BRACE DAMPERS: AN ALTERNATIVE STRUCTURAL SYSTEM FOR IMPROVING THE EARTHQUAKE PERFORMANCE OF BUILDINGS

by

Roger E. Scholl*

SUMMARY

A new device, referred to as the Brace Damper, has been developed specifically to improve the earthquake performance of buildings. Typically installed at the cross points in braced frame structure systems, the brace damper produces two types of righting forces when deformed: 1) a spring force proportional to displacement (drift), and 2) a damping force proportional to velocity. The increased damping facilitated by the brace damper reduces earthquake demand forces and energy; the increased stiffness serves to increase the potential energy capacity of a structure. Example analyses show that expected structural and non-structural damage from severe earthquakes can be substantially reduced by including brace dampers in building construction.

INTRODUCTION

Conventional lateral force resisting structural systems in seismic design include moment-resisting frames, braced frames, and shear walls. In recent years, the eccentric-braced frame, K-braced frame and other systems have been successfully proposed as alternatives for resisting lateral loads. Now another engineered alternative is proposed and is referred to as the Brace Damper.

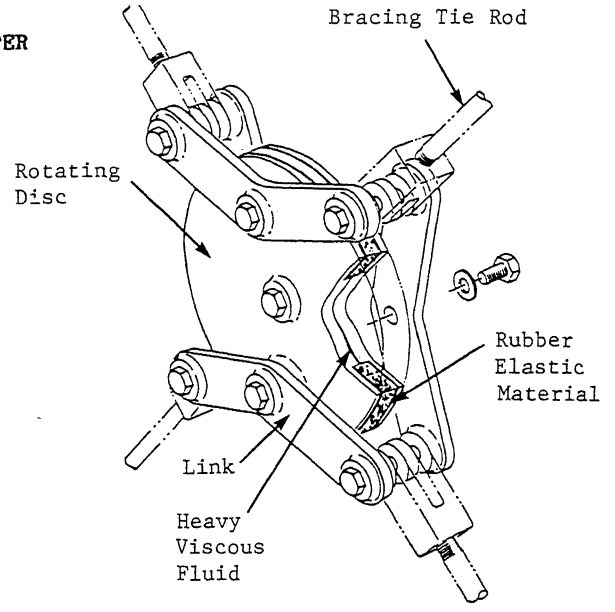
Choice of a proper structure system for providing lateral force resistance is one of the most difficult tasks for the structural engineer. Braced steel frames are known for their efficiency in providing lateral stiffness and strength. Conventional concentric braced steel frames are brittle, however, and do not facilitate much lateral deformation. Properly designed ductile moment resisting space frames provide adequate resistance to earthquake energy demands, but at the expense of excessive drifts causing both nonstructural and structural damage during severe earthquakes, and at the expense of greater quantities of construction steel. An integration of these two diverse approaches to provide resistance to lateral forces would lead to more economical structures as well as to structures that perform better under lateral force loading conditions such as earthquakes.

PHYSICAL AND ENGIEERING CHARACTERISTICS OF BRACE DAMPERS

The Brace Damper was develped by the Oiles Industry Co., Ltd., of Tokyo, Japan. Mechanically, the Brace Damper consists of four links and two rotating plates (See Figure 1). The Brace Damper is typically installed at the cross point of diagonal braces in an X-braced system. The four links are the means for attaching the damper to the bracing. When interstory drift imposed by lateral forces occurs, the two plates are forced to counter-rotate. The principal engineering feature of the Brace Damper is that the counter rotary motion of the plates produces two types of righting forces: 1) a spring force proportional to displacement (drift) and 2) a damping force proportional to velocity.

* Consulting Engineer, Redwood City, California

FIGURE 1 SCHEMATIC OF BRACE DAMPER



The functional stiffness and damping of the Brace Damper can be set at will (within a given range) and accordingly, the device can be used to alter stiffness and/or damping in a structure. The Brace Damper is thin and can be installed in any standard wall space. It is also permanently lubricated and requires no maintenance. Engineering and physical properties of typical size brace dampers are given in Table 1.

