Seismic damage assessment of steel components

A. Benavent-Climent
Department of Structural Mechanics, University of Granada, Spain

Abstract

This paper discusses a new approach to the assessment of damage in structural steel components under cyclic inelastic loading stories of the type experienced in earthquakes. The approach is based on a new damage model proposed by the author. It is shown that the seismic performance of a steel component depends on two structural performance parameters: b and m. The parameter b is influenced by the material properties, the prevailing loading condition and the geometry of the steel component. The parameter m reflects the influence of the structural system, hysteretic behavior and characteristics of the earthquake. This approach takes into account that the cumulative damage—in addition to being affected by the total amount of dissipated energy, the maximum deformation and the number and amplitude of the cycles of deformation—is also path-dependent.
Keywords: damage model, fatigue, seismic damage, steel component.

1 Introduction

The great damage caused by recent earthquakes, such as Northridge (1994) or Kobe earthquakes (1995), has highlighted that code design for life safety does not protect adequately the structure adequately against damage; performance goals other than life safety (e.g. damage control) must be taken into account explicitly in the seismic design of new structures (performance-based seismic design). In the field of earthquake engineering, there is an increasing trend toward employing damage as a measure of seismic performance in the assessment and design of structures [1-3]. To this end, accurate damage models that take into account cumulative damage effects (cumulative models) are needed for quantifying realistically the expected damage under different design earthquakes. These models can also provide the basis for judging the safety of existing structures and reference for retrofit decision making.

WIT Transactions on Engineering Sciences, Vol 57,
www.witpress.com, ISSN 1743-3533 (on-line)
doi:10.2495/MC070391

The issue of structural damage characterization under seismic actions has received much attention, and several approaches have been proposed. One approach is based on the classical Manson–Coffin model for metallic materials [4, 5] and Miner's linear damage accumulation rule [6], which governs low-cycle fatigue behavior of metals [7-12]. Manson–Coffin's model postulates that, under constant amplitude cycling, the number of cycles to failure, N_f, can be related to the plastic deformation range of the cycle (in terms of strain, rotation, deflection etc.), $\Delta\delta_p$, by an equation of the type:

$$N_f = \frac{1}{C(\Delta\delta_p^c)} \tag{1}$$

where C and c are the structural performance parameters. The damage per cycle of amplitude $\Delta\delta_p$ is $1/N_f$. If the loading history consists of a sequence of $n_1,n_2,.., n_N$ closed cycles of different amplitudes $\Delta\delta_{p1},\Delta\delta_{p2}...,\ \Delta\delta_{pN}$, respectively, Miner's rule postulates that the accumulated damage, D, is:

$$D = \sum_{i=1}^{N} \frac{n_i}{N_{fi}} = C\sum_{i=1}^{N}[n_i(\Delta\delta_{pi}^c)]\cdot \tag{2}$$

Failure is predicted when D=1. If the response history consists of arbitrary individual excursions (i.e. in the case of a seismic response history), cumbersome cycle counting methods (i.e. the rain-flow method) must be applied to convert the individual excursions into a sequence of closed cycles of constant amplitude so that Eq. (2) can be applied. This is one of the shortcomings of this approach.

Another approach accounts for damage as a combination of maximum deformation and dissipated energy [13-16], and here the model proposed by Park and Ang [14] is the most widely used. Park and Ang's model defines the damage index D by:

$$D = \frac{\mu_{\mathrm{max}}}{\mu_{\mathrm{u}}} + \frac{\beta W}{Q_{\mathrm{y}}\,\delta_{\mathrm{u}}}, \tag{3}$$

where $\mu_{\max}$ (=$\delta_{\max}/\delta_y$) is the maximum deformation $\delta_{\max}$ normalized with respect to the yield deformation, δ_y; μ_u (=δ_u/δ_y) is the normalized ultimate deformation δ_u under monotonic loading; W is the dissipated hysteretic energy; Q_y is the yield strength; and β is a parameter characterizing the damage contribution due to cumulative plastic strain energy. In this model, μ_u and β can be viewed as the basic structural performance parameters. This model also presents important shortcomings: (1) the methodology for determining the "key" parameter β is not well stated; (2) failure is not identified by a single value of D; and (3) the model assumes that the response of the structure up to the limit state is path independent, that is, not influenced by the distribution of the plastic cycles during the deformation history.

