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SEISMIC-PARAMETER-BASED STATISTICAL PROCEDURES FOR THE ESTIMATION OF STRUCTURAL DAMAGE

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ABSTRACT

Two statistical methods are presented for the evaluation of the postseismic damage status of buildings based on seismic intensity parameters. The first is multilinear regression analysis, and the second is discriminant analysis. Twenty seismic parameters have been extracted from the accelerograms employed. The overall damage index of Park and Ang, the maximum inter-story drift ratio, and the maximum softening of DiPasquale and Cakmak have been used to describe the structural damage status. The multilinear regression has been utilized for the explicit evaluation of the damage indices. The discriminant analysis furnished the postseismic damage grade of the structures. The proposed statistical methods have been applied to an eight-story reinforced concrete frame structure. A set of 400 natural accelerograms has been implemented for the training phase of the models. Another second set of synthetic accelerograms has been used to verify the statistical methods. The results have shown that 94.75–97.50% and 70–90% of the first and second sets of accelerograms, respectively, were correctly classified. The damage grades for 87.75–97.50% and 70–90% of the first and second sets of accelerograms, respectively, were correctly predicted by the discriminant analysis. Thus, these results led to the conclusion that the multilinear regression analysis and discriminant analysis are useful tools for the prediction of the postseismic damage status when the seismic parameters are known.

Keywords: Damage indices; Seismic parameters; Multilinear regression; Discriminant analysis

1. INTRODUCTION

Several seismic parameters have been presented in the literature during the last several decades. These parameters can be used to express the intensity of the seismic excitations and to simplify its description. Post-seismic field observations and numerical investigations have indicated the interdependency between the seismic parameters and damage status of buildings after earthquakes (Cabanas et al. 1997, Elenas and Meskouris 2001). The latter can be expressed by proper damage indices, whereas the interdependency between the considered quantities can be quantified numerically by appropriate correlation coefficients.

This paper expands on the above, providing alternative statistical procedures for the approximate assessment of the structural post-seismic damage grade. Thus, the first step in the proposed new methodology is to choose a set of seismic parameters that represent the properties of the accelerograms. Next, the intensity parameters chosen in the first step are evaluated for a set of natural or artificial accelerograms. The damage indices for all considered accelerograms are subsequently calculated for the building of interest. Statistical analyses are applied to the resulting data from the previous steps to extract a statistical model for the prediction of the post-seismic damage status of structures. Finally, the new methodology is verified by a blind prediction.

As mentioned previously, the proposed statistical methods are used to estimate the post-seismic structural damage status. Approximation methods are well known and broadly used in earthquake engineering. The evaluation of the fundamental period of a building from its geometric dimensions is such an approximation provided by several antiseismic codes. Thus, the fundamental period can be

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approximated using a simple formula depending on the type of structure (e.g., steel, reinforced concrete, masonry, frame or wall) and the geometric dimensions of the examined building instead of needing to solve a complicated eigenvalue problem that depends on mass and stiffness matrices.
The proposed methodologies can be used, for instance, by public authorities for a quick and accurate estimation of the post-seismic damage status or for estimating damage scenarios for specific buildings. Another application is the implementation of the proposed procedure in an integrated circuit (microchip), which can be used for real-time damage estimation of important buildings after severe earthquakes when combined with a seismic accelerograph and wireless data transmission device. In all of these cases, the proposed techniques are simpler to apply than the more complicated and time-consuming nonlinear dynamic analyses. The novel application of the proposed statistical procedures in the specific field of earthquake engineering is also feasible for nonspecialists, in contrast to nonlinear dynamic analyses, which can only be performed by expert engineers.

