Research Article

Seismic-Parameter-Based Statistical Procedures for the Approximate Assessment of Structural Damage

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This study describes two statistical methodologies to estimate the postseismic damage status of structures based on seismic parameters as novel combined procedures in earthquake engineering. Thus, a multilinear regression analysis and discriminant analysis are utilized considering twenty seismic parameters. Overall damage indices describe the postseismic damage status. Nonlinear dynamic analyses furnish the damage indices, which are considered as exact indices and references for the subsequent study. The aim is to approximate the postseismic damage indices or the damage grade of buildings using statistical methods, thus avoiding complex nonlinear dynamic analyses. The multilinear regression procedure evaluates the damage indices explicitly, and the discriminant analysis furnishes the damage grade of the structures. The proposed methods are applied to a frame structure. A set of 400 natural accelerograms is used for the training phase of the models. The quality of the models is tested initially by the same set of natural accelerograms and then by a blind prediction using a second set of synthetic accelerograms. The results of both proposed methods have shown a correct classification percentage ranging from 87.75% to 97.50% and from 70% to 90% for the sets of the natural and synthetic accelerograms, respectively.

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 their description. Postseismic field observations and numerical investigations have indicated the interdependency between the seismic parameters and damage status of buildings after earthquakes [1, 2]. 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 Postseismic damage grade. Thus, the first step in the proposed new methodology is to choose a set of seismic parameters that represents 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 Postseismic 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 Postseismic 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 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 Postseismic damage status or for estimating damage

scenarios for specific buildings. 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 (SA); spectral velocity (SV); spectral displacement (SD); and the intensity of Fajfar/Vidic/Fischinger ($I_{F/V/F}$). Table 1 provides an overview of the used parameters and the literature references.

The Arias intensity [3] is a measure of the total energy content of seismic excitation and is defined by the following relation:

$$I_0 = \frac{\pi}{2g} \int_0^{t_e} (\ddot{x})^2 dt. \quad (1)$$

Here, I_0 is the Arias intensity, t_e is the total seismic duration, and $\ddot{x}$ is the seismic ground acceleration.

The root mean square acceleration RMS_a [4] is a measure of the mean energy content calculated by the following relation:

$$RMS_a = \sqrt{\frac{1}{t_e} \int_0^{t_e} (\ddot{x})^2 dt}. \quad (2)$$

The Husid diagram [5] is the time history of the seismic energy content scaled to the total energy content. It is defined by the following relation:

$$H(t) = \frac{\int_0^t (\ddot{x})^2 dt}{\int_0^{t_e} (\ddot{x})^2 dt}, \quad (3)$$

where $H(t)$ is the Husid diagram as a function of time t .

The strong motion duration of Trifunac/Brady is defined as the time elapsed between 5% and 95% of the Husid diagram [6] and is defined by the following relation:

$$T_{0.90} = T_{0.95} - T_{0.05}, \quad (4)$$

where $T_{0.90}$ is the strong motion duration and $T_{0.95}$ and $T_{0.05}$ are the time elapsed at 95% and 5% of the Husid diagram, respectively.

TABLE 1: Seismic parameters.

Number	Seismic parameters	Reference
1	PGA	[4]
2	PGV	[4]
3	PGA/PGV	[4]
4	I_0	[3]
5	RMS_a	[4]
6	$T_{0.90}$	[6]
7	$P_{0.90}$	[7]
8	SI_H	[8]
9	SI_K	[9]
10	SI_{MR}	[10]
11	EPA	[11]
12	EPA_{max}	[11]
13	E_{inp}	[13]
14	CAV	[1]
15	$DP_{A/S}$	[14]
16	CP	[15]
17	SD	[17]
18	SV	[17]
19	SA	[17]
20	$I_{F/V/F}$	[18]

The power $P_{0.90}$ [7] is a measure of the energy content per time unit of the seismic excitation and is defined by the following relation:

$$P_{0.90} = \frac{H_{0.95} - H_{0.05}}{T_{0.90}}. \quad (5)$$

Here, $P_{0.90}$ is the power of the seismic excitation, $H_{0.95}$ and $H_{0.05}$ the nonnormalized energy level, at 95% and 5% of the Husid diagram, respectively, and $T_{0.90}$ is the strong motion duration of Trifunac/Brady.

According to Housner [8], the spectrum intensity is given by the relation

$$SI_H = \int_{0.1}^{2.5} PSV(T, \xi = 2\%) dT, \quad (6)$$

where PSV is the pseudovelocity curve, T is the natural period of an single-degree-of-freedom (SDOF) system, and ξ is the damping coefficient. In the present work, the integral in (6) is evaluated for a damping ratio equal to 2% of the critical damping, as recommended by Housner when he introduced this quantity [8]. However, it is now generally accepted that the Housner spectral intensity is valid for any level of damping (e.g., 5% of critical damping) [7].

In the definition of the spectrum intensity after Kappos [9]:

$$SI_K = \int_{T_n - t_1}^{T_n + t_2} PSV(T, \xi) dT, \quad (7)$$

the integration limits are dependent on the fundamental period of the structure T_n and the period intervals t_1 and t_2 . Kappos suggested $t_1 = t_2 = 0.2T_n$.

Martinez-Rueda [10] introduced a spectral intensity definition accounting for the dynamic properties of the structure which is defined as

$$SI_{MR} = \frac{1}{T_h - T_y} \int_{T_y}^{T_h} PSV(T, \xi) dT, \quad (8)$$

where T_h the hardening period and T_y is the yield period, which are computed using the tangent stiffness of the hardening branch and using the tangent stiffness after the appearance of the first plastic hinge of the idealized lateral response of the structure, respectively. The required idealized lateral response of the structure for the evaluation of T_h and T_y has been performed by a nonlinear pushover analysis as suggested by Martinez-Rueda [10].

The effective peak acceleration (EPA) [11, 12] is the average of the spectral ordinates of the elastic acceleration response spectrum (for 5% critical damping) in the period interval [0.1 s, 0.5 s], divided by a standard value 2.5. It is defined by the following relation:

$$EPA = \frac{\overline{SA}_{0.1s \div 0.5s}(\xi = 0.05)}{2.5}. \quad (9)$$

The maximum EPA (EPA_{max}) is a variation of the EPA definition. Here, the period interval is not constant (as in the previous case from 0.1 to 0.5 s) but is a sliding window of 0.4 s, which sweeps the entire elastic acceleration response spectrum for 5% critical damping and provides the maximum of all possible EPA values calculated in this manner.

The input seismic energy [13] is defined by the following relation:

$$E_{inp} = m \int_0^{t_e} (\ddot{u} + \ddot{x}) \dot{x} dt. \quad (10)$$

Here, E_{inp} is the input seismic energy of a SDOF system, t_e is the total seismic duration, m is the mass, u is the displacement of the system relative to the ground, and x is the ground displacement.

The cumulative absolute velocity (CAV) [1] is defined as the area under the absolute accelerogram:

$$CAV = \int_0^{t_e} |\ddot{x}| dt. \quad (11)$$

Araya and Saragoni [14] introduced the destructiveness potential ($DP_{A/S}$) of a seismic excitation, defined as

$$DP_{A/S} = \frac{I_0}{v_0^2}, \quad (12)$$

where I_0 is the Arias intensity and v_0 is the intensity of the seismic acceleration zero-crossings. v_0 is defined as the number of intersections of the acceleration ordinates with the time axis (points with zero acceleration) per time unit in a seismic acceleration time-history (accelerogram).

The central period (CP) is the reciprocal value of the number of positive zero-crossings (points where the sign of the considered function changes from negative to positive) of the seismic acceleration per time unit [15]. The

CP is a frequency content indicator and not an authentic seismic intensity parameter. However, this parameter has been used to characterize the seismic damage potential of seismic excitations (together with the term PGA/PGV and the strong motion duration of Trifunac/Brady $T_{0.90}$) [16]. For this reason, the CP has been included in the set of the seismic intensity parameters used in the present study.

The response spectra describe the maximum response of a SDOF system to a particular input motion as a function of the natural period (or natural frequency) and the damping ratio of the SDOF system [17]. For each ground motion measurement there are displacement (SD), velocity (SV), and acceleration (SA) response spectra, respectively.

Finally, the seismic intensity of Fajfar/Vidic/Fischinger [18] is defined as

$$I_{F/V/F} = v_{max}(T_{0.90})^{0.25}, \quad (13)$$

where v_{max} is the peak ground velocity (PGV) of the ground motion and $T_{0.90}$ is the strong motion duration of Trifunac/Brady.

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 wide 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. Table 2 provides data for all the utilized seismic events (event name, country, date, record station, component, and PGA). Figure 1 is a scatter plot of standard (Richter) magnitude versus epicentral distance of all utilised seismic excitations. Furthermore, Tables 3 and 4 provide the number of accelerograms used per country and per 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 in Table 1.

The number of the used accelerograms is essential for the accuracy of the statistical models. In addition, the utilized accelerograms must generate a whole spectrum of damage (low, medium, large, and total) for statistical completeness. However, regional (locally possible) seismic records do not exist in sufficient quantity for most of the seismically active areas. Therefore, a set of worldwide natural acceleration records has been utilized for the present study. Alternatively, artificial accelerograms compatible with the design spectrum, combined with a scaling procedure (to generate a whole spectrum of damage), could be employed. In that case, the variability of the frequency content is limited.

4. Damage Indices

Among the many structural response parameters, attention is focused on those that can best describe seismic damage. The attention is focused on overall structural damage indices

TABLE 2: Data for the utilized seismic events.