TABLE 1
MECHANICAL AND ENGINEERING PROPERTIES OF VARIOUS SIZE BRACE DAMPERS

Spring Constant (ton/cm)	0.5	1	1.5	2	3	5	10
Damping Coefficient (kg-sec/cm)	22.5	41.5	58.3	73.7	103.5	152	255
Outside Dimension of Square Shape (cm)	25.5	32.6	38.2	43.2	50.8	65.6	92.7
Rotating Plate Diameter (cm)	17	23	28	32	38	50	70
Thickness (cm)	8	9	9	9	10	12	14
Weight (kg)	4	7	12	18	28	50	120

EARTHQUAKE DEMAND ON STRUCTURES

Currently a variety of earthquake motion characterizations are widely used to prescribe earthquake demand on structures. These include: 1) Equivalent Lateral Force (ELF), 2) Time History, and 3) Response Spectrum. Earthquake input and structure response forces and displacements can be calculated using any of the above characterizations. In addition to these, Hudson (Ref. 1) described ground motion demand on structures in terms of energy as follows:

$$\text{Potential Energy Demand } (PE_D) = (1/2)\, mS_v^2 \qquad (1)$$

in which m = The mass of a single degree-freedom oscillator (Lb-sec^2/in)

S_v = Response spectrum velocity (in/sec)

Figures 2 shows response spectra for 5% and 30% damping that are used in subsequent sections of this paper. The curve representing UBC base shear was derived assuming $Z = 1$, $I = 1$, $C = 1/15\sqrt{T} \leq 0.12$, $K = 0.67$, $S = 1.2$, $T = 0.1N$ and assuming a straight line mode shape. The Hypothetical spectrum curves representing severe ground shaking were derived using the response spectrum shape for plus one sigma amplitude characteristics described by Newmark, Blume, and Kapur (Ref. 2) and subsequent recommendations (Ref. 3). On the basis of current seismological data, it might be conjectured that the hypothetical curves are representative of an upper bound on earthquake shaking.

EARTHQUAKE RESPONSE AND DAMAGE EVALUATION

There currently exist numerous publications including definitive text books that elucidate conventional procedures for calculating the response of structures subjected to earthquake ground motion. In addition to these, Housner (Ref. 4) and Blume (Ref. 5) describe procedures for estimating the nonlinear (inelastic) response of structures using energy procedures.

Conventional seismic analysis of structures involves consideration of only the shaking forces acting on a structure. Recently, Scholl (Ref. 6) determined that to comprehensively evaluate the overturning of rigid blocks subjected to earthquake excitation, the product of ground motion acceleration and displacement (energy) had to be considered. More recently, Scholl (Ref. 7) observed that the seismic energy demand typically increases substantially from say $T = 0.1$ to $T = 0.5$ while the response spectrum acceleration (force) remains somewhat constant in that period range.(See Fig. 2 for example) This reveals that a force analysis is not sufficient for evaluating the flexible body response of structures over the entire range of earthquake ground motion periods.

The damage, or lack of damage, sustained by structures during earthquakes is a generally accepted measure of the seismic performance of structures. Based on empirical observations and theoretical dynamic response considerations Scholl (Ref. 8) summarized three distinct procedures for estimating ground-motion caused damage to structures. On the basis of that experience it is clear that a significant indicator of damage is interstory drift. Damage in the vast array of buildings constructed is highly variable and is dependent on the structural and nonstructural detailing provided by the designer, but the following generalizations appear appropriate:

1. at $\gamma \simeq 0.001$, nonstructural damage is probable
2. at $\gamma \simeq 0.002$, nonstructural damage is likely
3. at $\gamma \simeq 0.007$, nonstructural damage is relatively certain and structrual damage is probable
4. at $\gamma \simeq 0.015$, nonstructural damage is certain and structural damage is likely

where γ = interstory drift strain = interstory drift ÷ story height

Thus both structural and nonstructural damage occur at relative small drifts. Nonstructural equipment damage in buildings is largely a function of acceleration in the building but is not pertinent to this paper.

