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Performance-based seismic assessment of steel frames using endurance time analysis

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ABSTRACT

The current performance-based seismic assessment procedure can be computationally intensive as it requires a large number of time history analyses (THA) each requiring time intensive post-processing of results. This study proposes the endurance time analysis (ETA) method as an alternative method to THA and incremental dynamic analysis (IDA). ETA is a time history based dynamic pushover procedure that applies a set of gradually intensifying acceleration functions to the structure and monitors the performance of the building accordingly. In this paper, the application of ETA in the seismic assessment of multistory steel concentrically braced frames is compared with THA and IDA methods. Moreover, the progressive failure of the frames is investigated using the ETA method. The results of this analysis show that ETA can estimate THA as well as IDA, with considerably less computational effort.

1. Introduction

Depending on the importance of the structure and the seismicity condition of a site, several different types of seismic analysis and design methods can be used. Limitations of traditional seismic analysis procedures and recent progress in computational technology have motivated researchers during the past years to develop new analysis methods. The performance evaluation of an existing structural system can be conducted using different analysis methodologies. The ways in which these methods vary includes capturing the seismic response of the structure, analysis procedure, computational effort, accuracy, and overall capabilities of the analysis. Fig. 1 summarizes some of the available methods that can be implemented in seismic analysis and performance assessment of structures.

One of the common methodologies is time history analysis (THA), which considers the fundamental characteristics of the input ground motion in the analysis procedure. By comparison, this is neglected in the response spectrum analysis (RSA) method. Different sources of nonlinearity in the response of the structure including material and geometry nonlinearities can be considered using THA. However, the response of the system is highly dependent on ground motion characteristics, especially when performing a nonlinear analysis. Another method used in performance-based seismic analysis and structural design is nonlinear static analysis (NSA) [1]. Two major NSA procedures include displacement ductility evaluation (DDE) and pushover analysis (POA). In DDE, the displacement ductility demand of the structure is estimated based on a linear elastic response spectrum analysis. Since all inelastic action will be due to the flexural response, the elastic moment demand and the nominal moment capacity of the section are used to determine the displacement ductility demand on the structure. For complex structures, where plastic hinges can form in several locations over the height of the building, POA (collapse mechanism analysis) should be used to assess the actual performance of the structure [2,3]. POA is a nonlinear static procedure in which the magnitude of applied load/displacement is increased incrementally according to a predefined pattern. The analysis continues until a control point on the structure reaches a target displacement. POA is capable of mobilizing principal nonlinear modes of structural behavior up to collapse of the structure. The results may depend, however, on the chosen pattern of the imposed load.

In order to consider the inherent randomness of the ground motions and reducing dependency of responses to seismic inputs, two other groups of nonlinear dynamic analyses are discussed in this section. These are referred to as wide-range analyses and narrow-range analyses. The wide-range analysis is suitable for making probabilistic assessments of the structural response over a wide range of tolerable probability levels, while narrow-range analyses are appropriate for making probabilistic assessments for a tight