Number	Seismic event	Country	Date	Station	Component	PGA (g)	Magnitude	Epicentral distance (km)
1	Long Beach	USA	10/3/1933	No. 288	S08W	0.131	6.4	38.1
2	Long Beach	USA	10/3/1933	No. 288	N82W	0.152	6.4	38.1
3	Long Beach	USA	10/3/1933	No. 131	NS	0.193	6.4	19.2
4	Long Beach	USA	10/3/1933	No. 331	EW	0.156	6.4	19.2
5	Long Beach	USA	10/3/1933	No. 136	N39E	0.100	6.4	45.2
6	Long Beach	USA	10/3/1933	No. 136	N51W	0.064	6.4	45.2
7	Lower California	USA	30/12/1934	No. 117	S00W	0.157	6.5	65.6
8	Lower California	USA	30/12/1934	No. 117	S90W	0.179	6.5	65.6
9	Helena Montana	USA	3/10/1935	No. 323	S00W	0.144	6.2	6.3
10	Helena Montana	USA	3/10/1935	No. 323	S90W	0.143	6.2	6.3
11	Imperial Valley	USA	18/5/1940	No. 117a	S00E	0.341	6.9	16.9
12	Imperial Valley	USA	18/5/1940	No. 117b	S00E	0.342	6.9	16.9
13	Imperial Valley	USA	18/5/1940	No. 117a	S90W	0.210	6.9	16.9
14	Imperial Valley	USA	18/5/1940	No. 117b	S90W	0.210	6.9	16.9
15	Santa Barbara	USA	30/6/1941	No. 283	N45E	0.234	5.5	14.9
16	Santa Barbara	USA	30/6/1941	No. 283	S45E	0.172	5.5	14.9
17	Northern California	USA	9/3/1949	No. 28	N89W	0.194	5.2	19.9
18	Puget Sound	USA	13/4/1949	Olympia	N176E	0.161	7.1	26.4
19	Puget Sound	USA	13/4/1949	Olympia	N266E	0.274	7.1	26.4
20	Western Washington	USA	13/4/1949	No. 325	N04W	0.162	7.1	17.0
21	Western Washington	USA	13/4/1949	No. 325	N86E	0.275	7.1	17.0
22	Kern County	USA	21/7/1952	No.283	S48E	0.129	7.5	87.7
23	Kern County	USA	21/8/1952	No. 1095	N21E	0.152	7.5	46.4
24	Kern County	USA	21/8/1952	No. 1095	S69E	0.176	7.5	46.4
25	Eureka	USA	21/12/1954	89509	349	0.165	6.6	23.5
26	Eureka	USA	21/12/1954	89509	79	0.253	6.6	23.5
27	Eureka	USA	21/12/1954	No. 1022	79	0.149	6.6	35.0
28	Eureka	USA	21/12/1954	No. 1022	N46W	0.189	6.6	35.0
29	Eureka	USA	21/12/1954	No. 1023	N44E	0.156	6.6	39.7
30	Eureka	USA	21/12/1954	No. 1023	N46W	0.197	6.6	39.7
31	Southern California	USA	18/3/1957	No. 272	NS	0.163	4.7	6.3
32	Southern California	USA	18/3/1957	No. 272	EW	0.086	4.7	6.3
33	Hollister	USA	8/4/1961	No. 028	N89W	0.176	5.6	22.2
34	Hollister	USA	8/4/1961	No. 028	N89W	0.169	5.6	22.2
35	Hokkaido	Japan	23/4/1962	Kushiro Met.	EW	0.281	7.0	75.0
36	Hokkaido	Japan	23/4/1962	Kushiro Met.	NS	0.517	7.0	75.0
37	Niigata	Japan	16/6/1964	Niigata	NS	0.129	7.5	59.0
38	Niigata	Japan	16/6/1964	Niigata	EW	0.171	7.5	59.0
39	Ibaraki	Japan	14/11/1964	Atom. En. Res	NS	0.260	5.1	8.0
40	Suruga Bay	Japan	20/4/1965	Shimizu	EW	0.153	6.1	30.0
41	Puget Sound	USA	29/4/1965	Olympia	N176E	0.134	6.5	63.0
42	Puget Sound	USA	29/4/1965	Olympia	N266E	0.195	6.5	63.0
43	Puget Sound	USA	29/4/1965	No. 325	S04E	0.134	6.5	61.0
44	Puget Sound	USA	29/4/1965	No. 325	S86E	0.194	6.5	61.0
45	Etorofu	Japan	5/4/1966	Hoshina-A	EW	0.500	5.4	4.0
46	Parkfield California	USA	27/6/1966	No. 013	N65E	0.480	5.6	3.5
47	Parkfield California	USA	27/6/1966	No. 014	N05W	0.348	5.6	3.5
48	Parkfield California	USA	27/6/1966	No. 014	N85E	0.426	5.6	7.1

TABLE 2: Continued.

Number	Seismic event	Country	Date	Station	Component	PGA (g)	Magnitude	Epicentral distance (km)
49	Parkfield California	USA	27/6/1966	No. 015	N50E	0.233	5.6	7.1
50	Parkfield California	USA	27/6/1966	No. 015	N40W	0.270	5.6	11.2
51	Parkfield California	USA	27/6/1966	No. 097	N65W	0.264	5.6	11.2
52	Parkfield California	USA	27/6/1966	No. 097	S25W	0.341	5.6	12.0
53	Parkfield California	USA	27/6/1966	No. 014	N05W	0.348	5.6	12.0
54	Parkfield California	USA	27/6/1966	No. 014	N85E	0.426	5.6	14.0
55	Ariake	Japan	12/11/1966	Ariake	N40E	0.129	5.5	19.0
56	Ibaraki	Japan	19/11/1967	Atom. En. Res	NS	0.464	6.0	50.0
57	Ibaraki	Japan	19/11/1967	Atom. En. Res	EW	0.350	6.0	50.0
58	Wakayama	Japan	30/3/1968	Wakayama Harbor	NS	0.293	5.0	5.0
59	Wakayama	Japan	30/3/1968	Wakayama Harbor	EW	0.442	5.0	5.0
60	Hyuganada	Japan	1/4/1968	New Ind. City	NS	0.288	7.5	60.0
61	Hyuganada	Japan	1/4/1968	New Ind. City	EW	0.351	7.5	60.0
62	Honshu	Japan	16/5/1968	Muroran Harbor	EW	0.155	8.2	290.0
63	Honshu	Japan	16/5/1968	Shin-Ishikari B.	L	0.182	8.2	319.0
64	Honshu	Japan	16/5/1968	Shin-Ishikari B.	T	0.194	8.2	319.0
65	Honshu	Japan	16/5/1968	Horoman Bridge	L	0.126	8.2	159.0
66	Honshu	Japan	16/5/1968	Miyako Harbor	NS	0.165	8.2	187.0
67	Honshu	Japan	16/5/1968	Miyako Harbor	EW	0.158	8.2	187.0
68	Honshu	Japan	16/5/1968	Aomori Harbor	NS	0.227	8.2	243.0
69	Honshu	Japan	16/5/1968	Aomori Harbor	EW	0.197	8.2	243.0
70	Honshu	Japan	16/5/1968	Hachinohe Harbor	NS	0.312	8.2	187.0
71	Honshu	Japan	16/5/1968	Hachinohe Harbor	EW	0.206	8.2	187.0
72	Honshu	Japan	16/5/1968	Miyako Harbor	NS	0.155	8.2	226.0
73	Honshu	Japan	16/5/1968	Miyako Harbor	EW	0.124	8.2	226.0
74	Aomori Prefecture	Japan	23/5/1968	Miyako Harbor	NS	0.133	6.3	78.0
75	Aomori Prefecture	Japan	12/6/1968	Miyako Harbor	NS	0.190	7.2	81.0
76	Aomori Prefecture	Japan	12/6/1968	Miyako Harbor	EW	0.164	7.2	81.0
77	Saitama Prefecture	Japan	1/7/1968	Shinagawa	EW	0.127	6.1	58.0
78	Hokkaido	Japan	21/1/1970	Horoman Bridge	L	0.149	6.7	46.0
79	Hokkaido	Japan	21/1/1970	Horoman Bridge	T	0.215	6.7	46.0
80	Iwate Prefecture	Japan	1/4/1970	Miyako Harbor	NS	0.190	5.8	17.0
81	Iwate Prefecture	Japan	1/4/1970	Miyako Harbor	EW	0.162	5.8	17.0
82	Miyazaki Prefecture	Japan	26/7/1970	New Ind. City	EW	0.140	6.7	21.0
83	San Fernando	USA	9/2/1971	No. 279	S16E	1.148	6.6	6.7
84	San Fernando	USA	9/2/1971	No. 279	S74W	1.055	6.6	6.7
85	San Fernando	USA	9/2/1971	No. 151	N54W	0.123	6.6	41.5
86	San Fernando	USA	9/2/1971	No. 110	N21E	0.309	6.6	29.8
87	San Fernando	USA	9/2/1971	No. 110	N69W	0.265	6.6	29.8
88	San Fernando	USA	9/2/1971	No. 111	S00W	0.153	6.6	29.8
89	San Fernando	USA	9/2/1971	No. 135	S00W	0.167	6.6	34.8
90	San Fernando	USA	9/2/1971	No. 135	N90E	0.207	6.6	34.8
91	San Fernando	USA	9/2/1971	No. 217	S00W	0.147	6.6	38.6
92	San Fernando	USA	9/2/1971	No. 137	N50W	0.126	6.6	41.2
93	San Fernando	USA	9/2/1971	No. 137	S40W	0.169	6.6	41.2
94	San Fernando	USA	9/2/1971	No. 285	S51E	0.203	6.6	32.9
95	San Fernando	USA	9/2/1971	No. 285	S75W	0.174	6.6	32.9
96	San Fernando	USA	9/2/1971	No. 199	S00E	0.158	6.6	38.8

TABLE 2: Continued.

