A NEW DAMAGE INDEX FOR SEISMIC PERFORMANCE OF STEEL FRAMES

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ABSTRACT

A new method of seismic design of steel moment resisting framed structures is developed. This method is able to control damage at all levels of performance in a direct manner. More specifically, the method a) can determine damage in any member or the whole of a designed structure under any given seismic load, b) can dimension a structure for a given seismic load and desired level of damage and c) can determine the maximum seismic load a designed structure can sustain in order to exhibit a desired level of damage. In order to accomplish these things, a new seismic damage index and a detailed damage scale are developed. The new damage index takes into account the interaction between axial force and bending moment at a section, strength and stiffness degradation as well as low cycle fatigue. The damage scale is constructed on the basis of extensive parametric studies involving a large number of frames and seismic motions.

1. INTRODUCTION

In earthquake-resistant design of steel structures, different design methods have been used in practice or proposed by researchers. Among them, one can mention the force-based design (FBD), the displacement-based design (DBD), the performance based design (PBD) and the hybrid force/displacement based design (HFD).

According to the first method [1], which is the current practice in existing seismic codes, seismic forces are used as the main design parameters. This approach demands the design of the building against structural failures which might endanger human life on the basis of recommended constant values of the behavior (or strength reduction) factor, q. Finally, deformations beyond which service requirements are no longer met after the detailing of the structure are checked near the end of the design. The DBD [2] idea is the direct satisfaction of the serviceability requirements, the most important of which is the limitation of displacements. Thus, the DBD determines first the target displacements, then the appropriate stiffness of the structure and finally the structural and member forces which lead to the dimensions of structural members. Thus, the displacements play here the fundamental role in design. PBD [3] introduces a new framework in seismic design of structures by defining performance levels and objectives. Thus, three to five structural performance levels are defined and should be achieved for increasing levels of earthquake actions. These performance levels mainly describe the damage of a structure which is quantified through indices, such as the interstory drift ratio (IDR), or the member plastic rotations. The HFD [4] is a new seismic design method for steel frames which combines the advantages of the well-known force-based and displacement-based seismic design methods. The main characteristics of this method are: (1) treats both drift and ductility demands as input variables for the initiation of the design process through a q factor which depends on them and the characteristics of the structure (e.g. the number of stories); (2) makes use of current seismic code approaches as much as possible (e.g., conventional elastic response spectrum analysis and design); and (3) recognizes the influence of the type of the lateral load resisting system.

In this paper, the Direct Damage Controlled Design (DDCD) method, a new design method for steel moment resisting framed structures under earthquake excitation, is proposed. The basic advantage of DDCD is the dimensioning of beam members or whole framed structures with damage directly controlled at both local and global levels. In other words, the designer can select a priori the desired level of damage in a structural member or a whole structure and direct his design in order to achieve this preselected level of damage.

2. HYSTERETIC MODELS THAT INCORPORATE STRENTH AND STIFFNESS DEGRADATION

2.1 Preliminaries

Several hysteretic models have been developed. Some of them have hysteresis rules that account for stiffness deterioration by modifying the path by which the reloading branch approaches the backbone curve, e.g., the peak oriented model [5] or various ‘pinching’ models [6]. The need to model both stiffness and strength degradation led to the development of more versatile models like those of Ref. [7-10]. In the commercial computer program Ruaumoko 2D [11] for the seismic non-linear analysis of framed structures, stiffness and strength degradation can be taken into account through a linear function that depends on the inelastic cycles a member sustains. This model is described in the next subsection.

2.2 Ruaumoko model

Ruaumoko is a program that performs nonlinear dynamic analysis with the aid of the finite element method [11]. It utilizes, among others, a material behavior model that takes into account strength degradation with the number of inelastic cycles. More specifically, for the two dimensional (2D) case, the strength loss in each loading direction is governed by a parameter f that is multiplied by the initial strength and is a linear function of the number of inelastic cycles. This parameter is given by the equation

$$f = \frac{S_r - 1}{n_2 - n_1}(n - n_1) + 1 \tag{1}$$

where n_1 is the cycle at which degradation begins, n_2 the cycle at which degradation stops and S_r, the residual strength as a function of the initial yield strength. The stiffness deteriorates so that the yield displacement remains constant.

