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QUANTITATIVE SAFETY ASSESSMENT OF STEEL MEMBERS UNDER SEVERE SEISMIC EXCITATIONS

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ABSTRACT

In this paper, the previous damage models for structural components under seismic repeated loading were reviewed systematically. A failure criterion for steel members under severe cyclic excitations as in strong earthquakes was described. A new approach to seismic damage assessment for steel members was proposed. This method was based on a series of the experimental and numerical investigations for steel members under very low cycle loading. The proposed damage assessment method was focused on the local strain history at the cross-section of the most severe concentration of deformation.

KEYWORDS

Damage model; damage index; steel structure; member; seismic repeated loading; very low cycle loading; local strain; crack.

INTRODUCTION

In the last 30 years, numerous approaches to seismic damage assessment have been proposed. This damage evaluation has been accompanied with high-storied and large-scalization in civil engineering structures such as buildings, bridges, offshore oil and gas platforms, nuclear power plants and so on. Although a lot of researches, both experimental and analytical, has been carried out to make a exact model or method to seismic damage assessment for structures and their members, a decisive one is not developed yet. Therefore it is necessary to develop a new damage assessment method considering exactly the physical aspects of seismic damage and failure mechanisms for structures and their members, which may finally be able to improve general reliability and damage estimation against earthquakes (Park, 1993).

It is obvious for economically-designed steel structures that brittle failure in welded connections or elastic instability failure is not a serious problem. The most important thing clarify the inelastic post-buckling behavior of members under repeated loading. In order to establish a safety assessment criterion for steel structures under severe seismic excitations, the structural behavior at the critical section of members under very low cycles of loading must be considered (Park et al., 1996). The reason is because structural damage and failure are often associated with plastic and/or unstable behavior of structural members due to large cyclic deformations with initiation of local buckling. The structural behavior at the critical section may be related to the local strain or stress at the critical cross-section of members. Thus, it appeared to the author that the damage criterion at the point of the local strain history of the critical cross-section is required to estimate the seismic safety more precisely.

A new approach to seismic safety assessment for steel members was proposed herein. This method was based upon a series of the experimental and numerical investigations for steel angle members under the very low cycles of loading. Importance of local strain parameter was discussed in relation to cracking to identify a quantitative relationship between damage and itself. The proposed damage estimation method is focused on the local strain history at the cross-section of the most severe concentration of local deformation.

DAMAGE MODELS UNDER SEISMIC REPEATED LOADING

The previous damage indices may be divided into four groups according to important factors used for damage assessment; (1) ductility (displacement, plastic strain), (2) stiffness (rigidity), (3) dissipated energy, and (4) combination of ductility and dissipated energy. Tables 1-3 show a systematic classification of the damage indices for structural steel components and reinforced concrete members subjected to cyclic loading, which have been proposed during the past 30 years. Here the local damage indices are defined as the damage indices for a individual component, i.e. structural element or member.

The damage indices based on ductility ratios are one of the first proposals to assess the damage contents due to cyclic loads. These indices are basically related to a ratio of the capacity (plastic strain) and demand (largest plastic strain resulting from cyclic loads) of structural components. This concept has been modified to contain the effect of cumulative damages by Yao and Munse (Table 1), Bertero & Bresler (Table 1), Blejwas & Bresler (Table 1), and Stephens & Yao (Table 2). In these damage models, however, it does not consider the effects of the loading sequence and the local instabilities in the plastic region of members accentuated by repeated loading.

Lybas & Sozen (Table 2), and Roufaiel & Meyer (Table 2) developed another types of damage indices connected with stiffness degradation, which presented the damage states as a simple ratio of stiffness. These types of damage indices cannot be used as a exact damage predictor under inelastic cyclic loading because of no consideration of the cumulative state of damage.

Gosain et al. (Table 2) proposed an energy index for seismic damage assessment. In recent years, many proposals for damage assessment associated with the dissipated energy have been made by Banon et al. (Table 2), Hwang and Scribner (Table 3), Darwin & Nmai (Table 3), and Chung et al. (Table 3). As pointed out in the above investigations, the energy dissipation capacity has been shown to have a potentially important influence on the damage of reinforced concrete members during strong earthquakes. Namely, these concepts seem to be very attractive for the derivation of a reliable damage model for reinforced concrete members. But it is extremely difficult to present the relationship between the damage level and the dissipated energy, since the number of cycles to failure and total energy dissipation capacity are not corresponding one-to-one.

