Engineering Structures 152 (2017) 424–436

Contents lists available at ScienceDirect

Engineering Structures

journal homepage: www.elsevier.com/locate/engstruct

Energy damage index based on capacity and response spectra

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ARTICLE INFO

Article history:
Received 17 March 2017
Revised 3 September 2017
Accepted 12 September 2017

Keywords:
Capacity curve
Damage assessment
Strain energy
Energy dissipated by hysteresis
Monte Carlo simulations

ABSTRACT

Non-linear dynamic analysis and the damage index of Park-Ang have been often used to assess expected seismic damage to a structure. Depending on the size of the structure and the duration of the record, the computational effort in dynamic analyses is usually high. In this research, a new damage index is proposed based on nonlinear static analysis. The damage index is a linear combination of two energy functions: (1) the strain energy associated with the stiffness variation and the ductility of the structure, and (2) the dissipated energy associated with hysteretic cycles. These two energy functions are obtained from the capacity curve of the structure and from the energy balance with the spectral acceleration. To show the ability of the index to represent damage, low-rise steel buildings were studied under the seismic actions that are expected in Mexico City. The results obtained with the new method show good agreement with those calculated by means of dynamic analyses using the Park-Ang damage index. On average, the Park-Ang damage index is well-fitted by the combination of 62% of the strain energy and 38% of the energy dissipated by hysteresis. Moreover, the new damage index can link damage to certain characteristics of seismic actions, such as their intensity and duration. Therefore, the new approach results in a practical, powerful tool for estimating seismic damage in buildings, especially as probabilistic approaches require massive computations.

1. Introduction

In assessments of the seismic performance of buildings, non-linear dynamic analysis (NLDA) has proved to be the most realistic, suitable, sophisticated, numerical tool to estimate the response of a structure as a function of time. When NLDA is used to assess the seismic response, the input is generally a group of accelerograms that can be recorded, synthetic or both. If NLDA is performed by increasing the ordinates of the selected accelerograms, it is known as incremental dynamic analysis (IDA) [1]. IDA can be used to obtain curves relating a measure of the seismic response of a structure (displacement at the roof, maximum inter-story drift, etc.) to a variable that describes seismic intensity, such as peak ground acceleration (PGA). The IDA has been used as the most appropriate tool for assessing damage in structures subjected to dynamic actions [1]. Several damage indices can be calculated from the dynamic response of a structure [2,3], and are related to a reduction in the capacity of buildings' structural elements. Some studies have proposed damage indices for reinforced concrete and steel buildings, considering parameters such as displacement ductility [4,5], strength and stiffness degradation [3], energy dissipation [6,7], cyclic fatigue [8], change in the natural period of the structure [9], or a combination of the above parameters [10–13]. Most of the damage indices proposed to date take values in the range of 0 to 1, where 0 indicates no damage and 1 collapse. Park and Ang [11] proposed one of the most frequently used seismic damage indices for reinforced concrete buildings, which considers both the maximum structural response and the cyclic load effect [14–16].

Considerable computational effort is required to calculate damage curves based on IDA. To avoid this effort, non-linear static analysis (NLSA) offers an interesting alternative due to its simplicity [17,18], but the results must be in good agreement with those provided by IDA. Several researchers have employed NLSA to estimate parameters related to the dynamic response of structures [19–23] or in risk studies at urban level [24–27]. In the present article, a new damage index for steel buildings is proposed that can be obtained from the capacity curve. It fits well with the damage index of Park and Ang. The mathematical formulation of the new damage index is based on energy functions and on the idea proposed by Pujades et al. [22] of using a calibration parameter to determine the contribution to damage of two or more simple functions and, thus, to obtain good agreement with a relatively more

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https://doi.org/10.1016/j.engstruct.2017.09.019