Number	Seismic event	Country	Date	Station	Component	PGA (g)	Magnitude	Epicentral distance (km)
97	San Fernando	USA	9/2/1971	No. 199	N90E	0.162	6.6	38.8
98	San Fernando	USA	9/2/1971	No. 122	S70E	0.266	6.6	31.0
99	San Fernando	USA	9/2/1971	No. 122	S20W	0.209	6.6	31.0
100	San Fernando	USA	9/2/1971	No. 166	S53E	0.236	6.6	41.2
101	San Fernando	USA	9/2/1971	No. 166	S37W	0.192	6.6	41.2
102	San Fernando	USA	9/2/1971	No. 416	EW	0.162	6.6	36.6
103	San Fernando	USA	9/2/1971	No. 148	N37E	0.196	6.6	39.9
104	San Fernando	USA	9/2/1971	No. 148	S53E	0.188	6.6	39.9
105	San Fernando	USA	9/2/1971	No. 160	N30W	0.242	6.6	40.9
106	San Fernando	USA	9/2/1971	No. 160	S60W	0.221	6.6	40.9
107	San Fernando	USA	9/2/1971	No. 211	NS	0.154	6.6	38.8
108	San Fernando	USA	9/2/1971	No. 211	EW	0.130	6.6	38.8
109	San Fernando	USA	9/2/1971	No. 266	S90W	0.189	6.6	34.9
110	San Fernando	USA	9/2/1971	No. 264	NOOE	0.198	6.6	38.1
111	San Fernando	USA	9/2/1971	No. 264	N90E	0.182	6.6	38.1
112	San Fernando	USA	9/2/1971	No. 267	S82E	0.208	6.6	30.7
113	San Fernando	USA	9/2/1971	No. 267	S08W	0.139	6.6	30.7
114	San Fernando	USA	9/2/1971	No. 466	N11E	0.220	6.6	28.3
115	San Fernando	USA	9/2/1971	No. 466	N79W	0.146	6.6	28.3
116	San Fernando	USA	9/2/1971	No. 461	S81E	0.140	6.6	27.6
117	San Fernando	USA	9/2/1971	No. 461	S09W	0.129	6.6	27.6
118	San Fernando	USA	9/2/1971	No. 126	S69E	0.168	6.6	27.6
119	San Fernando	USA	9/2/1971	No. 126	S21W	0.144	6.6	27.6
120	San Fernando	USA	9/2/1971	No. 128	N21E	0.346	6.6	23.9
121	San Fernando	USA	9/2/1971	No. 128	N69W	0.277	6.6	23.9
122	San Fernando	USA	9/2/1971	No. 220	N00E	0.164	6.6	29.1
123	San Fernando	USA	9/2/1971	No. 220	S90W	0.148	6.6	29.1
124	San Fernando	USA	9/2/1971	No. 226	S89W	0.154	6.6	34.8
125	San Fernando	USA	9/2/1971	No. 226	S01E	0.156	6.6	34.8
126	San Fernando	USA	9/2/1971	No. 279	S74W	1.055	6.6	6.7
127	San Fernando	USA	9/2/1971	No. 128	N69W	0.278	6.6	23.9
128	Adak	USA	2/5/1971	Adak	N090E	0.204	7.1	78.9
129	Adak	USA	2/5/1971	Adak	N090E	0.205	7.1	78.9
130	Santiago	Chile	9/7/1971	Santiago Univ.	N10W	0.156	7.5	117.0
131	Chiba	Japan	11/10/1971	Kashima Harbor	EW	0.169	5.2	12.0
132	Kushiro	Japan	11/10/1971	Kushiro	NS	0.143	5.8	33.0
133	Central California	USA	4/9/1972	USGS 1211	N61E	0.471	4.8	9.8
134	Central California	USA	4/9/1972	USGS 1211	N29W	0.516	4.8	9.8
135	Central North Island	New Zealand	5/1/1973	ATENE C	N40E	0.143	6.7	67.0
136	Central North Island	New Zealand	5/1/1973	ATENE C	N50W	0.153	6.7	67.0
137	Nemuro	Japan	17/6/1973	Kushiro	NS	0.201	7.4	112.0
138	Nemuro	Japan	17/6/1973	Kushiro	EW	0.127	7.4	112.0
139	Lefkada	Greece	4/11/1973	Lefkada	L	0.041	5.0	19.0
140	Lefkada	Greece	4/11/1973	Lefkada	T	0.074	5.0	19.0
141	Lefkada	Greece	4/11/1973	Lefkada	L	0.515	5.8	20.0
142	Lefkada	Greece	4/11/1973	Lefkada	T	0.258	5.8	20.0
143	Hollister	USA	28/11/1974	Gilroy Array	S67W	0.135	5.2	12.0
144	Hollister	USA	28/11/1974	Hollister	N87W	0.163	5.2	9.8

TABLE 2: Continued.

Number	Seismic event	Country	Date	Station	Component	PGA (g)	Magnitude	Epicentral distance (km)
145	Cape Mendocino	USA	12/1/1975	Petrolia	N75E	0.180	5.2	19.0
146	Oita Prefecture	Japan	21/4/1975	Beppu Harbor	EW	0.138	6.4	41.0
147	Cape Mendocino	USA	6/5/1975	Shelter Cove	N70E	0.168	5.3	63.0
148	Humboldt County	USA	7/6/1975	Petrolia	S60E	0.199	5.2	31.0
149	Humboldt County	USA	7/6/1975	Petrolia	N75E	0.158	5.2	31.0
150	Whittier Narrows	USA	7/6/1975	Petrolia	N15W	0.128	5.2	31.0
151	Oroville	USA	6/8/1975	USGS 0001	N90E	0.292	4.7	7.6
152	Oroville	USA	6/8/1975	USGS 0001	N00E	0.620	4.7	7.6
153	Oroville	USA	11/8/1975	USGS 0001	S55E	0.389	4.4	1.0
154	Oroville	USA	11/8/1975	USGS 0001	N35E	0.284	4.4	1.0
155	Denizli	Turkey	19/8/1976	Denizli Met.	NS	0.344	4.4	1.0
156	Vrancea	Romania	4/3/1977	Bucharest	EW	0.175	7.4	170.0
157	Vrancea	Romania	4/3/1977	Bucharest	SN	0.202	7.4	170.0
158	Vrancea	Romania	4/3/1977	Bucharest	NS	0.194	7.4	165.0
159	Vrancea	Romania	4/3/1977	Bucharest	EW	0.163	7.4	165.0
160	Izmir	Turkey	16/12/1977	Izmir	NS	0.409	5.3	6.0
161	Izmir	Turkey	16/12/1977	Izmir	EW	0.136	5.3	6.0
162	Miyagi Prefecture	Japan	12/6/1978	Ofunato Harbor	N41E	0.207	7.4	116.0
163	Miyagi Prefecture	Japan	12/6/1978	Ofunato Harbor	E41S	0.222	7.4	116.0
164	Miyagi Prefecture	Japan	12/6/1978	Siogama Harbor	NS	0.314	7.4	120.0
165	Miyagi Prefecture	Japan	12/6/1978	Siogama Harbor	EW	0.288	7.4	120.0
166	Dursunbey	Turkey	18/7/1979	Dursunbey	NS	0.234	5.2	10.0
167	Dursunbey	Turkey	18/7/1979	Dursunbey	EW	0.309	5.2	10.0
168	Coyote Lake	USA	6/8/1979	USGS 1409	S40E	0.251	5.9	14.4
169	Coyote Lake	USA	6/8/1979	USGS 1409	N50E	0.188	5.9	14.4
170	Coyote Lake	USA	6/8/1979	USGS 1413	N40W	0.299	5.9	10.3
171	Coyote Lake	USA	6/8/1979	USGS 1413	S50W	0.408	5.9	10.3
172	Imperial Valley	USA	15/10/1979	El Centro	230	0.367	6.6	27.6
173	Imperial Valley	USA	15/10/1979	El Centro	140	0.517	6.6	27.6
174	Imperial Valley	USA	15/10/1979	El Centro	230	0.349	6.6	18.5
175	Imperial Valley	USA	15/10/1979	El Centro	140	0.483	6.6	18.5
176	Imperial Valley	USA	15/10/1979	El Centro	230	0.405	6.6	33.9
177	Imperial Valley	USA	15/10/1979	El Centro	140	0.309	6.6	33.9
178	Imperial Valley	USA	15/10/1979	El Centro	140	0.138	6.6	27.8
179	Imperial Valley	USA	15/10/1979	El Centro	140	0.151	6.6	27.8
180	Imperial Valley	USA	15/10/1979	El Centro	230	0.162	6.6	27.6
181	Imperial Valley	USA	15/10/1979	El Centro	140	0.229	6.6	27.6
182	Imperial Valley	USA	15/10/1979	El Centro	230	0.275	6.6	27.3
183	Imperial Valley	USA	15/10/1979	El Centro	140	0.221	6.6	27.3
184	Imperial Valley	USA	15/10/1979	El Centro	230	0.147	6.6	36.3
185	Imperial Valley	USA	15/10/1979	El Centro	140	0.130	6.6	36.3
186	Imperial Valley	USA	15/10/1979	El Centro	360	0.137	6.6	33.9
187	Imperial Valley	USA	15/10/1979	El Centro	270	0.145	6.6	33.9
188	Imperial Valley	USA	15/10/1979	USGS 5058	S50W	0.362	6.6	27.9
189	Imperial Valley	USA	15/10/1979	USGS 5058	S40E	0.352	6.6	27.9
190	Imperial Valley	USA	15/10/1979	USGS 5115	S50W	0.365	6.6	31.5
191	Imperial Valley	USA	15/10/1979	USGS 5115	S40E	0.309	6.6	31.5
192	Imperial Valley	USA	15/10/1979	USGA 5028	S50W	0.450	6.6	27.1

TABLE 2: Continued.