Park et al. (Table 3), and Mizuhata & Nishigaki (Table 3) introduced a seismic damage model for reinforced concrete members including the ductility ratio and dissipated energy. Even though the various numerical factors in the damage index were obtained from the regression of numerous experimental data, the damage model was represented by a linear combination of damages caused by the maximum deformation and the hysteretic energy dissipation during earthquake. This linear summation resulted in a discrepancy, because these two variables were linearly independent.

Structural behavior and damage process under earthquake loading are very complex phenomena which are very difficult to model analytically or to reproduce in laboratory experiments. The literatures regarding the damage models are very vast, as indicated in Tables 1-3. The common approaches to seismic damage assessment are based on the concepts of Manson-Coffin hypothesis, ductility and stiffness ratios, or dissipated energy hypothesis for low cycle fatigue. But the states of damage and failure in structural members are not yet definite. A clear definition for damage and failure of members under severe cyclic loading is required. Almost all of these past models have disregarded local instability phenomena in the plastic regions of member related to damage and failure caused by earthquake loads.

DEFINITION OF FAILURE FOR STEEL MEMBERS UNDER STRONG SEISMIC EXCITATIONS

In order to represent quantitatively structural damage and failure for structural members subjected to earthquake loads, a clear definition of failure for members under repeated loading is required. Because failure mechanisms are often associated with very complex physical phenomena, however, it is difficult to describe the failure state definitely. There are basically five different approaches to define failure state for members subjected to cyclic loading: (1) the initiation of surface cracking (Goto et al., 1974), (2) the crack tip opening

Table 1. Classification of previously proposed damage indices (I)

Class	Related variable	Proposer	Equation	Remarks
LOCAL DAMAGE INDEX	Ductility (displacement, plastic strain) ratio	Yao & Munse (1962)	$D = \sum_{i=1}^{n} \left(\frac{\Delta q_{t,i}}{\Delta q_{t1,i}} \right)^{1/m}$	D ; cumulative damage ranged from 0.0 - 1.0 n ; number of cycles $\Delta q_{t,i}$; incremental positive plastic strain during i-th cycle $\Delta q_{t1,i}$; tension plastic strain to cause failure during i-th cycle m ; material constant object ; for structural steel components
		Bertero & Bresler (1977)	$D_i = \frac{d_i}{c_i}$	D_i ; damage index for a given element i d_i ; response(demand) of the element i c_i ; resistance(capacity) of the element i object ; for reinforced concrete members
		Blejwas & Bresler (1979)	$r_i = \frac{d_i - c_i^0}{c_i^u - c_i^0}$ $(0 \le r_i \le 1)$	r_i ; damage index for the i-th element d_i ; demand parameter from a combination of response parameters c_i^0 ; capacity at which damage is initiated c_i^u ; capacity at which damage is ultimate object ; for reinforced concrete members
		Banon, Biggs & Irvine (1981)	$\mu_\theta = \frac{\theta_{max}}{\theta_y}$ $\mu_\phi = \frac{\phi_{max}}{\phi_y}$ $NCR = \frac{\sum \lvert\theta_0\rvert}{\theta_y}$	μ_θ ; rotation ductility θ_{max} ; maximum rotation θ_y ; yield rotation μ_ϕ ; curvature ductility ϕ_{max} ; maximum curvature ϕ_y ; yield curvature NCR ; normalized cumulative rotation θ_0 ; plastic rotation object ; for reinforced concrete members
		Krawinkler & Zohrei (1983)	$D = C \sum_{i=1}^{n} (\Delta\delta_{pi})^c$	D ; cumulative damage index of member C, c ; damage parameters $\Delta\delta_{pi}$; plastic strain during the i-th cycle object ; for structural steel components

Table 2. Classification of previously proposed damage indices (II)

