SOME DAMAGE CONTROL CRITERIA
FOR A STEEL BUILDING WITH ADDED HYSTERESIS DAMPER

ASTUSHI WATANABE

Nippon Steel Corp.,
2-6-3, Otemachi, Chiyoda-ku, Tokyo, 100-71, Japan
Tel +81-3-3242-4111, Fax: +81-3-3275-5963
(Currently Doctoral Student, Department of Civil Engineering, Lehigh University,
B154, 117 ATLSS Drive, H-building, Bethlehem, PA 18015. U.S.A.
Tel (215) 758-6112, Fax: (215) 758-6250, e-mail: atw2@lehigh.edu)

Paper No. 449. (quote when citing this article)
Eleventh World Conference on Earthquake Engineering
ISBN: 0 08 042822 3
11 WCEE

ABSTRACT

This paper is based on and supports a new design concept and methodology for a new type of steel structure that significantly reduces damage to the building (Wada *et al.*,1992; Kasai,1996). The basic concept is to concentrate the earthquake energy on the special brace members which have stiffness as well as significant energy dissipation capacity; thereby, maintaining the beam and columns of the main structure elastic or undamaged. To realize this concept in the first process of design using elasto-plastic braces, the author has obtained closed-form relationship between maximum base shear and maximum displacement in terms of brace member ductility demand, the stiffness of brace and building ,and post yield stiffness ratio of the building to the elastic stiffness. This relationship is developed based on Damped Spectrum method and Energy Spectrum method. A series of inelastic dynamic response analyses of single-degree-of-freedom systems were computed and compared with the predictions by these methods. Also, a control method on residual deformation of the building is described.

KEYWORDS

Damage Control, Hysteresis Damper, Equivalent Damping, Equivalent Period, Residual Displacement, Strength Demands, Ductility Demands, Post Stiffness Ratio

INTRODUCTION

Damage cost from the recent earthquakes in California, Kobe, and other metropolitan areas has exceeded the social tolerance. Many modern buildings suffered damage and required substantial repair work, although they saved human lives. The current seismic design philosophy allowing inelastic deformation of structural members to dissipate seismic energy was considered to be acceptable, but it is now being questioned seriously. Therefore it is necessary to formulate a new seismic design concept that decreases damage to the buildings and components.

This paper is based on and supplements the concept of Damage Tolerant Structure (Wada, *et al.*, 1992) and Damage-Free Structure (Kasai, 1996). The basic concept of these damage control design methods is to concentrate the earthquake energy on the special ductile braces (Watanabe *et al.*, 1988) or visco-elastic dampers (Kasai, *et al.*, 1995), having significant energy dissipation capacity; thereby, maintaining the beam and columns of the main structure elastic or undamaged. A building designed with this concept can be used continuously after an earthquake by simply replacing the brace members, should they be damaged from the seismic event.

Many current seismic codes are based on elastic-plastic design that enables buildings to survive severe earthquakes with a small design base shear. How much the yield level capacity of the structure can be reduced within the limits of a predefined ductility ratio, is the concern of many structural engineers, since design yield level mainly governs the total steel weight of the structure. But extremely reduced design base shear can lead to large story displacement ductility demand on the building which may result in the building damage. Good design can be found in the balance of strength and ductility demands.

Newmark and Hall (1973) proposed $R_\mu=\mu$ for building having a moderate to long vibration period T and $R_\mu=\sqrt{2\mu-1}$ for a short vibration period T, using the perfect elasto-plastic systems, where R_μ is strength reduction factor and μ is displacement ductility. Nassar and Krawinkler (1991) developed refined formula using three parameters μ, T, and post yield stiffness to elastic stiffness ratio p. But most of existing studies considers small p ($0\leq p\leq 0.1$) except that by Ishimaru (1995), although, the effect of p to the behavior is significant. Ishimaru proposed the equivalent velocity spectrum for combination of viscous and hysteresis systems with a variety of p, which predict structural performance.