Number	Seismic event	Country	Date	Station	Component	PGA (g)	Magnitude	Epicentral distance (km)
193	Imperial Valley	USA	15/10/1979	USGA 5028	S40E	0.331	6.6	27.1
194	Imperial Valley	USA	15/10/1979	USGA 0942	S50W	0.429	6.6	27.1
195	Imperial Valley	USA	15/10/1979	USGA 0942	S40E	0.332	6.6	27.1
196	Imperial Valley	USA	15/10/1979	USGS 5054	S50W	0.762	6.6	6.7
197	Imperial Valley	USA	15/10/1979	USGS 5054	S40E	0.584	6.6	6.7
198	Imperial Valley	USA	15/10/1979	USGS 0958	S50W	0.438	6.6	27.2
199	Imperial Valley	USA	15/10/1979	USGS 0958	S40E	0.599	6.6	27.2
200	Imperial Valley	USA	15/10/1979	USGS 0952	S50W	0.360	6.6	27.6
201	Imperial Valley	USA	15/10/1979	USGS 0952	S40E	0.539	6.6	27.6
202	Imperial Valley	USA	15/10/1979	USGS 5165	NS	0.473	6.6	26.3
203	Imperial Valley	USA	15/10/1979	USGS 5165	EW	0.339	6.6	26.3
204	Imperial Valley	USA	15/10/1979	USGS 0955	S50W	0.349	6.6	27.2
205	Imperial Valley	USA	15/10/1979	USGS 0955	S40E	0.480	6.6	27.2
206	Livermore	USA	24/1/1980	CDMG 0364	EW	0.031	5.9	24.4
207	Livermore	USA	24/1/1980	CDMG 0364	NS	0.027	5.9	24.4
208	Mammoth Lakes	USA	27/5/1980	SER no. 3679	S34E	0.337	6.2	17.3
209	Mammoth Lakes	USA	27/5/1980	SER no. 3679	N56E	0.138	6.2	17.3
210	Mammoth Lakes	USA	27/5/1980	SER no. 1480	NS	0.389	6.2	19.8
211	Mammoth Lakes	USA	27/5/1980	SER no. 1480	EW	0.344	6.2	19.8
212	Mammoth Lakes	USA	27/5/1980	SER no. 3754	S15E	0.631	6.2	14.8
213	Mammoth Lakes	USA	27/5/1980	SER no. 3754	N75E	0.452	6.2	14.8
214	Korinthos	Greece	24/2/1981	Athens	L	0.221	6.7	69.0
215	Korinthos	Greece	24/2/1981	Korinthos	L	0.233	6.7	31.0
216	Korinthos	Greece	24/2/1981	Korinthos	T	0.299	6.7	31.0
217	Korinthos	Greece	24/2/1981	Xylokastro	L	0.289	6.7	35.0
218	Korinthos	Greece	24/2/1981	Xylokastro	T	0.114	6.7	35.0
219	Korinthos	Greece	25/2/1981	Athens	T	0.095	6.7	69.0
220	Korinthos	Greece	25/2/1981	Korinthos	L	0.117	6.7	33.0
221	Korinthos	Greece	25/2/1981	Korinthos	T	0.121	6.7	33.0
222	Lefkada	Greece	10/3/1981	Lefkada	L	0.057	5.8	53.0
223	Lefkada	Greece	10/3/1981	Lefkada	T	0.097	5.8	53.0
224	Lefkada	Greece	10/3/1981	Preveza	L	0.141	5.8	39.0
225	Lefkada	Greece	10/3/1981	Preveza	T	0.137	5.8	39.0
226	Lefkada	Greece	10/4/1981	Lefkada	L	0.023	4.7	34.0
227	Lefkada	Greece	10/4/1981	Lefkada	T	0.036	4.7	34.0
228	Westmoreland	USA	26/4/1981	WEST 2588	EW	0.354	5.6	11.2
229	Westmoreland	USA	26/4/1981	WEST 2588	NS	0.435	5.6	11.2
230	Lefkada	Greece	25/5/1981	Lefkada	L	0.057	4.7	25.0
231	Lefkada	Greece	25/5/1981	Lefkada	T	0.032	4.7	25.0
232	Lefkada	Greece	27/5/1981	Lefkada	L	0.117	5.5	3.0
233	Lefkada	Greece	27/5/1981	Lefkada	T	0.093	5.5	3.0
234	Ionian	Greece	17/1/1983	Argostoli	NS	0.162	7.2	28.7
235	Ionian	Greece	17/1/1983	Argostoli	WE	0.139	7.2	28.7
236	Heraklio	Greece	19/3/1983	Heraklio	T	0.170	5.7	38.0
237	Ionian	Greece	23/3/1983	Argostoli	NS	0.177	5.9	29.0
238	Ionian	Greece	23/3/1983	Argostoli	WE	0.234	5.9	29.0
239	Ionian	Greece	24/3/1983	Argostoli	NS	0.242	5.5	14.7

TABLE 2: Continued.

Number	Seismic event	Country	Date	Station	Component	PGA (g)	Magnitude	Epicentral distance (km)
240	Ionian	Greece	24/3/1983	Argostoli	WE	0.299	5.5	14.7
241	Coalinga	USA	2/5/1983	Coalinga	360	0.595	6.5	28.1
242	Coalinga	USA	2/5/1983	Coalinga	270	0.549	6.5	28.1
243	Coalinga	USA	2/5/1983	Coalinga	360	0.264	6.5	40.6
244	Coalinga	USA	2/5/1983	Coalinga	270	0.371	6.5	40.6
245	Coalinga	USA	2/5/1983	Coalinga	360	0.189	6.5	36.4
246	Coalinga	USA	2/5/1983	Coalinga	270	0.212	6.5	36.4
247	Horasan	Turkey	30/10/1983	Horasan	NS	0.149	6.9	30.0
248	Heraklio	Greece	1/3/1984	Heraklio	L	0.106	4.7	30.0
249	Heraklio	Greece	1/3/1984	Heraklio	T	0.197	4.7	30.0
250	Morgan Hill	USA	24/4/1984	Anderson Dam	340	0.283	6.2	15.0
251	Morgan Hill	USA	24/4/1984	Anderson Dam	250	0.416	6.2	15.0
252	Morgan Hill	USA	24/4/1984	Gilroy Array	0	0.183	6.2	38.0
253	Morgan Hill	USA	24/4/1984	Gilroy Array	0	0.215	6.2	38.1
254	Morgan Hill	USA	24/4/1984	Gilroy Array	90	0.280	6.2	38.1
255	Morgan Hill	USA	24/4/1984	Gilroy Array	270	0.217	6.2	37.9
256	Morgan Hill	USA	24/4/1984	Gilroy Array	360	0.328	6.2	37.9
257	Morgan Hill	USA	24/4/1984	No. 47380	90	0.210	6.2	38.8
258	Morgan Hill	USA	24/4/1984	No. 47380	0	0.154	6.2	38.8
259	Valparaiso	Chile	3/3/1985	Llolleo	100	0.187	7.8	120.0
260	Valparaiso	Chile	3/3/1985	Llolleo	10	0.182	7.8	120.0
261	Valparaiso	Chile	3/3/1985	Llolleo	100	0.437	7.8	120.0
262	Valparaiso	Chile	3/3/1985	Llolleo	10	0.698	7.8	120.0
263	Valparaiso	Chile	3/3/1985	Vina del Mar	290	0.233	7.8	43.0
264	Valparaiso	Chile	3/3/1985	Vina del Mar	200	0.356	7.8	43.0
265	Valparaiso	Chile	3/3/1985	Valparaiso	50	0.292	7.8	36.0
266	Valparaiso	Chile	3/3/1985	Valparaiso	140	0.163	7.8	36.0
267	Valparaiso	Chile	3/3/1985	Santiago	L	0.124	7.8	130.0
268	Valparaiso	Chile	8/4/1985	Llolleo	100	0.167	7.5	120.0
269	Valparaiso	Chile	8/4/1985	Llolleo	10	0.200	7.5	120.0
270	Michoacan	Mexico	19/9/1985	SCT1	N90W	0.168	8.1	400.0
271	Michoacan	Mexico	19/9/1985	La Union	N00E	0.163	8.1	107.0
272	Michoacan	Mexico	19/9/1985	Michoacan	N90W	0.162	8.1	111.0
273	Michoacan	Mexico	19/9/1985	La Union	N90E	0.147	8.1	107.0
274	Michoacan	Mexico	19/9/1985	Caleta de Campo	N00E	0.139	8.1	21.0
275	Michoacan	Mexico	19/9/1985	Caleta de Campo	N90E	0.138	8.1	21.0
276	Michoacan	Mexico	19/9/1985	La Villita	N00E	0.121	8.1	65.0
277	Michoacan	Mexico	21/9/1985	Michoacan	S00E	0.156	7.5	91.0
278	Michoacan	Mexico	21/9/1985	Michoacan	N90W	0.140	7.5	91.0
279	Michoacan	Mexico	21/9/1985	Papanao	S00E	0.252	7.5	130.0
280	Nahanni	Canada	9/11/1985	Slide Mountain	330	0.374	5.0	5.8
281	Nahanni	Canada	9/11/1985	Slide Mountain	240	0.451	5.0	5.8
282	Nahanni	Canada	23/12/1985	Iverson	10	1.080	6.9	8.0
283	Nahanni	Canada	23/12/1985	Iverson	280	1.319	6.9	8.0
284	Nahanni	Canada	23/12/1985	Iverson	10	0.224	6.9	8.0
285	Nahanni	Canada	23/12/1985	Slide Mountain	330	0.386	6.9	15.0
286	Nahanni	Canada	23/12/1985	Slide Mountain	240	0.534	6.9	15.0
287	Nahanni	Canada	23/12/1985	Nahanni	360	0.190	6.9	23.8

TABLE 2: Continued.