Nomenclature

Ac	area under the capacity curve
acc	accelerograms
$ADE(\delta)$	accumulated deformation energy of the capacity curve
$b_f/(2 \cdot t_f)$	width/thickness ratio of the beam flange of W section
COV	coefficient of variation of the probabilistic variables
c^1_{unit} and c^2_{unit}	coefficients for units conversion in the modified IMK model
D_{bi}, F_{bi}	coordinates of the ultimate capacity point of the bilinear curve
D_{ci}, F_{ci}	coordinates of the ultimate capacity point of the capacity curve
$DI_{EC}(\delta)$, $DI_{EC}(\theta)$ or $DI_{EC}(PGA)$	energy capacity damage index in function of the roof displacement, rotation and PGA, respectively
$DIe_{PA}(\delta)$ or $DIe_{PA}(\theta)$	Park and Ang damage index of a structural element
$DI_{PAw}(\delta)$, $DI_{PAw}(\theta)$ or $DI_{PAw}(PGA)$	Park and Ang damage index of a building in function the roof displacement, rotation and PGA, respectively
$\int_0^\delta dE$	hysteretic energy absorbed by the element during the earthquake
D_y, F_y	coordinates of the yield point of the bilinear curve
E	modulus of elasticity
EDRS	Energy Displacement Response Spectrum
E_D	energy dissipated by the structure in a single cycle of motion
$E_D(\delta)$	energy dissipated function
$E_D(\delta)_{NN}$, $E_D(\theta)_{NN}$ or $E_D(PGA)_{NN}$	normalized energy dissipated in function of the roof displacement, rotation or PGA, respectively
E_{so}	maximum strain energy associated to a cycle of motion
$E_{so}(\delta)$	strain energy function
$E_{so}(\delta)_{NN}$, $E_{so}(\theta)_{NN}$ or $E_{so}(PGA)_{NN}$	normalized strain energy in function of the roof displacement, rotation or PGA, respectively
E_y	yielding energy
$F(\delta)$	capacity curve
FR	connections type fully restrained
f_y	expected yield strength
h/t_w	ratio between the web depth and the thickness of W section
i	structural element *i*
I	inertia moment of W section
IDA	incremental dynamic analysis
IMK	modified Ibarra–Medina–Krawinkler model
j	each increment in the displacement of the capacity curve
k	residual moment constant
Ki	initial slope of the capacity curve
k_o	initial elastic stiffness
L/d	the ratio between the span and the depth of the beam or column
LHS	Latin Hypercube Sampling
M	bending moment in the structural element
M_c	capping moment strength or post-yield strength ratio
M_r	residual moment
M_w	moment magnitude scale
M_y	effective yield moment
$M*$	effective modal mass for the first mode of vibration of the building
N	number of damaged structural elements in the building
n	ultimate increment in the displacement of the capacity curve
NLDA	nonlinear dynamic analysis
NLSA	nonlinear static analysis
PA	Park and Ang damage index
P_{ad}	adaptive pushover analysis
PF_1	modal participation factor
PGA	peak ground acceleration
Qu	strength corresponding to the ultimate displacement
Qy	strength at the yielding point
R_y	strength reduction factor
Sa	acceleration spectrum
Sa_{EDRS}	input energy spectrum
$Sa_{matched}$	acceleration spectrum of the matched accelerogram
Sd	spectral displacement in the structure
Sd_{pp}	spectral displacement of the performance point
Sd_y	yielding spectral displacement
SMF	special moment frame building
Sv	velocity spectrum
T	structural period
T_a, T_b and T_c	limit periods used to define the Ry-μ_s-T relationship
T_1	fundamental period of the building
$T1_{SFM3}$ prob	the fundamental period of the probabilistic models SMF 3
V	base shear in the structure
V_y	base shear in the yielding energy
Z	plastic modulus
β	strength deteriorating parameter in the Park and Ang damage index
$\beta.*$	parameter of the Ry in function of the T_a, T_b and T_c
γ_E	energy factor
δ	roof displacement in the structure
δ_{Dy}	displacement in the yielding point of the bilinear curve
δ_u	ultimate roof displacement in the structure
δ_y	roof displacement in the yielding energy
η	calibration parameter in the energy capacity damage index
θ	rotation in the structural element
θ_p	pre-capping plastic rotation for monotonic loading
θ_{pc}	post-capping plastic rotation
θ_u	ultimate rotation capacity
θ_y	yield rotation
λ_i	ratio of the energy dissipated by hysteresis in the element *i* to the total hysteretic energy dissipated in the entire building
μ	mean value of the probabilistic variables
μ_E	energy ductility
μ_{PP}	ductility of the performance point
μ_s	ductility factor
ξ_{eq},	equivalent viscous damping
σ	standard deviation of the probabilistic variables
σ_{In}	standard deviation, assuming a lognormal fit of experimental data in θ_p and θ_{pc} in the modified IMK model
ω	tangent fundamental natural frequency in the modified Rayleigh method

complex damage index. Nevertheless, new functions that consider two types of energy from the capacity curve are used herein: (1) strain energy and (2) energy dissipated by hysteresis [28,29]. When both functions are combined, a new damage index is obtained that is compatible with that of Park and Ang. In order to consider the effect of the seismic hazard, the performance point is based on the concept of energy balance [30] and the application of the seismic evaluation is based on the study by Leelataviwat