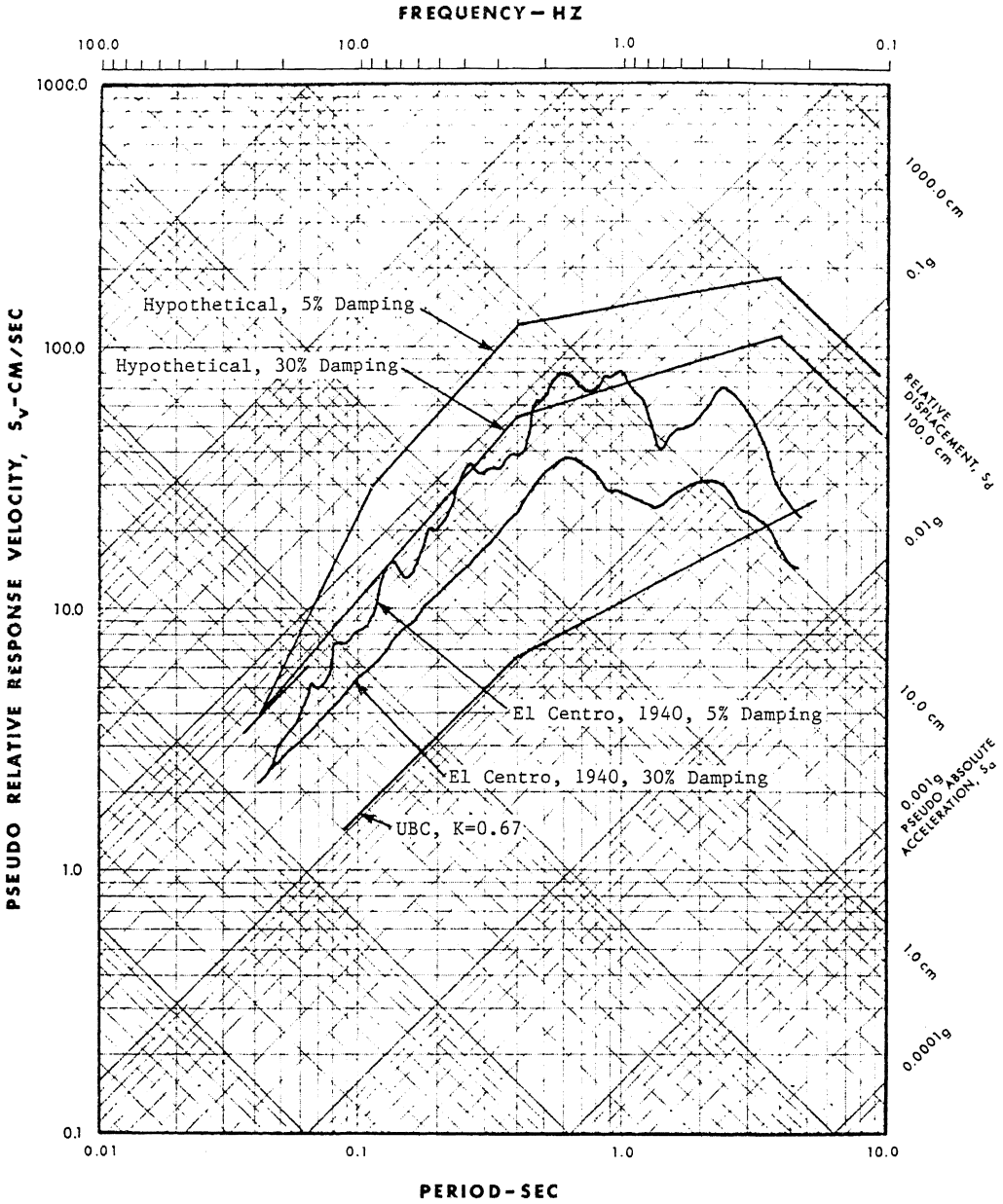


FIGURE 2 EXAMPLE RESPONSE SPECTRA

As pointed out by Housner (Ref. 4) and Blume (Ref. 5), the deflection or strain in a structure can be conveniently evaluated by equating the potential energy demand imposed by an earthquake and the potential or strain energy developed in a structure. For a simple single-degree-of-freedom system responding elastically, the potential energy in the structure is:

$$PE = (1/2)\ P\ \Delta \qquad (2)$$

where: PE = Potential Energy (in-Lb)
P = Force acting on the structure (Lb)
Δ = Strain deflection of the structure (in)

For a linear system stiffness ($K = P/\Delta$) can be substituted into Equation (2). Therefore:

$$PE = (1/2)\ K\ \Delta^2 \qquad (3)$$

From Eq. (3) it can be observed that there are two ways to increase the potential energy capacity: (1) increase stiffness, or (2) increase deflection. This observation is of paramount importance to the topic of this paper because judicious use of brace dampers in design can facilitate increasing both these quantities thereby substantially increasing the potential energy capacity.

For a simple elastic single-degree-of-freedom system, the potential energy demand (Eq. 1) can be equated to the potential energy (Eq. 3) and the displacement, Δ, can be readily determined. For elastic multi-degree-freedom systems, potential energy demand is distributed in accordance with modes and mode shapes (Ref. 7).