2. SEISMIC INTENSITY PARAMETERS

In general, the intensity parameters can be classified into peak, spectral and energy parameters. Twenty parameters have been selected to represent the seismic excitation. They have been chosen from all three seismic parameter categories. The following seismic parameters are considered: peak ground acceleration (PGA); peak ground velocity (PGV); the ratio PGA/PGV; Arias intensity (I_0); root mean square acceleration (RMS_a); the strong motion duration of Trifunac/Brady ($T_{0.90}$); seismic power ($P_{0.90}$); the spectral intensities of Housner (SI_H); Kappos (SI_K); and Martinez-Rueda (SI_{MR}); effective peak acceleration (EPA); maximum effective peak acceleration (EPA_{max}); seismic energy input (E_{inp}); cumulative absolute velocity (CAV); the seismic damage potential of Araya/Saragoni ($DP_{A/S}$); central period (CP); spectral acceleration (S_A); spectral velocity (S_V); spectral displacement (S_D); and the intensity of Fajfar/Vidic/Fischinger ($I_{F/V/F}$). Table 1 provides an overview of the used parameters and the literature references, where their definitions are presented.

Table 1. Seismic parameters.

No	Seismic parameters	Reference
1	PGA	Meskouris 2000
2	PGV	Meskouris 2000
3	PGA/PGV	Meskouris et al. 1993
4	I_0	Arias 1970
5	RMS_a	Meskouris 2000
6	$T_{0.90}$	Trifunac and Brady 1975
7	$P_{0.90}$	Jennings 1982
8	SI_H	Housner 1952
9	SI_K	Kappos 1990
10	SI_{MR}	Martinez-Rueda 1998
11	EPA	ATC 3-06
12	EPA_{max}	ATC 3-06
13	E_{inp}	Uang and Bertero 1990
14	CAV	Cabanas et al. 1997
15	$DP_{A/S}$	Araya and Saragoni 1984
16	CP	Vanmarcke and Lai 1980
17	S_D	Chopra 1995
18	S_V	Chopra 1995
19	S_A	Chopra 1995
20	$I_{F/V/F}$	Fajfar et al. 1990

3. SEISMIC ACCELERATION TIME HISTORIES

The expected damage potential of a seismic excitation on a particular structure is the prime consideration for the selection of the accelerograms for the presented methodology. Seismic excitations that generate a broad spectrum of damage, from negligible to severe, are considered for statistical reasons. Thus, the present investigation utilized 400 worldwide natural acceleration records with strong seismic activity. Tables 2 and 3 provide the number of accelerograms used per country and PGA range. All of the above-mentioned seismic acceleration time histories have been investigated by a computer-supported evaluation of their seismic parameters, as presented in the previous section and Table 1.

Table 2. Number of accelerograms employed per country.

Country	Number of accelerograms
Bulgaria	2
Canada	9
Chile	12
Greece	54
Japan	46
Mexico	10
New Zeeland	2
Romania	4
San Salvador	6
Turkey	8
USA	247

Table 3. Number of accelerograms employed per PGA range.

PGA-range [g]	Number of accelerograms
0.01 - 0.1	23
0.1 - 0.2	165
0.2 - 0.3	103
0.3 - 0.4	50
0.4 - 0.5	32
0.5 - 0.6	13
0.6 - 0.7	7
> 0.7	7

4. DAMAGE INDICES

Among the many structural response parameters, attention is focused on those that can best describe the seismic damage. The attention is focused on overall structural damage indices (OSDIs) because these parameters summarily lump all existing damage in columns and beams into a single value, which can then be easily interrelated with the single-value seismic parameters. For this purpose, the modified overall damage index of Park and Ang (Park and Ang 1985, Park et al. 1987) is used in the present study. In this model, the global damage is obtained as a weighted average of the local damage at the ends of each element, with the dissipated energy as the weighting function. The local damage index according to Park/Ang must be calculated using the following equation:

$$DI_{P/A,Local} = \frac{\theta_m - \theta_r}{\theta_u - \theta_r} + \frac{\beta}{M_y \theta_u} E_T \tag{1}$$

where θ_m is the maximum rotation during the load history, θ_u is the ultimate rotation capacity of the section, θ_r is the recoverable rotation at unloading, β is a constant parameter (0.1-0.15 for nominal strength deterioration (Reinhorn et al. 2009)), M_y is the yield moment of the section, and E_T is the dissipated hysteretic energy.