Class	Related variable	Proposer	Equation	Remarks
LOCAL DAMAGE INDEX	Ductility (displacement, plastic strain) ratio	Stephens & Yao (1987)	$D = \sum_{i=1}^{n} \left(\frac{\Delta\delta_{pt,i}}{\Delta\delta_{pf,i}} \right)^{\alpha}$ where, $\alpha = 1 - b \times \frac{\Delta\delta_{pc,i}}{\Delta\delta_{pt,i}}$	D ; damage for all n cycles $\Delta\delta_{pc,i}$; negative change in plastic deformation in the i-th cycle $\Delta\delta_{pt,i}$; positive change in plastic deformation in the i-th cycle $\Delta\delta_{pf,i}$; positive change in plastic deformation to cause failure in the i-th cycle b ; deformation ratio coefficient object ; for reinforced concrete members
	Stiffness (rigidity) degradation	Lybas & Sozen (1977)	$DR = \frac{K_0}{K_r}$	DR ; damage ratio K_0 ; initial tangent stiffness K_r ; reduced secant stiffness corresponding to maximum displacement object ; for reinforced concrete members
		Roufaiel & Meyer (1987)	$MFDR = \max\left[MFDR^{+}, MFDR^{-}\right]$ where, $MFDR^{+} = \frac{\frac{\phi_x^+}{M_x^+} - \frac{\phi_y^+}{M_y^+}}{\frac{\phi_m^+}{M_m^+} - \frac{\phi_y^+}{M_y^+}}$ $MFDR^{-} = \frac{\frac{\phi_x^-}{M_x^-} - \frac{\phi_y^-}{M_y^-}}{\frac{\phi_m^-}{M_m^-} - \frac{\phi_y^-}{M_y^-}}$	M_y/ϕ_y ; initial elastic stiffness M_x/ϕ_x ; secant stiffness at arbitrary local level M_m/ϕ_m ; secant stiffness at onset of failure $+$ and $-$; loading direction $MFDR = 1$ signifies the onset of failure. object ; for reinforced concrete members
	Dissipated energy	Gosain, Brown & Jiras (1977)	$I_w = \sum_{i=1}^{n} \frac{P_i \Delta_i}{P_y \Delta_y}$	I_w ; work index n ; number of load cycles in which $P_i/P_y \geq 0.75$ P_i, Δ_i ; load and corresponding displacement during i-th cycle P_y, Δ_y ; load and corresponding displacement at yield of flexural bars object ; for reinforced concrete members
		Banon, Biggs & Irvine (1981)	$E_n(t) = \frac{\int_0^t M(\tau)\theta(d\tau)}{\frac{M_y \theta_y}{2}}$	$E_n(t)$; normalized dissipated energy as a function of time "t" t ; elapse time M ; moment θ ; rotation M_y ; yield moment θ_y ; yield rotation object ; for reinforced concrete members

Table 3. Classification of previously proposed damage indices (III)

Class	Related variable	Proposer	Equation	Remarks
LOCAL DAMAGE INDEX	Dissipated energy	Hwang & Scribner (1984)	$I_E = \sum_{i=1}^{n} E_i \cdot \frac{K_i}{K_e} \cdot \left(\frac{\Delta_i}{\Delta_y}\right)^2$	I_E ; energy index E_i ; energy dissipated during the i-th cycle K_e, Δ_y ; elastic stiffness and yield deflection, respectively K_i, Δ_i ; flexural stiffness and maximum displacement in the i-th cycle, respectively n ; number of cycles with $P \geq 0.75P_y$ object ; for reinforced concrete members
		Darwin & Nmai (1986)	$D_i = \dfrac{E}{0.5P_y\Delta_y\left[1+\left(\frac{A'_s}{A_s}\right)^2\right]}$	D_i ; normalized dissipated energy index E ; total dissipated energy A'_s, A_s ; area of compression and tension steel, respectively P_y ; yield load Δ_y ; yield deflection object ; for reinforced concrete members
		Chung, Meyer & Shinozuka (1987)	$D_e = \sum_i \sum_j \left(\alpha^+_{ij}\frac{n^+_{ij}}{N^+_i} + \alpha^-_{ij}\frac{n^-_{ij}}{N^-_i}\right)$	D_e ; cumulative damage index i ; indicator of different displacement or curvature level j ; indicator of cycle number for given load level i N_i ; number of cycles(with curvature level i) to cause failure n_{ij} ; number of cycles(with curvature level i) actually applied α_{ij} ; damage accelerator +, − ; indicator of loading sense object ; for reinforced concrete members
	Linear combination of ductility ratio and dissipated energy	Park, Ang & Wen (1984)	$D_e = \frac{\delta_{max}}{\delta_u} + \frac{\beta}{Q_y\delta_u}\int dE$	D_e ; damage index δ_{max} ; maximum deformation experienced so far δ_u ; ultimate deformation under monotonic loading Q_y ; calculated yield strength dE ; dissipated energy increment β ; non-negative parameter object ; for reinforced concrete members
		Mizuhata & Nishigaki (1987)	$D_k(n)\frac{\lvert\delta_{max}\rvert}{\delta_F} + \sum_{i=1}^{k}\frac{E_{ki}}{E_r}$	$D_k(n)$; damage index of member k ; number of cycles δ_{max} ; maximum displacement experienced δ_F ; displacement at failure under monotonic loading E_{ki} ; dissipated energy in the i-th cycle E_r ; total dissipated energy object ; for reinforced concrete members

displacement (CTOD) criterion (Caddell, 1980), (3) the occurrence of severe cracking (Rolfe and Barsom, 1977), (4) the strength drop reaching below a certain value of the initial yield stress (Atalay and Penzien, 1975), (5) the complete rupture of the cross-section (Bourgund et al., 1989).