The shortcomings mentioned above endanger the consistency and reliability of these methods in predicting the level of damage and the closeness to failure of a structural steel component subjected to seismic actions. This paper discusses a different approach for assessing the structural performance of steel components subjected to arbitrarily applied stress reversals, such those induced by earthquakes. This approach is based on a new energy-based damage model proposed by Benavent-Climent [17]. In this model, structural performance is governed by two parameters, b and m. The former, b, depends on the material properties, the prevailing loading condition and the geometry of the steel component. The latter, m, is influenced by factors such as the structural system, the hysteretic behavior, and the characteristics of the earthquake. In contrast to existing methods, which consider damage as a combination of the total amount of dissipated energy and maximum deformation, the proposed model represents damage as a combination of: (a) the total dissipated energy and, (b) the portion of the total dissipated energy consumed on the skeleton part of the load-displacement curve. This paper shows that the model can easily be extended to other types of prevailing stress conditions. The model is intended to be used either for quantifying the level of damage in performance-based seismic design of new structures, or for evaluating the safety of existing buildings and establishing a framework for making decisions about seismic retrofitting.

2 Damage model

The load-displacement, Q-δ, curve of a steel component subjected to an arbitrarily changing history of deformation can be decomposed, in each domain of loading, into three parts: the skeleton part, the Bauschinger part and the unloading part [18]. The skeleton part, Q-${}_S\delta$, is constructed by connecting sequentially each loading path that exceeds the load level attained in preceding cycles in the same loading domain. The Bauschinger part, Q-${}_B\delta$, begins at Q=0 and terminates at the maximum load level previously attained in preceding cycles in the same loading domain. The rest of the curve is the unloading part. Fig 1 shows an example of decomposition for a steel component subjected to constant amplitude flexure deformations.

Akiyama et al. [19] and Benavent-Climent et al. [20] applied the concept of this decomposition to investigate experimentally the ultimate energy dissipation capacity (UEDC) of structural steel components subjected to forced flexure and shear cyclic deformations in the plastic range. In total, 49 round steel rods and 10 rectangular steel plates with slits were subjected to bending and shear under statically applied cyclic loads up to failure. Each Q-δ curve obtained from the tests was decomposed into the skeleton and the Bauschinger parts as explained above. The skeleton part was approximated by the trilinear curve shown with dotted lines in Fig. 1b, which is defined by the yielding load, Q_y, the yielding displacement, δ_y, the plastic stiffness K_{p1} and K_{p2}, and the load Q_B that determines the transition point from K_{p1} to K_{p2}. The plastic strain energy dissipated in the positive and negative loading domains in the skeleton part, ${}_SW_u^+$

www.witpress.com, ISSN 1743-3533 (on-line)

and ${}_SW_u^-$, and in the Bauschinger part, ${}_BW_u^+$ and ${}_BW_u^-$, was computed and expressed in a non-dimensional form as follows:

$$ {}_S\bar{\eta}^+ = \frac{{}_SW_u^+}{Q_y\delta_y} \;;\; {}_S\bar{\eta}^- = \frac{{}_SW_u^-}{Q_y\delta_y} \;;\; {}_B\bar{\eta}^+ = \frac{{}_BW_u^+}{Q_y\delta_y} \;;\; {}_B\bar{\eta}^- = \frac{{}_BW_u^-}{Q_y\delta_y} \quad (4)$$

Next, the total plastic strain energy dissipated in the positive and negative load domain up to failure was expressed by the following ratios:

$$ \bar{\eta}^+ = {}_S\bar{\eta}^+ + {}_B\bar{\eta}^+ \quad ; \quad \bar{\eta}^- = {}_S\bar{\eta}^- + {}_B\bar{\eta}^- \quad (5)$$