In this study, the numerical value of parameter β in equation (1) is equal to 0.1. This value corresponds to nominal strength degradation (Reinhorn et al. 2009). However, the numerical value of β can vary in the range between 0 (no strength degradation) and 0.4 (severe deterioration) (Reinhorn et al. 2009). A low β value corresponds to well-detailed reinforced concrete or steel members (Chai et al. 1994). In contrast, a high β value corresponds to poorly detailed reinforced concrete members or unreinforced masonry (Park et al. 1987).

The OSDI of Park/Ang is defined by the following equation:

$$DI_{P/A,Global} = \frac{\sum_{i=1}^{n} DI_{P/A,Local} E_i}{\sum_{i=1}^{n} E_i} \tag{2}$$

where E_i is the energy dissipated at location i and n is the number of locations at which the local damage is computed.

The maximum inter-story drift ratio (MISDR) has been selected as a second OSDI. It is defined as the maximum inter-story drift (u_{max}) normalized by the story height (h), as given by relation (3):

$$MISDR = \frac{|u_{max}|}{h} 100\ [\%]. \tag{3}$$

The maximum softening index of DiPasquale/Cakmak (DiPasquale and Cakmak 1987) has been selected as a third OSDI, which is based on the vibration parameters of the structure. It is given by the following expression:

$$\delta_M = 1 - \frac{T_0}{T_{max}} \tag{4}$$

where δ_M is the maximum softening, T_0 is the fundamental period, and T_{max} is the maximum natural period of the examined structure during the excitation. For the evaluation of T_{max}, a sliding time window can be used for the smoothness of the time-fundamental period curve (DiPasquale and Cakmak 1987, Rodriguez-Gómez and Cakmak 1990). The time-fundamental period curve can be evaluated by a nonlinear dynamic procedure that calculates the fundamental period of the structure by considering the stiffness degradation in every time step. T_0 is the initial, fundamental period of the structure that corresponds to the initial stiffness of the structure.

Table 4 presents the range limits of the four damage classes using the different damage indices (Gunturi and Shah 1992). The four levels (low-medium-large-total) correspond to undamaged or minor damage, repairable damage, irreparable damage, and the partial or total collapse of the structure.

Table 4. Damage classification limits.

	Damage classes			
	Low	Medium	Large	Total
OSDI of Park/Ang and of DiPasquale/Cakmak	≤ 0.3	0.3 < OSDI ≤ 0.6	0.6 < OSDI ≤ 0.8	> 0.8
MISDR [%]	≤ 0.5	0.5 < MISDR ≤ 1.5	1.5 < MISDR ≤ 2.5	> 2.5

5. APPLICATION

The proposed procedures have been applied to a reinforced concrete structure, shown in Figure 1 and designed in agreement with the rules of the recent Eurocodes for structural concrete and aseismic structures, Eurocode 2 and Eurocode 8 (EC2 2000, EC8 2004). The fundamental period of the frame was 1.20 s. The following loads have been considered: self-weight, seismic loads, snow, wind, and live loads. The cross-sections of the beams are treated as T-beams with a 40 cm width, 20 cm plate thickness, 50 cm total beam height, and 1.45 m effective plate width. The distances between each frame of the structure have been chosen to be 6 m. According to the EC8 Eurocode, the structure shown in Figure 1 is considered an "importance class II, ductility class M" structure with category B subsoil. The ordinate of the used elastic response spectrum of Eurocode 8 was 0.24g.