A steel angle consists of two steel plates, and experiences the fundamental failure modes (e.g. failure caused by compression and bending, or tension and bending) due to global bending and local buckling deformation under the very low cycle loading (Iwai et al., 1993). In the very low cycle loading tests of steel angles, inelastic buckling caused a sudden decrease in the compressive load-carrying capacity, but only a slightly decrease in the succeeding tensile load-carrying capacity. However, both the compressive and tensile load-carrying capacities were considerably decreased by the initiation of cracking. It is thus found that ultimate state of failure is closely related to the occurrence of a visible crack. In this investigation, therefore, the failure state for steel members under earthquake loading is characterized by the initiation of surface cracking.

SEISMIC DAMAGE ASSESSMENT FOR STRUCTURAL STEEL MEMBERS

The approaches to seismic damage estimation are often based upon the energy dissipation capacity. But the energy dissipation process and capacity depend heavily on the loading history and failure mode (Park, 1993). Therefore, it may be impossible that the damaged state under inelastic cyclic loading is clearly represented by a simple one-to-one correspondence between dissipated energy capacity and physical damage such as the outbreak of cracks.

In estimating the damage states under severe cyclic excitations as in strong earthquakes, the local strain on the inner and outer surface fibers of cross-section inducing the most severe stress concentration may be useful because it provides a quantitative means for predicting the failure state, namely the initiation of cracking under the very low cycles of loading, by the summation of the local strain. From the results of the analysis and experiment (Park et al., 1996), the failure of structural steel members subjected to the large cyclic compressive/tensile bending is related to the occurrence of the order of 104-255%, in the summation, of the local strain at the extreme fibers of the critical cross-section. In this case, the physical failure state is translated into the local strain value in the continuum finite element model, based on the results from the experiment and numerical analysis.

Fig. 1 presents the relations between the summation of local strain in the critical cross-section and the increasing number of cycles for all the models obtained from the analysis. Here the lower and upper dash lines correspond to 90% and 115% of the residual local strain at the ruptured portion under monotonic-tensile testing, respectively (Park et al., 1996). Based upon the failure definition described in the above section and the result of Fig. 1, a new damage assessment method for structural steel members under severe cyclic loading can be suggested. The new method may be described by the summation of the critical local strain in the course of increasing number of cycles, which is conservatively compared with the threshold value 90% of the local strain at the ruptured portion under the monotonic-tensile testing. That is, the failure state is indicated if the summation of the local strain in the critical cross-section exceeds the limit strain of 90%.

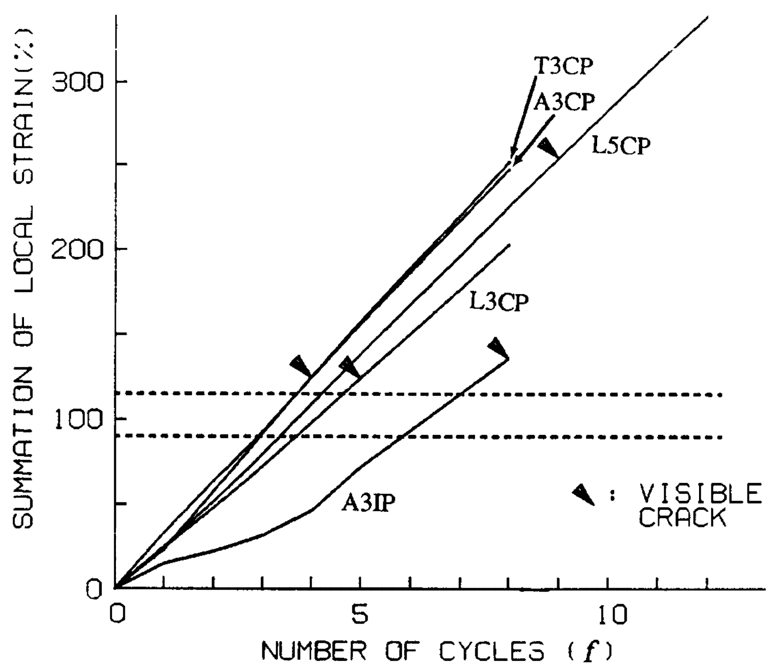
Thus, the damage indicator under cyclic excitations as in strong earthquakes may be formulated as follows;

$$\text{If} \quad \sum_{i=1}^{N} \varepsilon_i \geq \varepsilon_{limit} \quad , \quad \text{then failure state} \tag{1}$$