Number	Seismic event	Country	Date	Station	Component	PGA (g)	Magnitude	Epicentral distance (km)
288	Nahanni	Canada	23/12/1985	Nahanni	270	0.192	6.9	23.8
289	Kalamata	Greece	13/9/1986	Kalamata	L	0.217	6.0	10.0
290	Kalamata	Greece	13/9/1986	Kalamata	T	0.291	6.0	10.0
291	Kalamata	Greece	15/9/1986	Kalamata	L	0.328	4.1	7.0
292	Kalamata	Greece	15/9/1986	Kalamata	T	0.161	4.1	7.0
293	San Salvador	San Salvador	10/10/1986	No. 90005	270	0.524	5.4	5.7
294	San Salvador	San Salvador	10/10/1986	No. 90005	180	0.391	5.4	5.7
295	San Salvador	San Salvador	10/10/1986	No. 90018	0	0.214	5.4	6.2
296	San Salvador	San Salvador	10/10/1986	No. 90018	270	0.296	5.4	6.2
297	San Salvador	San Salvador	10/10/1986	No. 90006	180	0.412	5.4	4.3
298	San Salvador	San Salvador	10/10/1986	No. 90006	90	0.681	5.4	4.3
299	San Salvador	San Salvador	10/10/1986	No. 90014	90	0.339	5.4	4.5
300	San Salvador	San Salvador	10/10/1986	No. 90014	0	0.421	5.4	4.5
301	Strazhitsa	Bulgaria	7/12/1986	V. Tarnovo	L	0.067	5.7	225.0
302	Strazhitsa	Bulgaria	7/12/1986	V. Tarnovo	T	0.074	5.7	225.0
303	Whittier Narrows	USA	1/10/1987	No. 24157	90	0.149	6.1	27.0
304	Whittier Narrows	USA	1/10/1987	No. 24157	0	0.139	6.1	27.0
305	Whittier Narrows	USA	1/10/1987	Tarzana	90	0.527	6.1	43.3
306	Whittier Narrows	USA	1/10/1987	Tarzana	360	0.397	6.1	43.3
307	Whittier Narrows	USA	1/10/1987	Hollywood	0	0.201	6.1	25.2
308	Whittier Narrows	USA	1/10/1987	Inglewood	90	0.219	6.1	25.8
309	Whittier Narrows	USA	1/10/1987	Inglewood	0	0.246	6.1	25.8
310	Whittier Narrows	USA	1/10/1987	Long Beach	90	0.233	6.1	27.4
311	Whittier Narrows	USA	1/10/1987	Long Beach	0	0.144	6.1	27.4
312	Whittier Narrows	USA	1/10/1987	Alhambra	90	0.284	6.1	6.8
313	Whittier Narrows	USA	1/10/1987	Alhambra	360	0.250	6.1	6.8
314	Whittier Narrows	USA	1/10/1987	Whittier	180	0.382	6.1	15.6
315	Whittier Narrows	USA	1/10/1987	Whittier	90	0.607	6.1	15.6
316	Whittier Narrows	USA	1/10/1987	Vernon	7	0.267	6.1	15.7
317	Whittier Narrows	USA	1/10/1987	Vernon	277	0.240	6.1	15.7
318	Whittier Narrows	USA	1/10/1987	Norwalk	360	0.201	6.1	21.9
319	Whittier Narrows	USA	1/10/1987	Norwalk	360	0.235	6.1	21.9
320	Rhodos	Greece	5/10/1987	Rhodos	L	0.056	5.6	27.0
321	Rhodos	Greece	5/10/1987	Rhodos	T	0.052	5.6	27.0
322	Superstition Hills	USA	24/11/1987	Wildlife Liq.	360	0.131	6.6	30.0
323	Superstition Hills	USA	24/11/1987	Wildlife Liq.	90	0.129	6.6	30.0
324	Lefkada	Greece	24/4/1988	Lefkada	L	0.114	5.0	33.0
325	Lefkada	Greece	24/4/1988	Lefkada	T	0.208	5.0	33.0
326	Valsamata	Greece	18/5/1988	Valsamata	L	0.165	5.8	23.0
327	Valsamata	Greece	18/5/1988	Valsamata	T	0.174	5.8	23.0
328	Valsamata	Greece	22/5/1988	Valsamata	L	0.053	5.5	21.0
329	Valsamata	Greece	22/5/1988	Valsamata	T	0.079	5.5	21.0
330	Kyllini	Greece	16/10/1988	Zakynthos	NS	0.125	6.0	36.0
331	Kyllini	Greece	16/10/1988	Zakynthos	WE	0.167	6.0	36.0
332	Vartholomio	Greece	19/10/1988	Vartholomio	L	0.047	4.4	19.0
333	Vartholomio	Greece	19/10/1988	Vartholomio	T	0.035	4.4	19.0
334	Vartholomio	Greece	31/10/1988	Vartholomio	L	0.062	4.8	17.0
335	Vartholomio	Greece	31/10/1988	Vartholomio	T	0.035	4.8	17.0

TABLE 2: Continued.

Number	Seismic event	Country	Date	Station	Component	PGA (g)	Magnitude	Epicentral distance (km)
336	Nafpaktos	Greece	22/12/1988	Nafpaktos	L	0.112	5.0	10.0
337	Nafpaktos	Greece	22/12/1988	Nafpaktos	T	0.103	5.0	10.0
338	Loma Prieta	USA	17/10/1989	Anderson Dam	340	0.239	7.0	96.7
339	Loma Prieta	USA	17/10/1989	Anderson Dam	250	0.239	7.0	96.7
340	Loma Prieta	USA	17/10/1989	Hollister	255	0.281	7.0	48.1
341	Loma Prieta	USA	17/10/1989	Hollister	165	0.276	7.0	48.1
342	Loma Prieta	USA	17/10/1989	Palo Alto Hospital	302	0.341	7.0	46.3
343	Loma Prieta	USA	17/10/1989	Palo Alto Hospital	212	0.378	7.0	46.3
344	Loma Prieta	USA	17/10/1989	Hollister	180	0.217	7.0	45.0
345	Loma Prieta	USA	17/10/1989	Hollister	90	0.252	7.0	45.0
346	Loma Prieta	USA	17/10/1989	Stanford	360	0.255	7.0	51.5
347	Loma Prieta	USA	17/10/1989	Stanford	90	0.215	7.0	51.5
348	Loma Prieta	USA	17/10/1989	Calaveras	180	0.142	7.0	46.5
349	Loma Prieta	USA	17/10/1989	Calaveras	90	0.191	7.0	46.5
350	Loma Prieta	USA	17/10/1989	Apeel Array	133	0.223	7.0	64.1
351	Loma Prieta	USA	17/10/1989	Apeel Array	43	0.272	7.0	64.1
352	Loma Prieta	USA	17/10/1989	Emeryville	350	0.210	7.0	96.7
353	Loma Prieta	USA	17/10/1989	Emeryville	260	0.255	7.0	96.7
354	Loma Prieta	USA	17/10/1989	Anderson Dam	333	0.239	7.0	96.7
355	Loma Prieta	USA	17/10/1989	Anderson Dam	243	0.245	7.0	96.7
356	Loma Prieta	USA	17/10/1989	Emeryville	350	0.210	7.0	96.7
357	Loma Prieta	USA	17/10/1989	Emeryville	260	0.255	7.0	96.7
358	Loma Prieta	USA	17/10/1989	Emeryville	260	0.209	7.0	96.7
359	Loma Prieta	USA	17/10/1989	Emeryville	350	0.182	7.0	96.7
360	Loma Prieta	USA	17/10/1989	San Francisco	95	0.220	7.0	96.7
361	Aigio	Greece	17/5/1990	Aigio	T	0.203	5.0	20.0
362	Erzincan	Turkey	13/3/1992	Erzincan	NS	0.392	6.9	6.0
363	Erzincan	Turkey	13/3/1992	Erzincan	EW	0.492	6.9	6.0
364	Northridge	USA	17/1/1994	Los Angeles	166	0.257	6.4	13.7
365	Northridge	USA	17/1/1994	Los Angeles	76	0.462	6.4	13.7
366	Northridge	USA	17/1/1994	Los Angeles	348	0.133	6.4	31.0
367	Northridge	USA	17/1/1994	Los Angeles	258	0.147	6.4	31.0
368	Northridge	USA	17/1/1994	Canoga Park	90	0.334	6.4	6.0
369	Northridge	USA	17/1/1994	Los Angeles	360	0.260	6.4	42.2
370	Northridge	USA	17/1/1994	Los Angeles	270	0.154	6.4	42.2
371	Northridge	USA	17/1/1994	Malibu Canyon	360	0.173	6.4	14.9
372	Northridge	USA	17/1/1994	Malibu Canyon	270	0.161	6.4	14.9
373	Northridge	USA	17/1/1994	Littlerock	300	0.120	6.4	61.3
374	Northridge	USA	17/1/1994	Topanga	360	0.326	6.4	6.0
375	Northridge	USA	17/1/1994	Topanga	270	0.194	6.4	6.0
376	Northridge	USA	17/1/1994	Los Angeles	360	0.164	6.4	31.2
377	Northridge	USA	17/1/1994	Los Angeles	270	0.291	6.4	31.2
378	Northridge	USA	17/1/1994	Los Angeles	325	0.383	6.4	19.1
379	Northridge	USA	17/1/1994	Los Angeles	235	0.298	6.4	19.1
380	Northridge	USA	17/1/1994	Los Angeles	325	0.247	6.4	28.7
381	Northridge	USA	17/1/1994	Jensen	22	0.388	6.4	12.2
382	Northridge	USA	17/1/1994	Jensen	292	0.621	6.4	12.2
383	Northridge	USA	17/1/1994	USC no. 0003	S00E	0.444	6.4	2.2

TABLE 2: Continued.