For damage control design purposes, the maximum response level is as important as the yield level, since damage depends on the maximum level of the response. Note that p indicates the ratio of stiffness of the frame structure stiffness to total structure stiffness in this study *i.e.;*

$$p = K_f / (K_a+K_f), \qquad 1/p = 1 + K_a / K_f \qquad (1)$$

where K_a= added stiffness of elasto-plastic brace, K_f= frame stiffness. It is essential to develop base shear and displacement relations to minimize both of them for optimum design in terms of parameters p and μ (or K_a/K_f, see Fig.1). Special ductile braces for required stiffness ratio K_a/K_f and ductility demand μ can be designed and realized easily by using a variety of yield strength steels and configurations (Wada *et al.*, 1992). Also, using high strength steel to columns and beams keeps the frame structure elastic until the inter-story displacement angle of at least 1/100 (Watanabe *et al.*, 1993).

SEISMIC DESIGN BASE SHEAR FORCE AND DESIGN DISPLACEMENT

The purpose in this section is to propose and compare two methods to predict the relationship between maximum base shear normalized to the shear obtained from elastic analysis, $V_{max}/V_{max,el}$ and maximum displacement normalized to the displacement obtained from elastic analysis, $u_{max}/u_{max,el}$. This relationship is explicitly described in terms of p, μ, and T without dynamic nonlinear analysis. In this paper only the system with hysteresis damper is applied. Visco-elastic or viscous damper is not considered here.

Damped Spectrum Method

The Damped Spectrum method predicts inelastic peak responses based on the elastic response of the structure including three factors. The first factor is the period shifting effect, due to softening of the brace caused by yielding. The second factor is the spectrum shape effect which becomes important when period shifting occurs. The third factor is the damping effect caused by the energy dissipation of elasto-plastic braces.

The period shifting effect is obtained by calculating equivalent period T* for the maximum inelastic cycle of the bilinear system (AASHTO,1992).

$$T^* / T = \sqrt{\mu / (1 + p\mu - p)}, \qquad (\mu\geq 1) \qquad (2)$$

The equivalent pseudo-velocity spectrum $S_{pv}*(\xi,T*)$ is obtained by averaging the pseudo-velocity spectrum $S_{pv}(\xi,\tau)$ over the range of $T \leq \tau \leq T*$.

$$S_{pv}*(\xi,T*) = \left[\int_T^{T*} S_{pv}(\xi,\tau)\,(d\tau) \right] / (T* - T) \tag{3}$$

Note that ξ is the damping ratio when the building is elastic, and it is considered to be 2% in the present study unless noted. The spectrum shape effect is defined as $S_{pv}*(\xi,T*)/S_{pv}(\xi,T)$. The energy dissipation of the bilinear system is considered as an equivalent damping ratio, for the maximum inelastic cycle of the bilinear loop (AASHTO,1992), with damping ratio ξ. In reality the seismic response of the structure is composed of many small cycles and medium cycles. Newmark *et al.* (1971) and Iwan (1979) developed the average equivalent damping ratio $\xi*$, *i.e.*;

$$\xi* = \xi/\mu + (1/\mu) \int_1^{\mu} [\xi\,(T*/T) + 2(1-p)(\mu-1) / \{\pi\mu(1 + p(\mu - 1))\}]\, d\mu \qquad (\mu \geq 1) \tag{4}$$

The result after integration of Eq.4 is shown in Fig.2. Further, displacement reduction factor $D_{\xi*}$ in terms of $\xi*$ is obtained (Kasai *et al.*, 1995) by considering an equation in AIJ (1993) as well as the table given in the new version of NEHRP (1994). It is interesting to know that the same $D_{\xi*}$ can be obtained by modifying the Energy Spectrum method (see Appendix).

$$D_{\xi*} = 1.5 / \sqrt{1 + 25\xi*} \tag{5}$$

The original design basis earthquake spectrum is based on ξ=5% (NEHRP), for which $D_{\xi*}$ is 1.0. Eq.5 is approximately equal to upper bound and 5-10% larger than the mean value of $D_{\xi*}$ by Ashour (1987). The $V_{max}/V_{max,el}$ and $u_{max}/u_{max,el}$ relations are obtained in terms of $T*/T$, $D_{\xi*}$, and $S_{pv}*(\xi,T*)/S_{pv}(\xi,T)$, *i.e.*;

$$V_{max}/V_{max,el} = D_{\xi*}\,(T*/T)\,[S_{pv}*(\xi,T*) / S_{pv}(\xi,T)], \quad u_{max}/u_{max,el} = D_{\xi*}\,(T/T*)\,[S_{pv}*(\xi,T*) / S_{pv}(\xi,T)] \tag{6}$$

Fig.3 shows when K_f+K_a and S_a are constant. Fig.4 shows when K_f+K_a and S_{pv} are constant. Fig.5 shows when K_f and S_{pv} are constant. In a design process , Fig.3 and 4 are useful when the design under constant period is performed, and Fig.5 is useful when design under the constant K_f is performed. Fig.6 shows examples of hystereses with p=0.2 (*i.e.* K_a/K_f=4) and S_{pv}=constant. A discussion on the results will be presented in the following section in order to compare the results by dynamic analysis.