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Nomenclature

a	first constant coefficient in the recurrence–magnitude relation
a_g	endurance time acceleration parameter
$A(i\omega)$	filtered acceleration function
b	second constant coefficient in the recurrence–magnitude relation
C	collapse
$C'_1, C'_2, C'_3, C'_4, C'_5$	coefficients of PGA-based GMPE
C_1, C_2, C_3	coefficients of spectral acceleration-based GMPE
CC	complete collapse
CFC	collapse fragility curve
$\mathrm{CFC}^{\mathrm{ETA}}$	ETA-based collapse fragility curve
$\mathrm{CFC}^{\mathrm{IDA}}$	IDA-based collapse fragility curve
CLA	cloud analysis
CP	collapse prevention
DCR	demand to capacity ratio
DDE	displacement ductility evaluation
DSA	double-stripe analysis
DL	damage level
EDP	engineering demand parameter
$\mathrm{EDP_c}$	capacity measured in terms of the EDP
$\mathrm{EDP_d}$	demand measured in terms of the EDP
EDP_j	value of EDP for jth ground motion
$\overline{\mathrm{EDP}}$	mean value of EDP
$\overline{\mathrm{EDP}}_{\mathrm{ETA}}$	mean of maximum EDP*s* computed based on ETA
$\overline{\mathrm{EDP}}_{\mathrm{THA}}$	mean of maximum EDP*s* computed based on THA
$\mathrm{EDP}_{\mathrm{linear}}$	first-order trend-line in the ETA curve
$\mathrm{EDP}_{\mathrm{3rdorder}}$	third-order trend-line in ETA curve
$\mathrm{edp}_{\mathrm{c}_i}$	specified capacity value of EDP
$\mathrm{edp_d}$	specified demand value of EDP
Err%	percentage difference between $\overline{\mathrm{EDP}}_{\mathrm{ETA}}$ and $\overline{\mathrm{EDP}}_{\mathrm{THA}}$
ETA	endurance time analysis
ETAF	endurance time acceleration function
EWA	endurance wave analysis
FEMA	federal emergency management agency
$F(a_g)$	optimization function
GM1	first set of real ground motions
GM2	second set of real ground motions
GMPE	ground motion prediction equation
$g(t)$	stationary random acceleration function
$H_1(i\omega)$	Clough and Penzien low-pass filter function
$H_2(i\omega)$	Clough and Penzien high-pass filter function
IDA	incremental dynamic analysis
IM	intensity measure
$\mathrm{IM_c}$	intensity measure corresponding to the collapse capacity
im_i	intensity measure at the given seismic level
INBC	Iranian national building code
IO	immediate occupancy
i	imaginary unit
j	dummy index
k	dummy index
LS	life safety
$l(t)$	linear profile function
MSA	multi-stripe analysis
MLE	maximum likelihood estimation
m	number of intensity levels
M_S	surface wave magnitude
M	magnitude
$M(t)$	modulating function
NEHRP	national earthquake hazards reduction program
NSA	nonlinear static analysis
N_{DL}	number of damage levels
N_m	annual exceedance probability
n_j	number of ground motions used in MLE formulation
OP	operational
PBA	performance-based assessment
PBEE	performance-based earthquake engineering
PBSA	performance-based seismic assessment
PL	performance level
PO	performance objective
POA	pushover analysis
PGA	peak ground acceleration
$\mathrm{PGA_{ETAF}}$	PGA associated with ETAF
PGV	peak ground velocity
P–Δ	large deformation effects
$P[C\|\mathrm{IM}]$	probability of collapse at a given IM
RTR	record-to-record
RSA	response spectrum analysis
r	number of total time steps in generating an ETAF
R	source-to-site distance
R^2	coefficient of determination
SCBF	steel concentrically braced frame
SSA	single-stripe analysis
SRN	stationary random nature
s	number of real ground motions or ETAFs
S_a	spectral acceleration
$S_a^{T_R,T_1}$	average of the spectral acceleration over real ground motions at the first-mode period of the structure
S_a^{ETAF,T_1}	smoothed response spectrum used for the generation of ETAFs at the first-mode period of the structure
$S_a(T_1)$	spectral acceleration at the structure's first-mode period
S_a^{target}	target acceleration response spectrum
$S_a^{\mathrm{generated}}$	generated acceleration response spectrum
$S_{ac}(T)$	target acceleration response for structure with period T
$S_{ac}(T,t)$	target acceleration response at time t for structure with period T
$S_{uc}(T,t)$	target displacement response value for period T at time t
$S_a(T,t)$	ETAF acceleration response value for period T at time t
$S_u(T,t)$	ETAF displacement response value for period T at time t
$S_a(T_1)_{\mathrm{ETAF}}$	$S_a(T_1)$ associated with ETAF
STD	standard deviation of EDPs
THA	time history analyses
t	time
t_{eq}	equivalent target time
t_{target}	target time
$t_{\max}$	maximum duration of ETAFs
T	the natural period of structure
T_1	the first-mode translational period
T_R	return period
T_{eff}	effective period interval
$T_{\max}$	maximum period in the optimization process
$\mathrm{Time_{ETAF}}$	time associated with ETAF
UBC	uniform building code
x_j	specific level of intensity measure
$Z(t)$	non-stationary random acceleration function
z_j	number of collapses used in MLE formulation
$\alpha_0, \alpha_1, \alpha_2, \alpha_3$	trend-line coefficients in an ETA curve
β	standard deviation of $\ln S_a$
$\hat{\beta}$	estimated logarithmic standard deviation value
β^{IDA}	dispersion of collapse capacities in IDA-based method
β^{ETA}	dispersion of collapse capacities in ETA-based method
β_{RTR}	record-to-record uncertainty in IDA-based method
β_{SRN}	stationary random nature uncertainty in ETA-based method
χ_0	relative penalty in optimization function (weight parameter)
δt	time step used for generation of an ETAF
μ	logarithmic mean of S_a
$\hat{\mu}$	estimated logarithmic mean value