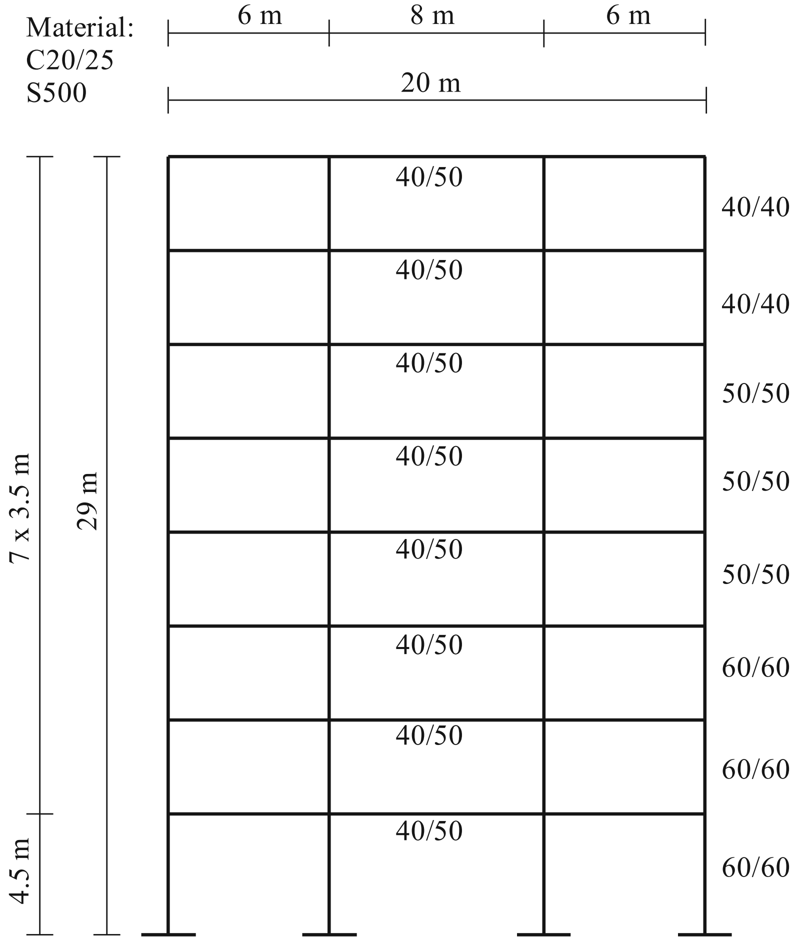

Figure 1. Reinforced concrete frame structure

After designing the frame structure, nonlinear dynamic analyses were carried out to evaluate the structural response. For this purpose, the computer program IDARC (Reinhorn et al. 2009) has been used. The hysteretic behavior of the beams and columns has been specified at both ends of each member using a three-parameter Park model (Reinhorn et al. 2009). This hysteretic model incorporates stiffness degradation, strength deterioration, nonsymmetric response, slip-lock, and a trilinear monotonic envelope. The parameter values that specify the above degrading parameters have been chosen from the cyclic force-deformation characteristics from experimental results of the typical components of the studied structure. Thus, nominal parameters for stiffness degradation and strength deterioration have been chosen. No pinching has been considered. Of the several response parameters, the focus is on the OSDIs. Consequently, the OSDI of Park/Ang and DiPasquale/Cakmak and the MISDR have been calculated for all of the natural accelerograms corresponding to all the excitations, which were used as the seismic input for the nonlinear dynamic analyses. For the evaluation of the OSDI of DiPasquale/Cakmak according to equation (4), a sliding time window of 2.5 s is used to smooth the time-fundamental period curve. This window size is between 2 and 5.5 times the fundamental period T_0 of the examined structure (T_0 = 1.20 s), as suggested by Rodriguez-Gómez and Cakmak (1990). The maximum value of the smoothed time-fundamental period curve is the maximum natural period of the examined structure during each seismic excitation considered (T_{max} in equation (4)).

6. MULTILINEAR REGRESSION ANALYSIS

In general, a multilinear regression analysis (Montgomery and Runger 2003) is used to express a dependent variable (y) by (n) independent variables (x_i, with i = 1, …, n) and a constant value (a) minimizing the error term (e), as shown by the relation:

$$y = a + b_1 x_1 + b_2 x_2 + ... + b_n x_n + e. \qquad (5)$$

In the present study, the dependent variable (y) represents each used OSDI (of Park/Ang and DiPasquale/Cakmak and the MISDR), and the independent variables (x_i) are the twenty seismic parameters (n = 20) shown in Table 1.