Energy Spectrum Method

Housner (1956) made a quantitative evaluation of the total amount of energy input that contributes to the building's responses by using of the velocity response spectra of the elastic system. He assumed that the energy input responsible for the damage in the elastic-plastic system is identical to the energy input of the elastic system, *i.e.*;

$$W_p + W_e \leq E(\text{input}) = M\,S_v^2 / 2 \tag{7}$$

where, E(input) = energy input attributable to damage as defined by Housner, S_v= the maximum velocity response of the elastic systems, W_p= the cumulated plastic strain energy, W_e = the kinetic energy and the elastic strain energy constituting the elastic vibrational energy, M= total mass of the structure. Housner made conservative estimate to $W_p + W_e$ by equating it to E(input).

Akiyama (1985,1993) considered Housner's theory and proposed that the input energy depends on the periods and the total weight of the building in the velocity constant portion. He used elasto-plastic model to estimate W_p and W_e with assumption that the cumulative plastic deformation ($u_{p,cum}$) can be approximately related to the maximum displacement (u_{max}) as following.

$$u_{max} = u_{p,cum} / 8 + 1 \tag{8}$$

$$W_e = (1/2)\, u_{max}^2\, K_f\,, \qquad W_p = 8\, u_{max}\, u_y\, K_a \tag{9}$$

Eq.8 agrees well with the results by Inoue (1995). The author applied these considering elastic energy of hysteresis damper. For bilinear system with K_a, u_y, K_f, and μ, where u_y is yield displacement (see Fig.1). W_e and W_p are expresses as:

$$W_e = (1/2)\, \mu^2\, u_y^2\, K_f\,, \qquad W_p = 8\, (\mu - 1)\, u_y^2\, K_a \tag{10}$$

From Eqs.7 and 10,

$$V_{max}/V_{max,el} = \{1 + p\,(\mu - 1)\} / \sqrt{p\,\{\mu^2 + 16\,(\mu - 1)\,(1 - p) / p\}} \tag{11}$$

$$u_{max}/u_{max,el} = \mu / \sqrt{p\,\{\mu^2 + 16\,(\mu - 1)\,(1 - p) / p\}} \tag{12}$$

Figs.7 and 8 show Eqs.11 and 12 for K_f=constant and K_a+K_f=constant, respectively. Note that these results are based on the constant velocity portion. A discussion on the results follows in the next section.

Statistical Study on $V_{max}/V_{max,el}$ and $u_{max}/u_{max,el}$ with SDOF Non-Linear Analyses

A series of inelastic response analyses were conducted to obtain normalized maximum base shear ($V_{max}/V_{max,el}$) and normalized maximum displacement ($u_{max}/u_{max,el}$) relations of the SDOF system. The nonlinear dynamic time history analyses were performed for the following 2,040 permutations: (1) Six different post yielding stiffness ratios, p=0.01, 0.1, 0.2, 0.3, 0.4 and 0.5. (2) Displacement ductility ratios μ=2,3,4,5, and 6. (3) Seventeen earthquake motions comprising 8 earthquake motions (Table 1) and 9 artificial motions that are based on design basis earthquake spectra for 3 different soil categories. (4) Four different periods, T=0.25, 0.5, 1.0, and 2.0 (sec), where T=0.25 belongs to constant acceleration spectrum, and T=0.5, 1.0, and 2.0 to constant velocity spectrum.