From the results of the tests, it was concluded that at the ultimate state, and for each domain of loading, ${}_S\bar{\eta}$ and $\bar{\eta}$ can be related by the following expressions:

$$ {}_S\bar{\eta} \le \frac{\tau_B^2 - 1}{2k_{p1}} : \qquad \bar{\eta} = {}_S\bar{\eta} - \frac{7.33}{k_{p1}}\left[\sqrt{2k_{p1}\,{}_S\bar{\eta} + 1} - 1\right] + 0.5b \quad (6)$$

$$ {}_S\bar{\eta} > \frac{\tau_B^2 - 1}{2k_{p1}} : \; \bar{\eta} = {}_S\bar{\eta} - \frac{7.33}{k_{p2}}\left[\sqrt{2k_{p2}\,{}_S\bar{\eta} + \tau_B^2 - \frac{k_{p2}}{k_{p1}}(\tau_B^2 - 1)} - \tau_B + \frac{k_{p2}}{k_{p1}}(\tau_B - 1)\right] + 0.5b \quad (7)$$

where $k_{p1}=K_{p1}/K_e$, $k_{p2}=K_{p2}/K_e$, $K_e=Q_y/\delta_y$, $\tau_B=Q_B/Q_y$ and b is a non-dimensional coefficient that depends on the type of steel, the loading condition (flexure, shear etc.) and the geometry of the steel component. Q_y, δ_y, τ_B, k_{p1} and k_{p2} can easily be predicted analytically from the stress-strain relationship of the steel. b is the "key" structural performance parameter governing the UEDC of the steel component. It can be obtained by testing one specimen under constant amplitude cyclic loading [19] and is very sensitive to the detailed shape of the steel component in the region of plastic deformations.

For the purposes of illustration, Eqs.(6) and (7) are drawn with dotted lines in Fig. 2 in the normalized $\bar{\eta}$ versus ${}_S\bar{\eta}$ space, for mild steel rods subjected to flexure. It is worth noting in Eqs.(6) and (7) that the total amount of plastic strain energy that the steel component can dissipate up to collapse, $\bar{\eta}$, depends on the amount of energy consumed on the skeleton part ${}_S\bar{\eta}$, rather than on the maximum deformation δ_{max}. To clarify this point, let us consider for example a mild steel rod subjected to flexural cyclic deformations up to a given instant t_i, at which it reaches the maximum deformation in the positive domain δ^+_{max} for the first time. The condition of the rod at $t=t_i$ can be represented by a point with coordinates (${}_S\bar{\eta}_i$, $\bar{\eta}_i$) in the $\bar{\eta}$ versus ${}_S\bar{\eta}$ space shown in Fig. 2. Let us consider that after this instant t_i, the rod is forced to undergo more cycles of plastic deformation with amplitude less than or equal to δ^+_{max} in the positive load domain. The total amount of energy that the rod can dissipate in this domain until failure, $\bar{\eta}_u$, varies according to the path followed by the rod from $t=t_i$ in the $\bar{\eta}$ versus ${}_S\bar{\eta}$, space. That is, if the path is such that the energy consumed on the skeleton part, ${}_S\bar{\eta}$, does not increase in the positive load domain (i.e. path ① in Fig.2), the rod will fail when $\bar{\eta} = \bar{\eta}_{u,1}$; otherwise failure will occur for a value of $\bar{\eta}$, smaller than $\bar{\eta}_{u,1}$. The minimum value that $\bar{\eta}$ can attain is $\bar{\eta}_{u,2}$ (i.e. the path ② in Fig.2), which corresponds to the situation in which after instant t_i the rod continues deforming monotonically in the positive domain of loading, thus consuming only the skeleton part. The intermediate situation between path ① and path ② is represented by path③, in which after instant t_i the rod continues deforming cyclically and consuming energy in both the skeleton and the Bauschinger parts.

