG	-	-	-	-	-	-	-	-	-	-	1
	BOT FLG	-	-	-	-	-	-	D	-	-	-	
FL 5	TOP FLG	-	D	-	D	-	-	-	-	-	D	6
	BOT FLG	-	-	-	-	D	-	D	-	D	-	
FL 4	TOP FLG	-	D	-	-	-	D	D	D	-	D	9
	BOT FLG	-	-	D	-	D	-	D	-	D	-	
FL 3	TOP FLG	-	D	-	-	-	-	-	D	-	-	4
	BOT FLG	-	-	D	-	-	-	-	-	D	-	
FL 2	TOP FLG	D	-	-	-	-	-	-	-	-	-	3
	BOT FLG	D	-	-	-	-	-	-	-	D	-	
FL 1	TOP FLG	-	-	-	-	-	D	-	-	D	D	5
	BOT FLG	D	-	-	-	-	-	D	-	-	-	
PLAZA	TOP FLG	-	-	-	-	-	-	-	-	-	-	-
	BOT FLG	-	-	-	-	-	-	-	-	-	-	

- = NO DAMAGE
D = DAMAGED
L = LEFT SIDE OF COLUMN
R = RIGHT SIDE OF COLUMN

TOTAL NUMBER = 41.000

(b) with dampers

Fig. 13. Simulated connection damage (Tabas)