2.3 Calibration with experiments

For the calibration of the above material model of Ruaumoko 2D, results from experiments performed at the laboratory ATLSS of Lehigh University [12] were used. The experimental studies focused on the cyclic inelastic performance of full-scale welded unreinforced flange moment connection specimens.
The experiment C2 was simulated by Ruaumoko 2D and the moment-rotation curves of the right beam of the connection were evaluated. The experimental curve is shown in Figure 2, together with the one simulated by Ruaumoko 2D. The agreement between the experimental and the numerical curves is considered to be satisfactory.

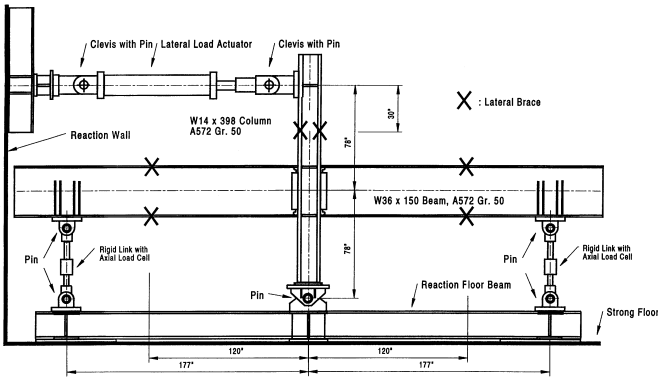

Fig. 1. Interior connection specimen

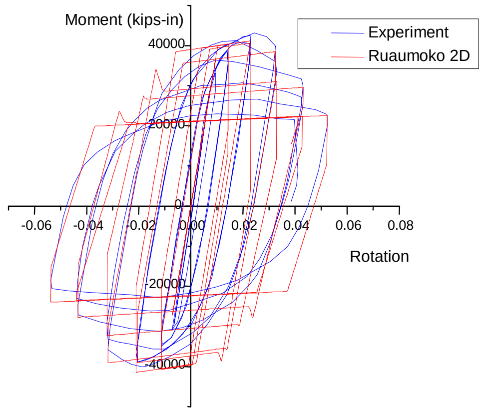

Fig. 2. Moment-rotation curves

3. PROPOSED DAMAGE INDEX

In this section the proposed damage index is presented. It is defined at the section of a steel member and takes the following form:

$$D_S = \frac{c}{d} = \frac{\sqrt{(M_S - M_A)^2 + (N_S - N_A)^2}}{\sqrt{(M_B - M_A)^2 + (N_B - N_A)^2}} \tag{2}$$

In the above, the bending moments M_A, M_S and M_B and the axial forces N_A, N_S and N_B as well as the distances c and d are those shown in the bending moment M - axial force N interaction diagram of Fig. 3 for a plane beam-column element. The bending moment M_S and axial force N_S are those acting in a specific section. Figure 3 includes a lower bound damage curve, the limit between elastic and inelastic material behavior and an upper bound damage curve, the limit between inelastic behavior and complete failure. Thus, damage at the former curve is zero, while at the latter curve is equal to one. Equation 2 is based on the assumption that damage evolution varies linearly between the above two damage bounds.
The lower bound curve can be described as

$$\begin{aligned}&\frac{0.88M}{f\cdot M_{pl}}+\frac{N}{f\cdot N_{pl}}=1\\&\textit{for}\quad \mathrm{M}\le 0.9M_{pl}\quad \textit{and}\quad N\le 0.2N_{pl}\\&\frac{M-M_{pl}}{f\cdot M_{pl}}+\frac{N}{f\cdot N_{pl}}=1\\&\textit{for}\quad \mathrm{M}> 0.9M_{pl}\quad \textit{and}\quad N> 0.2N_{pl}\end{aligned} \tag{3}$$