Number	Seismic event	Country	Date	Station	Component	PGA (g)	Magnitude	Epicentral distance (km)
384	Northridge	USA	17/1/1994	USC no. 0006	N90E	0.430	6.4	11.1
385	Northridge	USA	17/1/1994	USC no. 0009	S90W	0.248	6.4	12.0
386	Northridge	USA	17/1/1994	USC no. 0013	N09E	0.434	6.4	12.7
387	Northridge	USA	17/1/1994	USC no. 0014	S55E	0.434	6.4	15.5
388	Northridge	USA	17/1/1994	USC no. 0016	N90E	0.257	6.4	16.5
389	Northridge	USA	17/1/1994	USC no. 0021	N00W	0.409	6.4	26.5
390	Northridge	USA	17/1/1994	USC no. 0053	S16W	0.381	6.4	6.0
391	Northridge	USA	17/1/1994	USC no. 0054	S65W	0.324	6.4	25.2
392	Northridge	USA	17/1/1994	USC no. 0055	N90E	0.503	6.4	13.0
393	Northridge	USA	17/1/1994	USC no. 0056	N46E	0.411	6.4	21.5
394	Northridge	USA	17/1/1994	USC no. 0057	N00W	0.389	6.4	25.6
395	Northridge	USA	17/1/1994	USC no. 0091	S70E	0.420	6.4	24.9
396	Northridge	USA	17/1/1994	USC no. 0095	N90W	0.186	6.4	42.9
397	Lefkada	Greece	25/2/1994	Lefkada	L	0.171	5.8	16.0
398	Lefkada	Greece	25/2/1994	Lefkada	T	0.197	5.8	16.0
399	Kobe	Japan	17/1/1995	Kobe	EW	0.619	6.9	1.0
400	Kobe	Japan	17/1/1995	Kobe	NS	0.821	6.9	1.0

FIGURE 1: Plot of standard seismic magnitudes versus epicentral distances.

(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 [19, 20] 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 and Ang must be calculated using the following equation:

$$DI_{P/A,Local} = \frac{\theta_m - \theta_r}{\theta_u - \theta_r} + \frac{\beta}{M_y \theta_u} E_T, \quad (14)$$

TABLE 3: Number of accelerograms employed per country.

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

TABLE 4: 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

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 [21]), 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 (14) is equal to 0.1. This value corresponds to nominal strength

TABLE 5: Damage classification limits.

	Damage classes			
	Low	Medium	Large	Total
OSDIs of Park and Ang ($DI_{P/A,Global}$) and of DiPasquale and Cakmak (δ_M)	≤ 0.3	$0.3 < OSDI \leq 0.6$	$0.6 < OSDI \leq 0.8$	> 0.8
MISDR (%)	≤ 0.5	$0.5 < MISDR \leq 1.5$	$1.5 < MISDR \leq 2.5$	> 2.5

degradation [21]. However, the numerical value of β can vary in the range between 0 (no strength degradation) and 0.4 (severe deterioration) [21]. A low β value corresponds to well-detailed reinforced concrete or steel members [22]. In contrast, a high β value corresponds to poorly detailed reinforced concrete members or to unreinforced masonry [20].

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

$$DI_{P/A,Global} = \frac{\sum_{i=1}^n DI_{P/A,Local} E_i}{\sum_{i=1}^n E_i}, \quad (15)$$

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

$$MISDR = \frac{|u|_{max}}{h} 100 [\%]. \quad (16)$$

The maximum softening index of DiPasquale and Cakmak [23] has been selected as a third OSDI, which is based on the vibration parameters of the structure. It is given by the following expression:

$$\delta_M = 1 - \frac{T_0}{T_{max}}, \quad (17)$$

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} the instantaneous natural period is required to be evaluated, which is accompanied by the actual time-dependent (instantaneous) tangent stiffness matrix. The latter varies with time due to nonlinear phenomena. The natural period computed for each time step of a nonlinear dynamic analysis presents high variability. The duration of these maximum values is very short, and therefore, their influence in the natural period of an equivalent linear system is not significant. Thus, a more meaningful indication of the change in the natural period can be obtained by observing a moving average of the instantaneous natural period using a sliding time window for the smoothness of the time-fundamental period curve [23, 24]. 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

FIGURE 2: Examined reinforced concrete frame structure.

period of the structure that corresponds to the initial stiffness of the structure.

Table 5 presents the range limits of the four damage classes using the different damage indices [25]. The four levels (low, medium, large, and total) correspond to undamaged or minor damage, repairable damage, irreparable damage, and partial or total collapse of the structure.

5. Application

The proposed procedures have been applied to a reinforced concrete structure, shown in Figure 2, and designed in agreement with the rules of the recent Eurocodes for structural concrete and aseismic structures, EC2 and EC8 [26, 27]. 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 2 is considered an “importance

class II, ductility class M" structure with category B subsoil. The ordinate of the used elastic response spectrum of EC8 was 0.24 g.

After designing and detailing the reinforced concrete frame structure, nonlinear dynamic analyses were carried out to evaluate the structural seismic response. For this purpose, the computer program IDARC [21] 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 [21]. 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 [20]. Thus, nominal parameters for stiffness degradation and strength deterioration have been chosen. In contrast, no pinching has been considered. Realistic hysteretic material models result in realistic proposed statistical models. On the other hand, simplified assumptions for the hysteretic material models, like ignoring the pinching effect (default option in the utilized computer program IDARC [21]), do not affect on principle the value of the proposed statistical models that are in any case procedures for the approximate assessment of structural damage. Of the several response parameters, the focus is on the OSDIs. Consequently, the OSDIs of Park and Ang and of DiPasquale and Cakmak (maximum softening index) and the MISDR have been calculated for all of the accelerograms corresponding to the excitations given in Table 2, which were used as the seismic input for the nonlinear dynamic analyses. For the evaluation of the global damage index of DiPasquale and Cakmak according to (17), 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/Cakmak [24]. 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 (17)). Finally, the hardening period T_h and yield period T_y needed for the assessment of the SI_{MR} in (8) are evaluated by a nonlinear pushover analysis, as described in Section 2. Their numerical values are $T_y = 1.2$ s and $T_h = 1.5$ s.

6. Multilinear Regression Analysis

In general, a multilinear regression analysis [28] is used to express a dependent variable (y) by (n)-independent variables (x_i , with $i = 1, \dots, n$) and a constant value (a) minimizing the error term (e), as shown by the relation

$$y = a + b_1x_1 + b_2x_2 + \dots + b_nx_n + e. \quad (18)$$

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

TABLE 6: Multilinear regression model for the OSDI of Park and 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

The analyses have been carried out using the statistical software STATGRAPHICS [29]. The regression results for the OSDIs after Park and Ang, for the MISDR, and for the maximum softening index after DiPasquale and Cakmak are given in Tables 6–9 and will be presented in detail subsequently. Thus, the result for the OSDI after Park and Ang is given in Table 6. The equation of the fitted multilinear model is

$$\begin{aligned}
 DI_{P/A, \text{Global}} = & -0.032 + 0.370PGA + 0.084PGV \\
 & + \frac{0.005PGA}{PGV} + 0.007Arias \\
 & - 0.074RMS_a - 0.0004T_{0.90} - 0.004P_{0.90} \\
 & + 0.036SI_H - 0.014SI_K - 0.379SI_{MR} \\
 & - 0.385EPA + 0.076EPA_{\max} + 0.037E_{\text{inp}} \\
 & - 0.020CAV - 0.202DP_{A/S} + 0.038CP \\
 & - 0.110SD + 0.167SV + 0.031SA \\
 & + 0.017I_{F/V/F}.
 \end{aligned} \quad (19a)$$

$R^2 = 0.859$, which indicates that the model as fitted explains 85.9% of the variability in the Park and Ang OSDI.

TABLE 7: Multilinear regression model for the OSDI of Park and Ang without the constant term.