The analyses have been carried out using the statistical software STATGRAPHICS (Statpoint Technolgies Inc. 2017). The regression result for the OSDI of Park/Ang is given in Table 5. The equation of the fitted multilinear model is:

$$DI_{P/A,Global} = -0.032 + 0.370PGA + 0.084PGV + 0.005PGA/PGV + 0.007Arias - 0.074RMS_a - 0.0004T_{0.90} - 0.004P_{0.90} + 0.036SI_H - 0.014SI_K - 0.379SI_{MR} - 0.385EPA + 0.076EPA_{max} + 0.037E_{inp} - 0.020CAV - 0.202DP_{A/S} + 0.038CP - 0.110S_D + 0.167S_V + 0.031S_A + 0.017I_{F/V/F} \qquad (6)$$

The R^2 = 0.858899, which indicates that the model as fitted explains 85.9% of the variability in the Park/Ang OSDI. In addition, Table 5 presents the 95% confidence intervals for the coefficients in the multilinear model. Each variable coefficient in equation (6) and in Table 5, or generally in equation (5), is interpreted as the change in the response based on a one-unit change in the corresponding explanatory variable keeping all other variables fixed. This interpretation is fictitious because it is not possible in a seismic accelerogram to change only one of the seismic parameters. Furthermore, in some explanatory variables (seismic parameters), a one-unit change can be easily observed (such as the SMD of Trifunac/Brady), whereas such a change cannot be easily observed in others (such as the S_D). In addition, the comparison between coefficients of different explanatory variables is not possible, because they are assigned quantities having different dimensions and units (Navidi 2011). Finally, the constant term ("a" in equation (5) and "-0.032" in equation (6)) has a physical meaning only in the case in which all of the explanatory variables x_i can simultaneously have zero values. Otherwise, as in the present study in which the seismic parameters with zero values are out of the observed value range, the constant term is an extrapolated value without a physical meaning. Similarly, Table 6 presents the multilinear model for the MISDR (R^2 = 0.799279) and Table 7 presents the model for the OSDI after DiPasquale/Cakmak (R^2 = 0.763627), along with their 95% confidence intervals.

Table 5. Multilinear regression model for the OSDI after Park/Ang.

Parameter	Estimate	Lower Limit	Upper Limit
CONSTANT	-0.032	-0.063	-0.001
PGA	0.370	0.234	0.507
PGV	0.084	-0.030	0.199
PGA/PGV	0.005	-0.004	0.014
I_0	0.007	0.004	0.009
RMS_a	-0.074	-0.156	0.008
$T_{0.90}$	0.0004	-0.001	0.001
$P_{0.90}$	-0.004	-0.015	0.006
SI_H	0.036	-0.004	0.077
SI_K	-0.014	-0.127	0.099
SI_{MR}	-0.379	-0.543	-0.215
EPA	-0.385	-0.652	-0.118
EPA_{max}	0.076	0.004	0.148
E_{inp}	0.037	0.013	0.061
CAV	-0.020	-0.052	0.011
$DP_{A/S}$	-0.202	-0.437	0.032
CP	0.038	-0.022	0.098
SD	-0.110	-0.428	0.207
SV	0.167	0.074	0.260
SA	0.031	-0.092	0.154
$I_{F/V/F}$	0.017	-0.018	0.051

Table 6. Multilinear regression model for the MISDR.

Parameter	Estimate	Lower Limit	Upper Limit
CONSTANT	-0.072	-0.290	0.147
PGA	2.331	1.380	3.281
PGV	0.359	-0.438	1.156
PGA/PGV	0.046	-0.017	0.109
I_0	0.043	0.028	0.058
RMS_a	-0.322	-0.893	0.249
$T_{0.90}$	0.003	-0.004	0.010
$P_{0.90}$	-0.019	-0.089	0.052
SI_H	0.410	0.129	0.691
SI_K	-0.520	-1.308	0.268
SI_{MR}	-2.726	-3.864	-1.587
EPA	-0.873	-2.731	0.985
EPA_{max}	0.019	-0.483	0.522
E_{inp}	-0.048	-0.216	0.120
CAV	-0.397	-0.619	-0.175
$DP_{A/S}$	1.475	-0.155	3.104
CP	-0.167	-0.583	0.250
SD	-2.292	-4.501	-0.084
SV	1.802	1.155	2.449
SA	0.236	-0.619	1.090
$I_{F/V/F}$	0.024	-0.217	0.264

Table 7. Multilinear regression model for the OSDI of DiPasquale/Cakmak.