where N_{pl} and M_{pl} are given by the following expressions

$$M_{pl} = f_y W_{pl} \tag{4}$$

$$N_{pl} = f_y A \tag{5}$$

where f_y is the yield stress of steel, W_{pl} is the plastic modulus and A is the section area. The upper bound curve can be expressed as

$$\frac{M}{f\cdot M_u}+\left(\frac{N}{f\cdot N_u}\right)^2=1 \tag{6}$$

where N_u and M_u are the ultimate axial force and bending moment, respectively, which cause failure of the section. The f is the scale factor of eq. 1 that multiplies the surfaces so as the phenomena of strength and stiffness degradation to be taken into account. Equations 3 to 6 have been used for the construction of the bounding curves of Fig. 3.
The increase of damage related to strength reduction due to low-cycle fatigue is taken into account through the assumption made by Sucuoğlu & Erberik [13]. More specifically, an amount of damage ΔD_s, related to this phenomenon, should be added to damage, D_s, computed by eq. 2. In fig. 4 the first and the n-th positive cycle are shown at the constant amplitude of yield rotation θ_y, with respective moment yield values of M_y and M_y^n. In this case, where the axial force is zero, damage is given by the following expression

$$D_S=\frac{M_S-M_y}{M_u-M_y}=\left(\frac{k_f}{k_o-k_f}\right)\left(\frac{k_o}{k}-1\right) \tag{7}$$

where k_o is the initial elastic stiffness, k_f, is the secant stiffness at the ultimate moment M_u, and k is the secant stiffness at the current cycle.

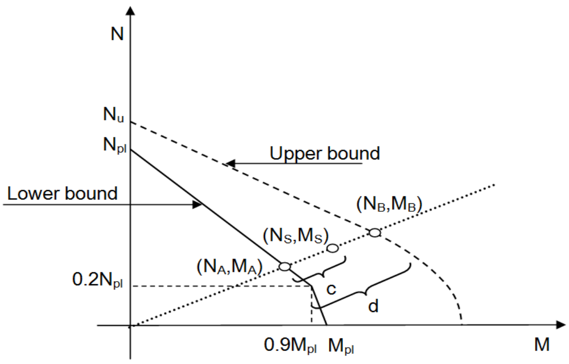

Figure 3. Definition of damage index.

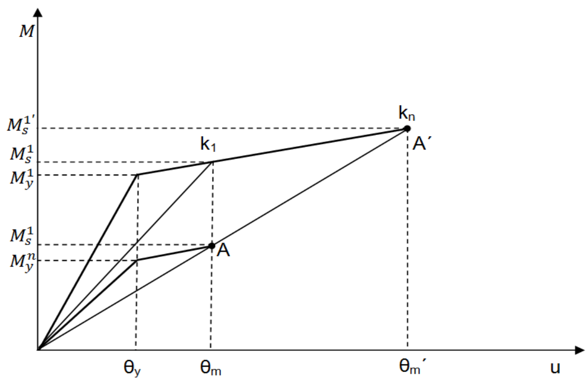

Figure 4. Increase of damage due to low-cycle fatigue.

In Fig. 4, k_1 and k_n are the corresponding effective stiffnesses substituted for k in eq. 7 in order to determine the damage D_{s1} και D_{s2}, in the first and the n-th cycles, respectively. Eventually the moment M_s^1 in the first cycle reduces to M_s^n by an amount ΔM_n in the n-th cycle, leading to an increase in damage due to the associated reduction in the effective stiffness from k_1 to k_n according to equation 5. The projection of the intercept of k_n with the hardening branch in the first cycle (point A′) on the moment axis indicates that the same amount of damage would be experienced at the n-th cycle if the system was pushed to the rotation θ_m' to reach the moment $M_s^{1\prime}$. Hence, equation 7 yields the associated damage D_{s2} at the n-th cycle when M_s is replaced by $M_s^{1\prime}$ In this case, the projected moment $M_s^{1\prime}$ is composed of the moment M_s^1 and an additional moment ΔM_s arising from strength loss. Thus, an amount of damage ΔD_s should be added to the system due to increase of moment which is equal to

$$\Delta D_S = \frac{M_s^{1'} - M_s^1}{M_u^1 - M_y^1} \tag{8}$$

This methodology can be adopted in the case that the axial force is not zero. In this case the additional moment due to stress loss is found in the same way as above and the damage is calculated with the aid of Eq. 2.