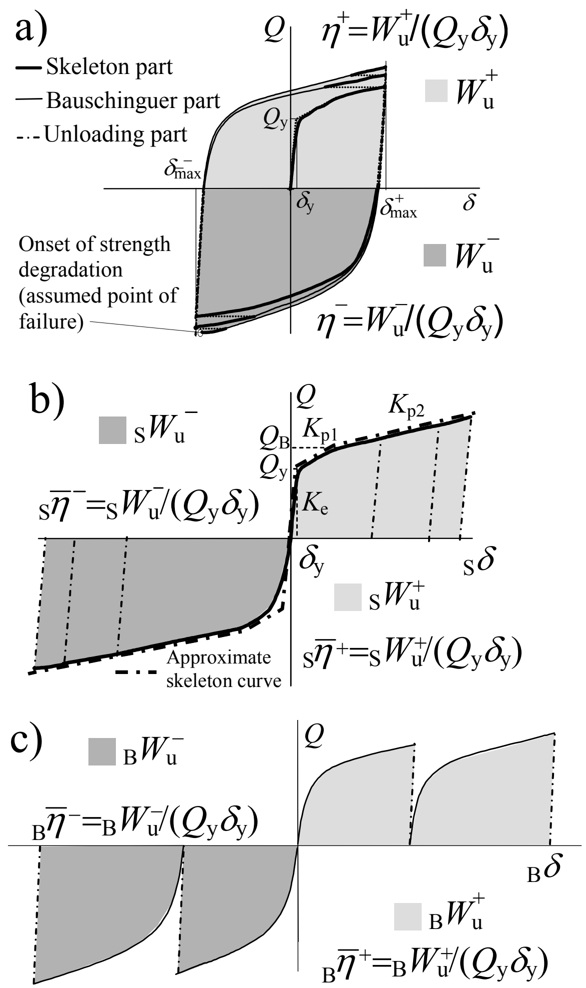

Figure 1: Decomposition of the load-displacement curve: a) original curve; b) skeleton part; c) Bauschinger part.

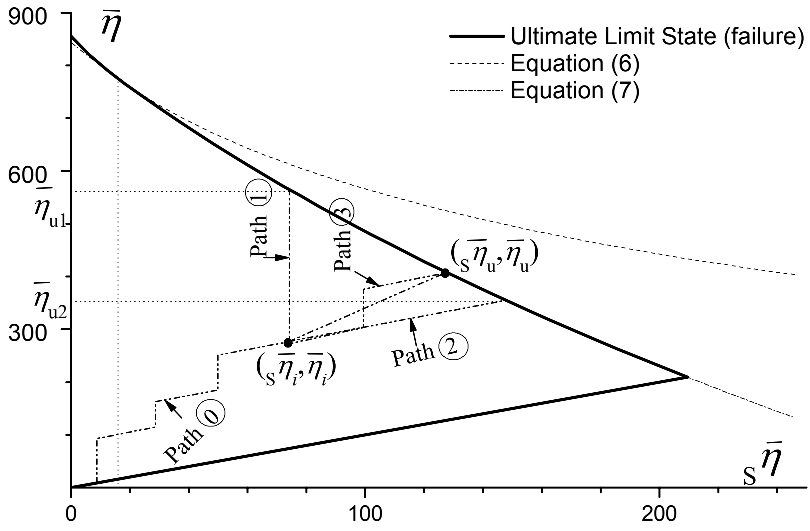

Figure 2: Energy consumption path of the steel component.

Path ③ represents the general situation of a steel component with stable hysteretic response like that shown in Fig. 1, subjected to seismic loadings. It is important to point out that along this path ③, due to the strain hardening effect of the steel, the maximum load attained by the rod in a given loading domain for the first time that it reaches the maximum displacement, δ_{max}, can increase in successive cycles even if δ_{max} is not exceeded. As these increments of maximum load in successive cycles involve additional consumption of energy on the skeleton part, it follows that ${}_S\bar{\eta}$ can increase even if the maximum deformation δ_{max} is not exceeded. As stated by Eqs.(6) and (7), the UEDC of the member $\bar{\eta}$ decreases as ${}_S\bar{\eta}$ increases. In other words, the UEDC of the steel component depends on ${}_S\bar{\eta}$ rather than on δ_{max}. This experimentally demonstrated fact is contrary to the assumption implicit in the Park and Ang model, which holds that if the structure is subjected to the same normalized maximum displacement (μ_{max}/μ_u), it will dissipate the same amount of normalized plastic strain energy ($\beta W/Q_y\delta_u$). This is the case irrespective of the portion of this energy that is consumed on the skeleton part, ${}_S\bar{\eta}$, which depends on the path followed by the member up to collapse. This path-independent characterization of ultimate state implicitly assumed by the Park and Ang and other damage models constitutes an important shortcoming, as mentioned in the introductory section of this paper.