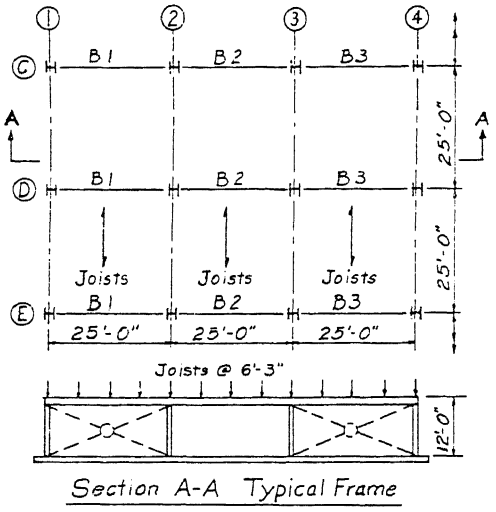
EXAMPLE EARTHQUAKE ANALYSIS

Example analyses of a simple one-story conventional building were performed to demonstrate the effectiveness of the Brace Damper. Figure 3 shows the plan and transverse section of the structure configuration. The structure was designed to resist earthquake forces in the transverse direction assuming three different systems for resisting lateral forces as follows:

- Ductile Moment Resisting Space Frame
- Concentric Braced Frame
- Brace Damper Frame

Each system was designed using lateral force provisions of the Uniform Building Code (UBC) and steel design specifications prescribed in the American Iron and Steel Institute's Manual of Steel Construction. Steel with a yield strength of 36,000 psi was assumed.

Because the objective of the example analyses was to demonstrate the effect of various lateral force resisting systems, it was assumed that the roof beams were infinitely rigid. This assumption produces structures that are laterally stiffer than typical structures, but this assumption makes hand parameter-study calculations feasible and conclusions regarding the earthquake performance of the various structural systems are not jeopardized. It should be noted, however, that the cost effectiveness of the various structural systems cannot be conclusively evaluated without considering the flexible beam design.



LOADS:

Dead Load - 20 psf

Live Load - 20 psf

Seismic Load - UBC Zone 4

$Z=1$, $I=1$, $S=1.2$
$C=1/15\sqrt{T} \leq 0.12$, $T=0.2N$

K=0.67 For Ductile Moment Frame
K=1.0 For Concentric Braced Frame
K=0.8 For Brace Damper Frame (Moment frame designed for 25% of V required)

Seismic Weight - Dead Load Only

FIGURE 3 LOADS AND DIMENSIONS - EXAMPLE BUILDING DESIGN

It was deemed desirable to achieve a total damping factor of 30%, with 25% from the brace damper and 5% from the basic structure. Two - 1.5 ton/cm dampers are required to achieve this. From Table 1, the damping coefficient (c) for a 1.5 ton/cm brace damper is 58.3 kg - sec/cm. Therefore, the damping for 2 - 1.5 ton/cm brace dampers is c = 326 Lb - sec/in. A standard assumption in earthquake engineering is that equivalent viscous damping is related as follows:

$$c = 2m\omega\lambda \quad \text{or} \quad \lambda = c/2m\omega \qquad (4)$$

where: λ = damping factor, c = damping coefficient (Lb - sec/in), m = mass (Lb - sec^2/in), and ω = angular frequency (rad/sec)

Using c = 326 Lb- sec/in and T = 0.46 sec from Table 2, Equation 4 gives an equivalent viscous damping value for the brace dampers of λ = 0.25

Table 2 gives the selected column size for the design lateral force, calculated response natural period (T), design base shear coefficient (C_b), base shear coefficient at yield (C_{by}), yield deflection (Δ_y) and yield potential energy (PE_y) for each of the three lateral force resisting systems considered. The calculated periods are for bare frame only and no nonstructural material participation is considered. Drift governed the design of the ductile moment frame and the P-Δ moment was considered. Comparing the lateral force resistance characteristics of the ductile moment frame and the concentric braced frame it is seen that, as expected, the latter is more seismically vulnerable. The extreme seismic brittleness of the concentric braced frame is most vividly revealed through comparing the yeild potential energy (PE_y) values for the three systems. The brace damper frame has a PE_y value more that 12 times that of the concentric braced frame -- and almost the same size columns are needed for the two systems. The large PE_y value for the brace damper frame is attributable to the large yield deformation feasible with relatively large stiffness. These combined characteristics are not feasible with a ductile moment resisting space frame.