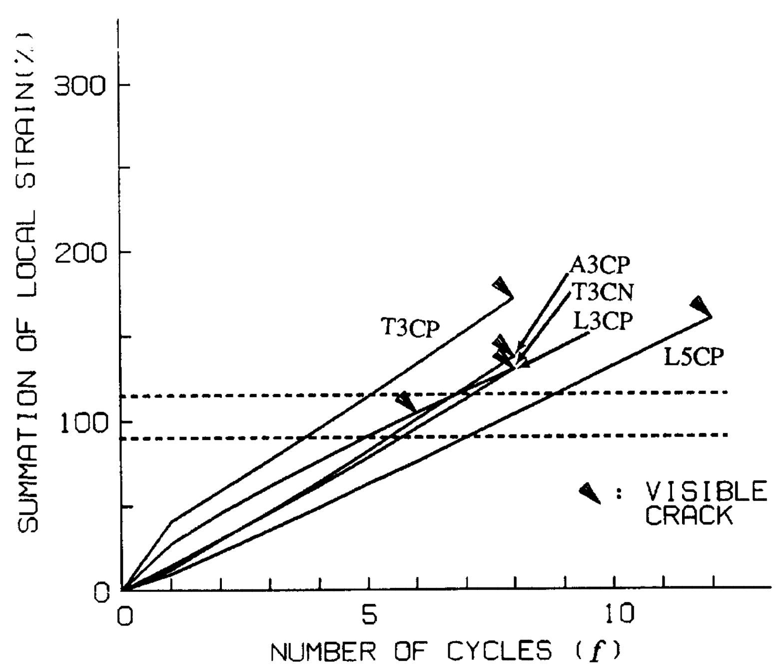
and

$$\text{If} \quad \sum_{i=1}^{N} \varepsilon_i < \varepsilon_{limit} \quad , \quad \text{then no failure state} \tag{2}$$

where N, ε and ε_{limit} are the number of cycles, the local strain in the critical cross-section and the limit strain, respectively. The value of ε_{limit} might be 90% in the case of structural steel members under the very low



(a) Inner surface



(b) Outer surface

Fig. 1. Relationships between the summation of local strain and the increasing number of cycles

cycle loading. In general, the local strain history may be calculated by the numerical approach using finite element analysis with material nonlinear and geometric nonlinear effects.

From Eqs. (1) and (2), the damage "*D*" of steel member to "*N*" cycles can be determined by the normalized form as follows;

$$D = \sum_{i=1}^{N} \left(\frac{\varepsilon_i}{\varepsilon_{limit}} \right) \tag{3}$$

Therefore, $D \geq 1$ indicates the failure state, while $D < 1$ denotes a degree of damage at the corresponding loading step. As indicated in Fig. 1, the increment in damage indicators as function of the number of increasing cycles can be observed. The failure state, i.e. the initiation of cracking, for all the models may be detected at the calculated damage indicator in the order of 1.2-1.5. It might be, therefore, considered that the threshold local strain 90% corresponds to the occurrence of the ultimate failure for steel members due to cyclic loads such as strong seismic loads.

CONCLUSIONS

From this survey of the literatures, it was apparent that the common approaches to seismic damage assessment were based on the concepts of Manson-Coffin hypothesis, ductility and stiffness ratios, or dissipated energy hypothesis. There was five approaches to define the failure state, i.e. the initiation of surface cracking, the crack tip opening displacement criterion, the occurrence of severe cracking, the strength drop reaching below a certain value of the initial yield stress and the complete rupture of the cross-section. A failure criterion related to the initiation of surface cracking was described, which applied for the failure definition for steel members under inelastic cyclic loading as in strong earthquakes. Based on a series of the experimental and analytical investigations using steel angle members subjected to the very low cycle loading, a new approach to seismic damage assessment for steel members has been proposed in conjunction with the suggested definition of failure. The new damage estimation method might be described by the summation of the critical local strain in the course of increasing number of cycles, which is compared with the threshold value 90% of the residual local strain at the ruptured portion under the monotoic-tensile testing. However, it might be a shortcoming that the new damage assessment method was derived from the experimental and analytical results for the steel angle members only. With respect to further research, the experimental and analytical investigations for structural steel members having various shapes under very low cycle loading may be needed to verify the proposed damage indicator.

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