The calculation of ΔD_s results from the following empirical expression

$$\Delta D_s = 0.56 \cdot n^{0.292} \cdot D_s^{0.914} \tag{9}$$

4. DAMAGE CONTROLLED STEEL DESIGN

4.1 Definition of performance levels

Damage is used here as a design criterion. Thus, the designer, in addition to a method for determining damage, also needs a scale of damage in order to decide which level of damage is acceptable for his design. Many damage scales can be proposed in order to select desired damage levels associated with the strength degradation and capacity of a structure to resist further loadings. Table A.1. in the appendix provides three performance levels (I.O. = Immediate Occupancy, L.S. = Life Safety and C.P. = Collapse Prevention) associated with modern performance-based seismic design with the corresponding limit response values (performance objectives) in terms of IDR = interstorey drift ratio, θ_{pl} = plastic rotation at member end, μ_θ = local ductility and d = damage. The relevant references are also shown in Table A.1. In the present work, an extensive parametric study was conducted for 36 plane steel moment-resisting frames subjected to 40 ground motions (23040 analyses = 36 frames*40 ground motions*16 analyses on average)for the evaluation of a damage scale. The frames were analyzed with the program Ruaumoko 2D using the incremental dynamic analysis method. The characteristics of the frames as well as of the ground motions can be found in [14]. The Levenberg-Marquardt algorithm (MATLAB 1997) was adopted for the non-linear regression analysis of the results of parametric studies, leading to two expressions, one for the beams and one for the columns, that give the maximum damage that is observed at a member, as a function of IDR:

$$D_c = 9.52(IDR - 0.007)^{0.80} \tag{10}$$

$$D_b = 13.5(IDR - 0.005)^{0.81} \tag{11}$$

where D_c and D_b are the maximum column and beam damage, respectively.

The ratio D_{ex}/D_{ap} of the exact values of damage at columns or beams obtained from inelastic dynamic analyses to the approximate ones calculated from eqs 10 and 11, respectively, is evaluated. Equation 10 corresponds to a ratio $D_{c,ex}/D_{c,ap}$ with mean value equal to 0.99, central value equal to 0.88 and standard deviation equal to 0.43. For equation 11 the abovementioned values for the ratio $D_{b,ex}$ /$D_{b,ap}$ are 0.97, 0.92 and 0.25, respectively. Using eqs 10 and 11 as well as the values of the maximum IDR provided in [15], a damage scale for beams and columns and the performance levels of these guidelines, is defined. This damage scale is shown in Table 1.

Performance Levels	Maximum column damage	Maximum beam damage
I.O.	≤0%	≤8%
L.S.	≤40%	≤57%
C.P.	≤77%	≤100%

Table 1. Damage scale proposed here for the performance levels of FEMA-273.

4.2 Damage controlled steel design

The application of the proposed Direct Damage Controlled Design (DDCD) method to plane steel members and framed steel structures is done with the aid of the Ruaumoko 2D program.

The user has three design options at his disposal in connection with damage controlled steel design:

a) determine damage in any member or the whole of a designed structure under any given seismic load
b) dimension a structure for given seismic load and desired level of damage
c) determine the maximum seismic load a designed structure can sustain in order to exhibit a desired level of damage.

The first option is the one usually done in current practice. The other two options are the ones which actually make the proposed design method a direct damage controlled one, with the second option providing the ability of easily applying capacity design (‘‘weak beams – strong columns’’).