Parameter	Estimate	Lower Limit	Upper Limit
CONSTANT	-0.090	-0.130	-0.050
PGA	0.293	0.119	0.466
PGV	0.094	-0.052	0.239
PGA/PGV	0.017	0.005	0.028
I_0	0.001	-0.002	0.004
RMS_a	-0.035	-0.140	0.069
$T_{0.90}$	0.0002	-0.001	0.001
$P_{0.90}$	-0.005	-0.018	0.008
SI_H	0.019	-0.032	0.071
SI_K	0.237	0.093	0.380
SI_{MR}	-0.178	-0.386	0.030
EPA	-0.288	-0.627	0.052
EPA_{max}	0.027	-0.065	0.118
E_{inp}	0.081	0.050	0.112
CAV	-0.032	-0.072	0.009
$DP_{A/S}$	-0.656	-0.953	-0.358
CP	0.097	0.021	0.173
SD	0.648	0.245	1.052
SV	-0.079	-0.197	0.039
SA	-0.040	-0.196	0.116
$I_{F/V/F}$	0.011	-0.032	0.055

The models, represented by the regression equations, are strictly valid only within the range of the data (in the present study these data are the values of the seismic parameters employed) used to develop these regression equations (training phase). Trying to predict the outcome outside the range of the data can be seriously misleading and is not advised.

7. TESTING THE MULTILINEAR REGRESSION MODEL

Two sets of accelerograms have been used to verify the quality of the regression model. The first one is identical to the 400 accelerograms used for the training phase of the regression model, as presented in section 3. The second set is from an entirely different set of 10 artificial accelerograms. In their original form, the latter is compatible with the design spectrum employed. Some of these accelerograms have been scaled to produce medium, large, and total structural damages for each of the examined damage indicators. The program SeismoArtif (Seismosoft 2016) has been utilized to create the aforementioned artificial accelerograms. The method used by the program for the artificial seismic accelerogram generation is the superposition of sinusoids having random phase angles and amplitudes derived from a stationary power spectral density function of the motion. The produced signals are then enveloped in a trapezoidal shape to simulate the non-stationary characteristics of the ground motion (Seismosoft 2016). The following input data are required for the generation of the artificial accelerograms: peak ground acceleration (PGA), total duration (TD), duration of the ascending and the descending part of the trapezoidal envelope, and design spectrum with which the artificial accelerograms must be compatible. Another option is to use natural or artificial spectral matching (Abrahamson 1992) accelerograms instead of the artificial accelerograms to verify the proposed models. Because the examined frame is designed using the same spectrum to which the artificial accelerograms are compatible, the expected damage grade will be low. However, the proposed statistical models must also be verified for medium, large, and total damage grades. Therefore, some of the artificial accelerograms have been scaled (the ordinates of seismic accelerogram have been multiplied by a factor with a value greater than one) to achieve medium, large, and total damage grades.

Table 8 presents the prediction results of all of the examined damage indicators. In this context, correct prediction means that the evaluated damage index lies within the 95% confidence interval of the multilinear regression model, as provided in Tables 5-7. Thus, the results the regression model display correct predictions, from 94.75% to 97.50% and from 80% to 90% for acceleration sets 1 and 2, respectively.

Table 8. Damage prediction results for the regression models.