Figs.10 and 11 show the mean spectra of 17 motions. Each motion is normalized to 0.4G for Peak Ground Acceleration. Dimensionless demand parameters become independent of the ground motion severity (scale). For all analyses, the bilinear hysteresis model with 2% critical damping was used. Fig.12 to 15 show the mean maximum inelastic response with μ=2 to 6 for p=0.01 to 0.5 under the condition of constant magnitude of K_a+K_f. The following observations can be made from these figures;

(1) The $V_{max}/V_{max,el}$ vs. $u_{max}/u_{max,el}$ relations appear to be similar for all case of T=0.5, 1.0 and 2.0 (Fig. 13 to 15). This indicates the relations are independent of T as long as it is in the range of constant velocity portion of spectrum. The smallest $u_{max}/u_{max,el}$ exists and it is approximately 0.7 when μ is between 2 and 3. For the same p, the smallest $V_{max}/V_{max,el}$ occurs at μ larger than 5.

(2) The normalized yield strength demand, $V_y/V_{max,el}$ depends strongly on μ. It is independent of p value in the range from p=0.1 to 0.5. It also increases when p is 0.01 by 10% approximately (Figs.16, 17).

(3) The results of $V_{max}/V_{max,el}$ of both Damped Spectrum method and Energy Spectrum method correspond well to the results from numerical analyses (Figs 3, 4, 7, 12 to 15), though two methods are based on different procedures assumptions. $u_{max}/u_{max,el}$ by Damped Spectrum method is generally 5-10% larger than those by numerical analyses due to conservative estimation of $D_{\xi*}$. $u_{max}/u_{max,el}$ by Energy Spectrum method is 5-20% smaller than those by numerical analyses.

(4) The coefficient of variation of $u_{max}/u_{el,max}$ is approximately 0.25, which is independent of μ, p, and T, though it is not shown in this paper.

STATISTICAL STUDY OF RESIDUAL DISPLACEMENT DUCTILITY WITH SDOF NON-LINEAR ANALYSES

The work explained above was also extended to study statistically on the residual displacement ductility μ_{res} of the SDOF systems. It was obtained by analyzing the system for sufficient duration after termination of earthquake motion analyses. In all cases, the average of μ_{res} by 17 motions are less than 0.1, which are very small. Fig.18 shows the distribution of the μ_{res} of 17 earthquake data for p=0.3 as an example. A Gaussian distribution with a mean of zero, is assumed to represent the scattered data points of μ_{res}. Fig.19 shows the standard deviation of the residual displacement ductility $S(\mu_{res})$, which covers 68% probability. The author proposes a 2-parameter equation of $S(\mu_{res})$ for the bilinear systems in the following form.

$$S(\mu_{res}) = \left\{0.6\ (\mu - 1)\ (1 - 2p)\ \exp[-\sqrt{3.2\ p\ (\mu - 1)}] + 0.8\ p\ \right\}\ (1 - p)^2 \tag{13}$$

where, $2 \leq \mu \leq 6$,and $0.01 \leq p \leq 0.5$. The $S(\mu_{res})$ decreases with the increase of p. It is interesting to know that when p is larger than 0.2, $S(\mu_{res})$ is less than 0.25 for all the μ's ($2 \leq \mu \leq 6$) considered in the present study. Since yield inter-story displacement of ordinal steel structure is less than 20mm, residual ductility inter-story displacement may be less than 5mm with 68% probability, so far as p is larger than 0.2.

In reality, the hysteresis of steel structure is not a bilinear system. As the displacement increases, more members yield, and building's post yield stiffness decreases. Also, due to Bauschinger effect, through the reversed loading, stress-strain curve displays smaller stress than that of highly idealized bilinear systems. The real residual displacement should be smaller than the result of this study, due to these effects.

CONCLUSION

This paper has presented the trends of maximum base shear and maximum displacement with respect to the elastic response of a steel structure having added hysteresis damper. Obtained trends are predicted based on two distinct theories and are confirmed by numerical simulations. Also, the residual displacement ductility was predicted based on a statistical method. In this limited study, value of post yield stiffness ratio between 0.2 and 0.3, displacement ductility μ between 3 and 5 are recommended to minimize the maximum base shear, maximum displacement, and residual displacement. In the preliminary design process, the frame structure can be designed elastic based on K_f and u_{max}, and hysteresis damper can be designed for stiffness and strength based on the required K_a and u_{max}/μ. It is useful, since the frame structure and hysteresis damper can be designed independently.