5. CONCLUSIONS

On the basis of the preceding developments, the following conclusions can be stated:

1. A new method of seismic design of plane steel moment resisting frames subjected to ground motions, the Direct Damage Controlled Design (DDCD), has been developed.
2. The method works with the aid of the finite element method incorporating material and geometric nonlinearities.
3. It uses a new damage index that accounts for the interaction between the axial force and the bending moment at a member section, incorporates cyclic strength and stiffness deterioration and accounts for the phenomenon of low-cycle fatigue.
4. It incorporates a damage scale derived on the basis of extensive parametric studies.
5. This method allows the designer to either determine the damage level for a given structure under any given seismic load, or dimension a structure for given seismic load and desired level of damage, or determine the maximum seismic load a designed structure can sustain in order to exhibit a desired level of damage.

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Appendix

Performance Levels	IDR	θpl	μ_θ	Damage
I.O.	1-2% [16] 1.5% [3] 0.5% [17] 0.7 % transient negligible permanent [15]	$\leq\theta_y$ [15]	2 [15]	≤5% [17] 0.1-10% [18]
L.S.	2-3% [16] 3.2 % [3] 1.5% [17] 2.5 % transient 1% permanent [15]	$\leq 6\theta_y$ [15]	7 [15]	≤20% [17] 10-30% [18]
C.P.	3-4% [16] 3.8%[3] 3% [17] 5 % transient 5% permanent [15]	$\leq 8\theta_y$ [15]	9 [15]	≤50% [17] 30-60% [18]

Table A.1. Performance levels and corresponding limit response values given by several authors.

ΕΝΑΣ ΝΕΟΣ ΔΕΙΚΤΗΣ ΒΛΑΒΗΣ ΓΙΑ ΤΗΝ ΣΕΙΣΜΙΚΗ ΕΠΙΤΕΛΕΣΤΙΚΟΤΗΤΑ ΤΩΝ ΜΕΤΑΛΛΙΚΩΝ ΚΑΤΑΣΚΕΥΩΝ

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Περίληψη: Παρουσιάζεται ένα νέος δείκτης βλάβης για επίπεδες μεταλλικές κατασκευές που υφίστανται σεισμική φόρτιση. Ο δείκτης αυτός ορίζεται στη διατομή ενός μέλους και λαμβάνει υπόψη του την αλληλεπίδραση μεταξύ της καμπτικής ροπής M και της αξονικής δύναμης N που δρούν στην υπό εξέταση διατομή. Η αλληλεπίδραση αυτή ορίζεται με δύο χαρακτηριστικές καμπύλες στο επίπεδο M-N όπου η πρώτη αντιστοιχεί στην μεταβατική κατάσταση μεταξύ της ελαστικής και της ανελαστικής συμπεριφοράς, με την βλάβη να είναι ίση με μηδέν, ενώ η δεύτερη αντιστοιχεί στην οριακή κατάσταση της πλήρους αστοχίας της διατομής, όπου η βλάβη ισούται με μονάδα. Ο προτεινόμενος δείκτης ορίζεται θεωρώντας γραμμική μεταβολή της βλάβης μεταξύ των δύο χαρακτηριστικών καμπυλών. Η μη-γραμμικότητα υλικού λαμβάνεται υπόψη με τη θεωρία συγκεντρωμένης πλαστικότητας εξετάζοντας επίσης τις απομειώσεις αντοχής και δυσκαμψίας καθώς και την ολιγοκυκλική κόπωση. Η γεωμετρική μη-γραμμικότητα λαμβάνεται υπόψη εξετάζοντας την επιρροή των μεγάλων μετατοπίσεων και τα φαινόμενα P-δ και P-Δ. Τέλος, παρουσιάζονται χαρακτηριστικά παραδείγματα από τα οποία γίνονται φανερές οι δυνατότητες της προτεινόμενης μεθοδολογίας ενώ ταυτόχρονα αποδεικνύεται η αποτελεσματικότητα της μέσω της σύγκρισης με υπάρχοντες δείκτες βλάβης.