	Correctly predicted damage values	
OSDI	**Set 1 400 accelerograms**	**Set 2 10 accelerograms**
$DI_{P/A,Global}$	386 (96.50%)	9 (90%)
MISDR	390 (97.50%)	8 (80%)
δ_M	379 (94.75%)	8 (80%)

8. DISCRIMINANT ANALYSIS

In the case for which it is sufficient to know the post-seismic damage grade as provided in Table 4 (low, medium, large, total), then a statistical discriminant analysis can be performed. The purpose of the discriminant analysis is to classify objects into one or more possible groups based on a set of features that describe the objects (Sharma 1996). In general, we assign an object to one of some predetermined groups based on observations made about the object.
The principle is to find the discriminant functions for each group. The element to be classified belongs to the group with the greatest value of the discriminant functions. In the present study, four groups and twenty independent parameters are used. Thus, the discriminant functions are given by the relations:

$$\begin{aligned} F_1 &= a_{0,1} + a_{1,1}X_{1,1} + a_{2,1}X_{2,1} + \ldots + a_{20,1}X_{20,1} \\ F_2 &= a_{0,2} + a_{1,2}X_{1,2} + a_{2,2}X_{2,2} + \ldots + a_{20,2}X_{20,2} \\ F_3 &= a_{0,3} + a_{1,3}X_{1,3} + a_{2,3}X_{2,3} + \ldots + a_{20,3}X_{20,3} \\ F_4 &= a_{0,4} + a_{1,4}X_{1,4} + a_{2,4}X_{2,4} + \ldots + a_{20,4}X_{20,4} \end{aligned} \tag{7}$$

where F_i is the discriminant function, $a_{j,i}$ are parameters to be specified, and $X_{j,i}$ are known quantities. Here, i = 4 (four damage grade groups) and j = 20 (twenty seismic parameters).

9. TESTING THE DISCRIMINANT ANALYSIS MODEL

The same two sets of accelerograms from the regression analysis have been used to verify the quality of the discriminate analysis. Table 9 presents the prediction results of the examined damage indicators. Correct prediction means that the damage grade estimate by the discriminant analysis agrees with the corresponding evaluation of the nonlinear dynamic analysis. The results display correct predictions from 87.75% to 97.50% and from 70% to 90% for acceleration sets 1 and 2, respectively.

Table 9. Damage prediction results for the discriminant analysis.

	Correctly predicted damage grade	
OSDI	**Set 1 400 accelerograms**	**Set 2 10 accelerograms**
$DI_{P/A,Global}$	390 (97.50%)	7 (70%)
MISDR	351 (87.75%)	8 (80%)
δ_M	374 (93.50%)	9 (90%)

10. CONCLUSIONS

Two statistical methods have been presented for the evaluation of the post-seismic damage status of buildings based on the parameters of the seismic excitation. The first is multilinear regression analysis, and the second is discriminant analysis. Twenty seismic parameters have been extracted from the accelerograms employed. The overall damage index of Park/Ang, the MISDR, and the maximum softening of DiPasquale/Cakmak have been used to describe the damage status of the structures. The multilinear regression has been used for the explicit evaluation of the damage indices. In contrast, the discriminant analysis furnished the post-seismic damage grade of the structures. These statistical procedures, combined with seismic intensity parameters, are novel methodologies in earthquake engineering and advantageous for the estimation of the damage indices and damage grades of buildings after severe seismic excitations.

The proposed statistical methods have been applied to an eight-story reinforced concrete frame structure designed by the rules of the EC2 and EC8 Eurocodes for reinforced concrete and antiseismic structures, respectively. A set of 400 natural accelerograms has been applied for the training phase of the models. In addition to the first set, the second set with synthetic accelerograms has been used to verify the statistical methods. The numerical results have shown that 94.75-97.50% and 70-90% of the first and second sets of accelerograms, respectively, were correctly classified by their damage potential for the examined structure within the confidence interval provided by the regression analysis. Further, the damage grades for 87.75-97.50% and 70-90% of the first and second sets of accelerograms, respectively, were correctly predicted by the discriminant analysis. Thus, these results led to the conclusion that the multilinear regression analysis and discriminant analysis are useful statistical tools for the estimation of the post-seismic damage status when the seismic parameters are known.

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