On the basis of this experimental background, Benavent-Climent [17] proposed a new model that defines the damage index of the steel component at a given stage i (prior to failure) characterized by (${}_S\bar{\eta}_i$, $\bar{\eta}_i$) as follows:

$$\mathrm{ID}_i = \max\{\overline{\mathrm{ID}}_i^+, \overline{\mathrm{ID}}_i^-\} \tag{8}$$

where $\overline{\mathrm{ID}}_i^+$ and $\overline{\mathrm{ID}}_i^-$ are the index of damage in the positive and negative domains defined by:

$$\overline{\mathrm{ID}}_i = \frac{\overline{\eta}_i}{\overline{\eta}_\mathrm{u}} \tag{9}$$

This index measures the level of damage between 0 (no damage) and 1 (failure). As explained above, the value of $\overline{\eta}_\mathrm{u}$ at a given stage i depends on how the energy dissipated by the steel component is distributed between the skeleton part and the Bauschinger part. The distribution of energy between these two parts changes through the entire duration of the response and is strongly influenced by the structural system and the characteristics of the earthquake. This makes the prediction of $\overline{\eta}_\mathrm{u}$ for design purposes a cumbersome problem that is addressed in next section.

3 Energy demand for performance assessment

A seismic damage assessment based on the model given by Eqs. (5) and (6) requires information on the energy demand ratio $\overline{\eta}/{}_S\overline{\eta}$ imposed by an earthquake on a steel component. Previous research [21, 22] showed that this ratio is influenced by parameters such as the properties of the structural system (yield level, period etc.), the hysteretic behavior of the component (which, in turn, depends on the axial force acting on the component, the prevailing stress condition etc.), and the characteristics of the earthquake.

Through dynamic response analyses allowing for the Bauschinger effect, Akiyama and Takahashi [21] found that for the particular case of beams and columns in steel moment-resisting frames (flexural systems), ${}_S\overline{\eta}$ and $\overline{\eta}$ can be approximately related by:

$$\overline{\eta}/{}_S\overline{\eta} = m \tag{10}$$

$$m = \frac{(1+p)}{(0.39+1.09p)} \tag{11}$$

where p is the axial force ratio, i.e. $p=N/Af_y$ (here N is the axial force, f_y the yield stress of the steel, and A the cross section of the member). Eqs. (10) and (11) provide an average value of the responses obtained from the dynamic analyses. For illustrative purposes, in Fig. 3 the relation between ${}_S\overline{\eta}$ and $\overline{\eta}$ given by Eqs.(10) and (11) is compared with the actual response obtained from dynamic response analyses for p=0 (indicated by the symbol ○).

www.witpress.com, ISSN 1743-3533 (on-line)

As observed in Fig. 2, Eq. (7) is very close to Eq. (6); thus, for the sake of simplicity, Eq. (7) can be adopted for the entire range of ${}_S\overline{\eta}$. By substituting the ordinate $\overline{\eta}_u$ of the point where the line defined by $\overline{\eta} = \overline{\eta}_i + m({}_S\overline{\eta} - {}_S\overline{\eta}_i)$ intersects the ultimate limit state curve given by Eq. (7) in Eq. (9), the damage index $\overline{\overline{\mathrm{ID}}}_i$ in a given domain of loading is:

$$\overline{\overline{\mathrm{ID}}}_i = \frac{\overline{\eta}_i}{\frac{7.33m}{m-1}\left\{\chi + \frac{1}{k_{p2}}\left[\frac{7.33}{m-1} - \sqrt{\frac{7.33^2}{(m-1)^2} + \frac{7.33k_{p2}m\chi}{0.5(m-1)} - bk_{p2} + (\tau_B - 7.33)^2\frac{k_{p1}-k_{p2}}{k_{p1}} + \frac{40k_{p2}}{k_{p1}} - 53.73}\right]\right\}} \tag{12}$$

$$\chi = \frac{{}_S\overline{\eta}_i}{7.33} - \frac{\overline{\eta}_i}{7.33m} - \frac{\tau_B - 1}{k_{p1}} + \frac{\tau_B}{k_{p2}} + \frac{b}{14.66} \tag{13}$$

The "key" structural performance parameters that govern the damage model given by Eq. (12) are b and m. b reflects the influence of the material properties, prevailing loading condition and geometry of the steel component. m reflects the influence of the structural system, the hysteretic behavior and the characteristics of the earthquake. The rest of variables, i.e. k_{p1}, k_{p2} and τ_B, simply define the shape of the skeleton curve and (in comparison to b) have minor influence on the structural performance of the steel component. Further research is needed to clarify the relation between ${}_S\overline{\eta}$ and $\overline{\eta}$ for structural systems other than the flexural system described above. A comprehensive statistical evaluation of the ratio $\overline{\eta}/{}_S\overline{\eta}$ is in progress for bracing systems.

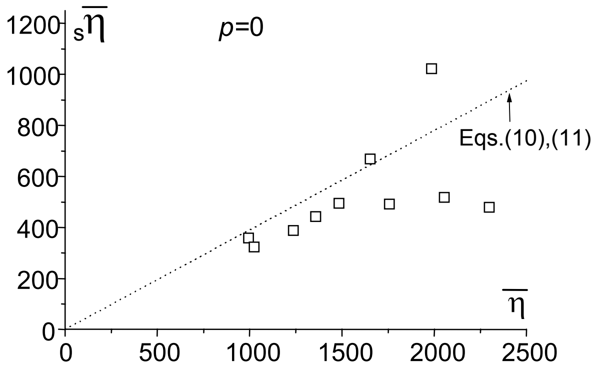

Figure 3: Comparison of Eqs. (8), (9) with the actual response obtained from numerical simulations.

4 Conclusions

In this paper a new approach to the seismic damage assessment of structural steel components is discussed. The approach is based on a low-cycle fatigue damage model proposed by the author. In this approach, the structural performance is governed by two "key" parameters: *b* and *m*. *b* reflects the influence of the material properties, prevailing loading condition and geometry of the steel component. *m* reflects the influence of the structural system, the hysteretic behavior and the characteristics of the earthquake. In its current form, the formulation is applicable to steel flexural systems with stable hysteretic response governed by bending/shear (whether or not combined with axial forces). Further research is needed to extend this formulation to other structural systems, such as bracing systems. The methodology can be used to quantify the level of damage in performance-based seismic design of new structures or to evaluate the safety of existing buildings and make decisions about seismic retrofitting.

Acknowledgements

The research reported in this paper was funded by the Spanish Ministry of Education, through the grant BIA2005-00591. The research received funds from the European Union (*Fonds Européen de Dévelopment Régional*).

References

[1] Kunnath, S.K., Reinhorn & A.M., Lobo R.F., *IDARC version3.0: a program for the inelastic seismic analysis of RC structures*, Technical Report NCEER-92-0022 State University of New York: Buffalo, N.Y., 1992.

[2] Krawinkler, H. & Nassar, A.A., Seismic design based on ductility and cumulative damage demands and capacities. *Nonlinear Seismic Analysis and Design of Reinforced Concrete Buildings*, eds. P. Fajfar & H. Krawinkler, Elsevier: New York, pp. 77-95, 1992.

[3] Fajfar, P. & Gaspersic, P., The N2 method for the seismic damage analysis of RC buildings, *Earthquake Engineering and Structural Dynamics*, **25**, pp. 31-46, 1996.