Parameter	Estimate	Lower Limit	Upper Limit
PGA	0.324	0.195	0.454
PGV	0.072	-0.042	0.186
PGA/PGV	-0.001	-0.008	0.005
I_0	0.007	0.005	0.009
RMS_a	-0.095	-0.175	-0.016
$T_{0.90}$	-0.001	-0.002	0.000
$P_{0.90}$	-0.003	-0.014	0.007
SI_H	0.028	-0.012	0.068
SI_K	-0.010	-0.123	0.104
SI_{MR}	-0.337	-0.496	-0.178
EPA	-0.370	-0.637	-0.102
EPA_{max}	0.076	0.003	0.148
E_{inp}	0.043	0.019	0.067
CAV	-0.020	-0.052	0.012
$DP_{A/S}$	-0.202	-0.437	0.034
CP	0.008	-0.044	0.060
SD	-0.086	-0.403	0.232
SV	0.154	0.062	0.247
SA	0.045	-0.077	0.168
$I_{F/V/F}$	0.016	-0.018	0.051

In addition, Table 6 presents the 95% confidence intervals for the coefficients in the multilinear model. Each variable coefficient in (19a) and in Table 6, or generally in (18), 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 after Trifunac/Brady), whereas such a change cannot be easily observed in others (such as the SD). In addition, the comparison between coefficients of different explanatory variables is not possible because they are assigned quantities having different dimensions and units [30]. Finally, the constant term (“ a ” in (18) and “-0.032” in (19a)) 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. However, Table 7 presents the coefficients and their 95% confidence intervals for the multilinear regression model of the OSDI after Park and Ang, without the constant term ($R^2 = 0.900218$). As indicated by R^2 , both models (with and without the constant term) explain a high percentage of the variance (85.9% and 90%, resp.) in the dependent variable.

Similarly, Table 8 presents the multilinear model for the MISDR ($R^2 = 0.799279$) and Table 9 presents the model for the OSDI (maximum softening index) after DiPasquale and Cakmak ($R^2 = 0.763627$), along with their 95% confidence

TABLE 8: 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

intervals. The fitted multilinear models for these global structural damage indices are provided by

$$\begin{aligned}
 DI_{P/A,Global} = & -0.072 + 2.331PGA + 0.359PGV \\
 & + \frac{0.046PGA}{PGV} + 0.043Arias \\
 & - 0.322RMS_a + 0.003T_{0.90} - 0.019P_{0.90} \\
 & + 0.410SI_H - 0.520SI_K - 2.726SI_{MR} \\
 & - 0.873EPA + 0.019EPA_{max} - 0.048E_{inp} \\
 & - 0.397CAV - 1.475DP_{A/S} - 0.167CP \\
 & - 2.292SD + 1.802SV + 0.236SA \\
 & + 0.024I_{F/V/F},
 \end{aligned} \tag{19b}$$

$$\begin{aligned}
 DI_{P/A,Global} = & -0.090 + 0.293PGA + 0.094PGV \\
 & + \frac{0.017PGA}{PGV} + 0.001Arias \\
 & - 0.035RMS_a + 0.0002T_{0.90} - 0.005P_{0.90} \\
 & + 0.019SI_H + 0.237SI_K - 0.178SI_{MR} \\
 & - 0.288EPA + 0.027EPA_{max} + 0.081E_{inp} \\
 & - 0.032CAV - 0.656DP_{A/S} + 0.097CP \\
 & + 0.648SD - 0.079SV - 0.040SA \\
 & + 0.011I_{F/V/F}.
 \end{aligned} \tag{19c}$$

TABLE 9: Multilinear regression model for the OSDI of DiPasquale and 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.

The initial set of explanatory variables (see (19a), (19b), and (19c)) contains a relative large number of seismic intensity parameters. Thus, in the present study twenty seismic intensity quantities have been used (Table 1) to avoid an a priori omission of important, independent (explanatory) seismic parameters. A regression model with only a few explanatory variables (e.g., two to three accidentally selected parameters) runs the risk of becoming an unreliable or invalid regression model. However, a statistically systematic reduction of the initial regression model is possible. This reduction has been realized in the present study by a stepwise backward elimination procedure combined with an elimination criterion.

Thus, the multilinear regression model, which contains the twenty examined seismic parameters (Model 1), can be simplified to consider fewer parameters without losing significant model quality. This simplification is realized by successive elimination of the independent variables with a P value greater than or equal to 0.05 (elimination criterion) because that term is not statistically significant at the 95% or higher confidence level. This lack of significance is explained by the strong correlation between several seismic parameters.

The P value is the smallest level of significance that would lead to rejection of the null hypothesis H_0 with the given data.

FIGURE 3: P values in ascending order for all of the terms of regression equation (19a).

In the case of multilinear regression analysis, the null hypothesis (H_0) and alternative hypothesis (H_a) are $H_0 : b_i = 0$ and $H_a : b_i \neq 0$, where b_i are the coefficients of the explanatory variables in (18) [30]. Thus, the importance to keep a seismic intensity parameter in the regression model is determined by the previously described explicit statistical procedure.

As an example, Figure 3 presents the P values for all of the terms of regression equation (19a) (Table 6) in ascending order. The SI after Kappos parameter has the maximum P value (0.809) which is greater than 0.05. Therefore, this specific parameter can be eliminated from the considered model. Subsequently, a new regression equation is evaluated without the Kappos SI parameter in its explanatory variables, and the corresponding P values are different from those of the initial equation (19a) (with all of the seismic parameters considered initially). In the new regression equation the explanatory variable (seismic parameter), with the maximum P value greater than 0.05, will be eliminated. This elimination procedure is repeatedly applied until all of the P values of the explanatory variables remain less than 0.05. Applying the described procedure to (19a), the sequence of the eliminated parameters is SI after Kappos, SA, SD, SMD, $I_{F/V/F}$, $P_{0.90}$, CP, $DP_{A/S}$, and PGA/PGV. The result of this procedure is the reduced regression model shown in Table 10. It is not recommended to eliminate all of the explanatory variables that have a P value greater than 0.05 in Figure 3 at once from the regression model in (19a) because the P values are different in the case of stepwise elimination of explanatory variables and in the case of their at-once elimination.

TABLE 10: Reduced multilinear regression model for the OSDI of Park and Ang.

Parameter	Estimate	Lower limit	Upper limit
Constant	-0.015	-0.031	0.001
PGA	0.367	0.235	0.500
PGV	0.111	0.017	0.206
I_0	0.007	0.005	0.009
RMS_a	-0.082	-0.149	-0.014
SI_H	0.037	0.003	0.072
SI_{MR}	-0.354	-0.496	-0.212
EPA	-0.430	-0.683	-0.176
EPA_{max}	0.086	0.016	0.157
E_{inp}	0.023	0.012	0.035
CAV	-0.028	-0.046	-0.009
SV	0.166	0.120	0.213

TABLE 11: Reduced multilinear regression model for the MISDR.

Parameter	Estimate	Lower limit	Upper limit
Constant	0.003	-0.104	0.110
PGA	2.217	1.309	3.126
I_0	0.039	0.029	0.050
SI_H	0.190	0.041	0.339
SI_{MR}	-2.542	-3.393	-1.691
CAV	-0.353	-0.475	-0.231
$DP_{A/S}$	0.974	0.281	1.668
SD	-2.431	-4.541	-0.320
SV	1.836	1.413	2.259

Thus, in accordance with the proposed stepwise elimination procedure with a P value greater than 0.05 criterion, Tables 10, 11, and 12 show the reduced multilinear models (Model 2) for the OSDI of Park and Ang ($R^2 = 0.856131$), the MISDR ($R^2 = 0.795159$), and the OSDI (maximum softening index) of DiPasquale and Cakmak ($R^2 = 0.757862$) with their respective 95% confidence intervals.

The independent variables in the reduced regression model are selected by the proposed statistical procedure primarily to predict the damage indicator and not to explain it physically. Therefore, it is not unexpected that different seismic parameters are used for the adequate prediction of different damage indices (Tables 10–12).

The multilinear regression is a robust procedure, even for the case in which two or more explanatory variables are linearly dependent or highly correlated. Perfect multicollinearity occurs when two or more explanatory variables in the multilinear regression are linearly dependent. This multicollinearity type is rare. Imperfect multicollinearity occurs when two or more regressors are highly correlated. Perfect and imperfect multicollinearity does not affect the overall predictive power or reliability of the regression model. However, multicollinearity does reduce the effectiveness of a regression analysis if its primary purpose is to determine the specific effects of the various independent variables. The goal of the present study is to predict the dependent

TABLE 12: Reduced multilinear regression model for the OSDI of DiPasquale and Cakmak.

Parameter	Estimate	Lower limit	Upper limit
Constant	-0.092	-0.126	-0.059
PGA	0.289	0.131	0.448
PGA/PGV	0.015	0.005	0.024
SI_K	0.191	0.073	0.310
SI_{MR}	-0.243	-0.398	-0.087
E_{inp}	0.085	0.063	0.107
CAV	-0.021	-0.038	-0.005
$DP_{A/S}$	-0.743	-0.995	-0.491
CP	0.121	0.051	0.190
SD	0.468	0.144	0.793

variable (damage indices) from a set of independent variables (seismic intensity parameters); therefore, multicollinearity does not affect the quality of the proposed multilinear regression model. Nevertheless, the proposed stepwise backward elimination procedure reduces the number of independent variables and consequently provides models with less multicollinearity. An a priori omission of several independent variables includes the risk of becoming an untrustworthy model.

7. Testing the Multilinear Regression Model

To verify the quality of the full (Model 1) and the reduced (Model 2) regression models, in general, it is sufficient that the validation data set is a fraction of the training data set. In the present study, the validation data consists of 410 accelerograms, separated into two sets. The first one is identical to the 400 accelerograms used for the training phase of the regression model, as shown in Table 2. The second set is from an entirely different set of 10 artificial accelerograms. In their original form, the latter are compatible with the design spectrum employed. Thus, set 1 is used as internal and set 2 as external validation set. This is common validation procedure in statistical analyses. Two utilized artificial accelerograms are presented in Figure 4. Because the examined frame structure 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 the considered seismic accelerogram have been multiplied by a factor with a value greater than one) to achieve medium, large, and total damage grades. Thus, some of these accelerograms have been scaled to produce medium, large, and total structural damages for each of the examined damage indicators. The program SIMQKE [31] 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

FIGURE 4: Two utilized artificial accelerograms.