APPENDIX ($D_{\xi*}$:Reduction Factor due to Damping)

Consider two linear damped elastic systems with a same natural period and different damping ratios (ξ_1 and ξ_2). Applying Eq.7 and Akiyama's proposal to these elastic systems, Eqs.14 are obtained.

$$E_{1(input)} = W_{\xi 1} + W_{E1}\ , \qquad E_{2(input)} = W_{\xi 2} + W_{E2}\ , \qquad E_{1(input)} = E_{2(input)} \tag{14}$$

where,$W_{\xi 1}$and $W_{\xi 2}$ are energy dissipated by damping. Considering in Eq.9, assume that the dissipated energy by damping during earthquake is equal to the damping energy loss per two maximum cycles. $D_{\xi*}$, which is defined as u_1/u_2, (u_1 and u_2 are maximum displacement of each systems) is calculated using the following formula:

$$D_{\xi*} = \sqrt{\{(4\pi\xi_2 + 1/2)\ /\ (4\pi\xi_1 + 1/2)\}} \tag{15}$$

Normalizing to ξ_2=0.05, Eq.2.4 is obtained.

ACKNOWLEDGEMENTS

The author thanks Dr.K. Kasai (Lehigh Univ.) for valuable advice and suggestions, who was supposed to be a co-author, but his name is not listed so, due to the policy of the WCEE regarding number of papers allowed for submission.

REFERENCES

AASHTO (1992). In: Design Proceedings for Seismically Isolated Bridges, Dynamic Isolation Systems, Inc., Berkeley, California.

AIJ (1993). In:" Recommendations for the Design of Base Isolated Buildings", Architectural Institute of Japan (AIJ), Tokyo (in Japanese).

Ashour, S.A. (1987). "Elastic Seismic Response of Buildings with Supplemental Damping", Ph.D. Dissertation, Dept. of Michigan, Ann Arbor, MI.

Akiyama, H. (1985). In: "Earthquake -Resistant Limit-State Design for Buildings", Univ. of Tokyo Press.

Housner G.W. (1956). " Limit Design of Structures to Resist Earthquakes", 1st WCEE.

Inoue, K. (1995). "Seismic Design Using Hysteretic Damper",Proceeding of a New Direction in Seismic Design , Tokyo, 9-10 Oct. pp95-111.

Ishimaru, S. (1995). "Seismic Performance Design Method Based on Energy Concept", Proceeding of Symposium on A New Direction in Seismic Design, Tokyo, Japan, October.

Iwan, W.D. and N.C. Gates (1979). "Estimating Earthquake Response of Simple Hysteretic Structures", *Journal of the Engineering Mechanics Division,* ASCE, JUNE, pp.391-405

Kasai, K. and Y. Fu (1995). "Seismic Analysis and Design Using VE Dampers", Proceeding of a New Direction in Seismic Design , Tokyo, 9-10 Oct. pp113-140

Kasai, K. and Y. Fu (1996). "A Static Equivalent Force Design Method for a Damage Free-Building with Viscoelastic Dampers", 11th WCEE, June.

Nassar, A.A. and H. Krawinklar (1991). " Seismic Demands for SDOF and MDOF Systems", John A. Blume Earthquake Eng. Center Report No.95,Dept of Civil Eng., Stanford Univ., June

Newmark, N.M. and E. Rosenblueth (1971). In: "Fundamentals of Earthquake Engineering", Prentice Hall, Inc., Englewood Cliffs, NJ,

Newmark, N.M. and W.J. Hall (1973). "Seismic Design Criteria for Nuclear Reactor Facilities", Report No.46, Building Practices for Disaster Mitigation, National Bureau of Standards, U.S. Department of Commerce, pp209-236.

NEHRP (1991). In: "Recommended Provisions for the Development of Seismic Regulations for new Buildings", FEMA, Building Seismic Safety Council, Washington D.C.

Wada, A., A. Watanabe et. al. (1992). "Damage Tolerant Structures", Fifth US-Japan Workshop on the Improvement of Building Structural Design and Construction Procedures, ATC-15.

Watanabe, A. et al. (1988). "Properties of Brace Encased in Buckling-restraining Concrete and Steel Tube", Proceeding of 9'th WCEE, Tokyo-Kyoto, Vol.IV.

Watanabe, A. et al. (1993). "Study of Damage Tolerant Structue (Part 2,3 Experimental Study of High Strength Steel Frame and Ductile Brace with Low yield Strength Steel)", Summaries of Technical Papers of Annual Meeting, AIJ, Japan (in Japanese).