[4] Manson, S.S., Behavior of materials under conditions of thermal stress. *Heat Transfer Symposium, Engineering Research Institute*, University of Michigan: Ann Arbor, pp. 9-15, 1953.

[5] Coffin, L.F., A study on the effect of cyclic thermal stresses in ductile metals, *Transactions of the American Society of Mechanical Engineers,* **76**, pp. 931-950, 1954.

[6] Miner, M.A., Cumulative damage in fatigue, *Journal of Applied Mechanics,* **12,** pp. 159-164, 1945.

[7] Kasiraj, I. & Yao, J.T.P., Fatigue damage in seismic structures, *Journal of Structural Engineering ASCE,* **95(8)**, pp. 1673-1692, 1969.

[8] Suidan, M.T. & Eubanks, R.A., Cumulative fatigue damage in seismic structures, *Journal of Structural Engineering ASCE,* **99(5)**, pp. 923-943, 1973.

[9] Krawinkler, H. & Zhorei, M., Cumulative damage in steel structures subjected to earthquake ground motions, *Computers and Structures*, **16(1-4)**, pp. 531-541, 1983.
[10] Cosenza, E., Manfredi, G. & Ramasco, R., The use of damage functionals in earthquake engineering: a comparison between different methods, *Earthquake Engineering and Structural Dynamics,* **22**, pp. 855-868, 1993.
[11] Ballio, G. & Castiglioni, C.A., A unified approach for the design of steel structures under low and/or high cycle fatigue, *Journal of Constructional Steel Research,* **34**, pp. 75-101, 1995.
[12] Daali, M.L. & Korol, R.M., Low-cycle fatigue damage assessment in steel beams, *Structural Engineering and Mechanics*, **3**, pp. 341-358, 1995.
[13] Banon, H. & Veneziano, D, Seismic safety of reinforced concrete members and structures, *Earthquake Engineering and Structural Dynamics,* **10,** pp. 179-193, 1982.
[14] Park, Y.J. & Ang, A.H.S., Mechanistic seismic damage model for reinforced concrete, *Journal of Structural Engineering ASCE*, **111(4)**, pp. 722-739, 1985.
[15] Park, Y.J., Ang, A.H.S. & Wen, Y.K., Seismic damage analysis of reinforced concrete buildings, *Journal of Structural Engineering ASCE,* **111(4)**, pp. 740-757, 1985.
[16] Fajfar, P., Equivalent ductility factors taking into account low-cycle fatigue, *Earthquake Engineering and Structural Dynamics,* **22(10)**, pp. 837-848, 1992.
[17] Benavent-Climent A., An energy-based damage model for seismic response of steel structures, *Earthquake Engineering and Structural Dynamics,* **36**, (in press), 2007.
[18] Kato, B., Akiyama, H. & Yamanouchi, H., Predictable properties of structural steels subjected to incremental cyclic loading, *Symposium on Resistance and Ultimate Deformability of Structures Acted on by Well Defined Loads*, IABSE: Lisbon, 1973.
[19] Akiyama, H., Takahashi, M. & Shi, Z., Ultimate Energy Absorption Capacity of Round-Shape Steel Rods Subjected to Bending, *Journal of Structural and Construction Engineering AIJ*, **475**, pp. 149-158, 1995.
[20] Benavent-Climent, A., OH, S.H. & Akiyama, H., Ultimate Energy Absorption Capacity of Slit-Type Steel Plates Subjected to Shear Deformations. Journal of Structural and Construction Engineering, Transactions of the Architectural Institute of Japan 1998; 503: 139-147.
[21] Akiyama, H, Takahashi, M. Influence of Bauschinger Effect on Seismic Resistance of Steel Structures, *Journal of Structural and Construction Engineering, AIJ*, pp. 49-57, **418**, 1990.
[22] Benavent-Climent, A., Seismic Design of Structures by Using Brace-Type Hysteretic Dampers, *Doctoral Thesis*, University of Tokyo: Tokyo, 1998.