TABLE 13: Damage prediction results for the regression models.

Regression	OSDI	Correctly predicted damage values	
		Set 1 400 accelerograms	Set 2 10 accelerograms
Model 1	$DI_{P/A,Global}$	386 (96.50%)	9 (90%)
	MISDR	390 (97.50%)	8 (80%)
	δ_M	379 (94.75%)	8 (80%)
Model 2	$DI_{P/A,Global}$	385 (96.25%)	9 (90%)
	MISDR	388 (97.00%)	8 (80%)
	δ_M	380 (95.00%)	7 (70%)

are then enveloped in a trapezoidal shape to simulate the nonstationary characteristics of the ground motion [31]. 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 parts of the trapezoidal envelope, and design spectrum with which the artificial accelerograms must be compatible. Another option is to use natural accelerograms instead of the artificial accelerograms to verify the proposed models. Scaling an accelerogram to a greater PGA value is a simple procedure to produce accelerograms with more energy content and damage potential. The seismic intensity parameters have been evaluated also for all the utilized scaled accelerograms because they change with the scaling process.

Table 13 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 6–12. Thus, the results of Model 1 display correct predictions, from 94.75% to 97.50% and from 80% to 90% for acceleration sets 1 and 2, respectively. In addition,

FIGURE 5: Regression line and the 95% confidence interval of the correct versus predicted Park and Ang OSDI values.

Figure 5 shows the regression line and the 95% confidence interval of the correct versus predicted Park and Ang OSDI values. In this case the coefficient of determination $R^2 = 85.9\%$. This indicates that the model as fitted explains 85.9% of the variability in Park and Ang damage index. Furthermore, the percentages of correct prediction for Model 2 are from 95% to 97% and from 70% to 90% for acceleration sets 1 and 2, respectively. Model 2, with its reduced number of seismic parameters in the regression model, is not of poorer quality than Model 1, and the results of acceleration set

TABLE 14: Correlation coefficients between correct and predicted OSDI values based on regression analyses.

Regression	OSDI	Correlation coefficients			
		Set 1		Set 2	
		400 accelerograms		10 accelerograms	
		Pearson correlation	Spearman rank correlation	Pearson correlation	Spearman rank correlation
Model 1	DI _{P/A,Global}	0.927	0.912	0.975	0.953
	MISDR	0.894	0.921	0.979	0.958
	δ_M	0.874	0.841	0.962	0.833
Model 2	DI _{P/A,Global}	0.925	0.913	0.977	0.953
	MISDR	0.892	0.933	0.982	0.967
	δ_M	0.871	0.846	0.982	0.827

2 are satisfactory, particularly for the OSDI of Park and Ang and the MISDR. Finally, Table 14 shows the Pearson and the Spearman rank correlation coefficients between the correct and predicted values of all examined OSDIs based on regression analyses. The numerical results show a high correlation in all cases.

8. Discriminant Analysis

In the case for which it is sufficient to know the Postseismic damage grade as provided in Table 5 (low, medium, large, and total), then a statistical discriminant analysis can be performed. The purpose of discriminant analysis is to classify objects into one or more possible groups based on a set of features that describe the objects [32]. In general, we assign an object to one of a number of predetermined groups based on observations made about the object. The discriminant analysis creates an equation which will minimize the possibility of misclassifying cases into their respective groups or categories.

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

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

where F_i are the discriminant functions, $a_{j,i}$ 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). The discriminant analysis has a direct analytical solution and therefore, calculations are very fast. The produced models are concise and easily programmed. On the other hand, the discriminant analysis is sensitive to outliers.

9. Testing the Discriminant Analysis Model

To verify the quality of the discriminate analysis, the same two sets of accelerograms from the regression analysis have

TABLE 15: Damage prediction results for the discriminant analysis.

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

been used. Table 15 presents the prediction results of all of the examined damage indicators. In this context, 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. Figure 6 shows the correct versus predicted damage grade based on the Park and Ang OSDI values. The points out of the diagonal line show the incorrect classified cases. Thus, 9 cases have been classified in damage group 2 (medium damage) instead of the correct damage group 1 (low damage) and 1 case has been classified in damage group 2 (medium damage) instead of the correct damage group 4 (total damage). The points in the diagonal line show the correct classified cases (390 cases, 97.5%). The corresponding discriminating functions have been evaluated with P values less than 0.05 and are statistically significant at the 95.0% confidence level. Thus, all points in Figure 6 are inside the 95.0% confidence interval.

10. Remarks on the Proposed Statistical Procedures

The proposed regression models (Tables 6–12) and discriminant analysis model are clearly tailored to the examined reinforced concrete frame structure. For another structure, the nonlinear dynamic analyses should be repeated to determine the coefficients in the regression models (Tables 6–9) and in the discriminant analysis model. Although the proposed procedures are applied to a two-dimensional reinforced concrete frame as an example, they could be applied to any two-dimensional frame and expanded to three-dimensional frames for other building materials (e.g., steel, masonry, timber) or to other building types (e.g., bridges, shell structures).

FIGURE 6: Discriminant analysis results. Correct versus predicted damage grade based on the Park and Ang OSDI values.

Furthermore, the set of twenty seismic intensity parameters considered here could be extended with additional quantities or partially or totally replaced by other parameters. However, the proposed procedure remains the same regardless of the structure examined and seismic intensity parameters employed. Thus, in the first step, a set of seismic intensity parameters is evaluated for a set of accelerograms. Then, nonlinear dynamic analyses furnish the damage indices of the considered structure. In the final step, the coefficients of the proposed statistical models (multivariate regression and discriminant analysis) are determined.

The proposed procedures are simple and accurate tools for the estimation of the damage index of a building by a multivariate regression analysis and of its Postseismic damage grade by discriminant analysis. These multivariate statistical techniques do not replace the exact nonlinear dynamic analyses, which are necessary to determine the coefficients of the statistical models, but they do serve as a simple utility for the Postseismic damage status estimation of a specific building based on seismic intensity parameters. Evaluation of the coefficients in statistical models, in which nonlinear dynamic analyses are obligatory, must be performed by professional engineers. If the models have been defined, then they may feasibly be applied by nonspecialists. The presumption is that the seismic parameters employed are known or estimated. The nonlinear analyses and determination of the proposed statistical models are complicated and time-consuming numerical procedures that must be performed by professional engineers (e.g., this can be realized in connection with the design procedure of a considered new building and the evaluated statistical models can be announced to the

building owner and the housing authority). The statistical models are determined by the evaluation of the coefficients in (18) and (20) for the regression and discriminant models, respectively. These statistical models can be easily and quickly applied once they are resolved. They could also be used by nonspecialists under the assumption that the seismic intensity parameters for an actual seismic event are provided by the responsible authorities (e.g., seismological institutes). The proposed methods convert the complicated dynamic nonlinear models to black boxes that take the ground motion information and predict the Postseismic damage levels. Thus, as mentioned in the introduction section, the proposed procedures can be applied to damage scenarios of important buildings (e.g., hospitals, schools, bridges, silos, and hangars) by public authorities. The proposed statistical models are faster and easier to use than the nonlinear dynamic analyses. Epigrammatic, nonlinear dynamic analyses can be utilized only by specialists, while on the other hand, the proposed statistical models can be utilized also by nonspecialists. Thus, nonspecialists can approximately evaluate the Postseismic damage status of buildings, ignoring the exact damage values provided by nonlinear dynamic analysis. The exact damage value needs not to be known once the statistical models are resolved. Furthermore, the proposed methods can handle probable uncertainties in the seismic ground motion and variability in structural or material properties by parametric analyses or by their combination with fuzzy logic procedures (fuzzy regression analysis). Finally, the proposed method can also be applied to a group of buildings. In that case, the damage parameters must be appropriately tailored to describe the Postseismic damage status and the proposed statistical procedures can be applied once on the group and not repeatedly on separate buildings. A proper damage status description could be a financial damage index [25].

Alternative procedures for seismic damage potential classification are presented in the literature and are based on intelligent techniques. Thus, fuzzy logic, artificial neural network, neurofuzzy and adaptive neurofuzzy inference systems, and support vector machines techniques have been used for seismic damage potential classification. The efficiency of the intelligent techniques was expressed by the correct classification rates of a set of accelerograms that were not used during the training process. The intelligent techniques provided a correct classification rate between 71% and 97.5%. The results depend on the used procedure, the overall structural damage indices ($MISDR$ or $DI_{P/A,Global}$), and the number of training samples [33]. The intelligent techniques in comparison with the proposed statistical procedures have no significant better correct classification rates and are not appropriate to be used by nonspecialists.

11. Conclusions

Two statistical methods have been presented for the evaluation of the Postseismic 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 and Ang, the MISDR, and the maximum softening of DiPasquale and 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 Postseismic 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 in accordance with 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, a second set of 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 in accordance with their damage potential for the examined structure within the confidence interval provided by the regression analysis. In addition, the regression model with a reduced number of considered seismic parameters is not of poorer quality than the model that takes all twenty seismic parameters into account. In contrast, 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. Finally, based on the regression and the discriminant analysis results of the 10 examined artificial accelerograms (set 2 in Tables 13 and 14), it can be suggested to use the Park and Ang OSDI and the maximum softening index of DiPasquale and Cakmak as appropriate Postseismic damage value and damage grade estimators of a building, respectively.

Conflict of Interests

The author declares that there is no conflict of interests regarding the publication of this paper.

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