TABLE 2
EXAMPLE ONE-STORY BUILDING DESIGN: LATERAL RESISTANCE CHARACTERISTICS

FRAMING SYSTEM	DESIGN COLUMNS			LATERAL FORCE-RESPONSE CHARACTERISTICS						
	Column	Size (inches)	Weight (Lb/ft)	Lateral Stiffness (k/in)	Calculated Period (sec)	UBC Design K factor	Design C_b	Δ_y (in)	C_{by}	PE_y (in-k)
DUCTILE MOMENT SPACE FRAME	Interior	6x6x1/4 Tube	18.8	11.2	0.58	0.67	.097	1.06	0.32	6.29
	Exterior	5x5x1/4 Tube	15.4							
CONCENTRIC BRACED FRAME	Interior	4 ϕ	10.8	23.5	0.40	1.0	.144	0.36	0.23	1.52
	Exterior	3 ϕ	7.6							
FRAME WITH BRACE DAMPER	Interior	4 ϕ	10.8	18.1	0.46	0.8*	.115*	1.47	0.71	19.6
	Exterior	3 1/2 ϕ	9.1							

* Moment frame designed for 25% of base shear required for K-factor=0.8
Stiffness of structure with Brace Dampers is 2.4 times that required for drift for K-factor=0.8

TABLE 3
EXAMPLE ONE-STORY BUILDING DESIGN: DEMAND-CAPACITY COMPARISON

RESPONSE SPECTRUM			STRUCTURE RESPONSE DEMAND			STRUCTURAL SYSTEM								
						DUCTILE FRAME			BRACED FRAME			BRACE DAMPER		
EARTHQUAKE	DAMPING (%)	PERIOD (sec)	S_a/g	S_v (in/sec)	PE_D (in-k)	S_a/g / C_b	S_a/g / C_{by}	PE_D / PE_y	S_a/g / C_b	S_a/g / C_{by}	PE_D / PE_y	S_a/g / C_b	S_a/g / C_{by}	PE_D / PE_y
EL CENTRO 1940 (N-S)	5	0.2	0.67	8.27	3.32	6.91	2.09	0.53	4.65	2.91	2.18			
		0.40	0.61	15.0	10.9				4.24	2.65	7.17			
		0.58	0.91	30.7	45.8	9.38	2.84	7.28						
	30	0.2	0.37	4.50	0.98							3.22	.52	0.05
		0.46	0.43	12.0	6.99							3.74	.61	0.36
HYPOTHETICAL	5	0.2	1.8	22.1	23.7	18.6	5.62	3.77	12.5	7.83	15.6			
		0.40	1.9	46.6	105.5				13.2	8.26	69.4			
		0.58	1.4	47.2	108.2	14.4	4.38	17.20						
	30	0.2	0.79	9.71	4.58							6.87	1.11	0.24
		0.46	0.78	22.1	23.7							6.78	1.09	1.21

For each of the three structure systems, Table 3 gives: 1) the demand spectral values from Fig. 2, 2) the response spectrum acceleration divided by base shear coefficient ($S_a/g \div C_b$), 3) the response spectrum acceleration divided by the yield base shear coefficient ($S_a/g \div C_{by}$), and 4) the demand potential energy divided by the yield potential energy ($PE_D \div PE_y$) for a single transverse bay. On the basis of this analysis it can be concluded that the postelastic response demand is greatest for the concentric braced frame and least for the brace-damper frame. Postyield response, inferring structural damage, can be expected for the ductile moment frame and for the concentric braced frame for both the El Centro earthquake and the hypothetical earthquake. Conversely, the brace-damper frame would be expected to respond elastically for the El Centro earthquake and sustain ony slight postyield response during the high-level hypothetical earthquake. With the PE_D and PE_y values in Table 3, inelastic response ductilities can be estimated using the procedure described by Blume (Ref. 5). If two 2-ton/cm dampers were used, the brace-damper frame would not be expected to yield for even the high amplitude hypothetical earthquake spectrum.

CONCLUSIONS

Results of the example earthquake design analyses given in the above section show that the brace-damper system can be beneficially used in construction to improve substantially the earthquake performance of structures. Results of other preliminary analyses conducted thus far indicate that brace dampers can be similarly effectively used in multi-story buildings. The improved earthquake performance characteristics of structures incorporating brace dampers results from a combination of the following factors: 1) reduced earthquake demand forces brought about by the high damping in the brace damper, 2) increased lateral stiffness achieved by the stiffness built into the brace dampers, and 3) increased structural yield deformation which is facilitated because the gravity-load carrying moment frame does not have to resist the lateral forces. The benefits of a dual structural system were generally recognized following the great 1906 San Francisco earthquake. Using the deformable brace damper in the dual structural system makes the system perform even better.

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