Second Conference on Smart Monitoring,
Assessment and Rehabilitation of Civil Structures

SMAR 2013

Effect of confinement on damage in reinforced concrete frames subjected to different seismic levels

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ABSTRACT: Concrete in reinforced concrete frame columns can be made confined by the inclusion of stirrups or transverse reinforcement. Confinement is one of several important factors which are needed to be taken into account when designing new structures or evaluating existing buildings. It can greatly improve the energy absorption capacity of columns resulting in lower damage levels than expected for structures subjected to earthquake excitations. The aim of this paper is to investigate the effect of confinement on the damage of a reinforced concrete frame subjected to different seismic levels based on current codes. The results show that the damage indices significantly reduce and the damage states change from moderate/severe to minor/moderate as the confinement of concrete increases.

1 INTRODUCTION

Concrete in reinforced columns can be made confined by the inclusion of adequately sized and spaced stirrups or transverse reinforcement. Uni-axial stress condition can be accepted in cases of unconfined concrete modelled by Hognestad (1951). However, the effect of lateral stress should be considered in case of concrete confined by transverse reinforcement. Many studies have been performed and various models for the stress-strain relationship have been proposed for confined concrete (Cusson and Paultre, 1994a, 1994b; Kent and Park, 1971; Mander *et al*, 1988; Park *et al*, 1982; Sheikh and Uzumeri, 1982). These attempts were to introduce appropriate models allowing for the incorporation of the confinement-induced enhanced maximum stress and its corresponding strain, and were focused on a single column. None, however, looked at the confinement effect on damage of frames subjected to seismic loads. It is important that the seismic capacity of reinforced concrete (RC) frames designed according to current codes is checked versus the seismic demand considering the confinement effect particularly as the inadequacy of many existing frames has been identified (Bracci *et al*, 1995). Considering the above, the current study looks at the potential damage suffered by an RC frame (expressed in terms of "damage index") allowing for the effect of confinement in columns. The results of the numerical analyses that are calibrated with experiments show that the evaluated potential damage is significantly reduced when confinement increases.

2 BEHAVIOUR OF CONCRETE

Hognestad (1951) model is commonly used for unconfined concrete. However, the strength of concrete increases significantly when confined by transverse reinforcement. Concrete confined by rectangular hoops has been extensively studied by researchers (Baker and Amarakone, 1964; Blume *et al*, 1961; Kent and Park, 1971; Sargin *et al*, 1971; Soliman and Yu, 1967). Figure 1 shows Kent and Park (1971) model which does not take into account the increase in maximum stress of confined concrete (Park and Paulay, 1975). In recognition of the issues in the Kent and Park (1971) model, Park et al (1982) modified the model shown in Figure 2, in which the maximum stress f'_c and the corresponding strain of 0.002 in Kent and Park (1971) model are multiplied by the factor K as shown in Equations 1 to 6, where ρ_s is the ratio of the volume of rectangular steel hoops to the volume of concrete core measured to the outside of the peripheral hoop; f'_c is in MPa; b'' is the width of the concrete core measured to outside of the peripheral hoop; s_h is the center-to-center spacing of hoop sets. This modified Kent and Park (1971) model agrees well with the test results of compressed concrete confined by hoop reinforcement presented by Scott et al (1982). For above reasons, the modified Kent and Park (1971) model is used in this study.

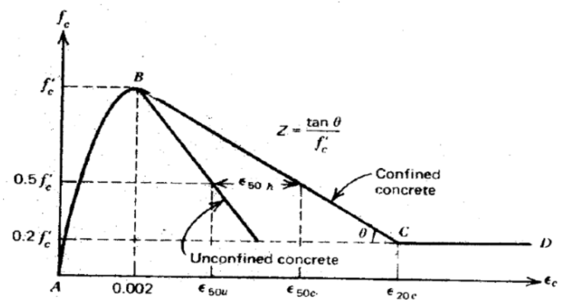


Figure 1. Kent and Park (1971) model for concrete confined by rectangular hoops.

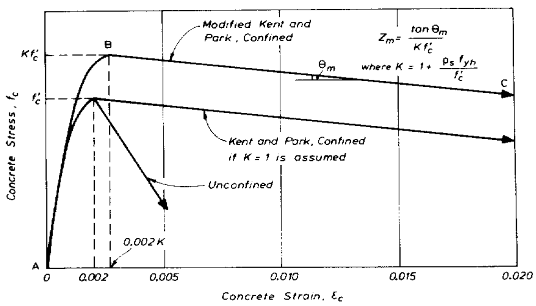


Figure 2. Modified Kent and Park (1971) model done by Park et al (1982).

Region AB: $\varepsilon_c \le \varepsilon_0$: $f_c = f_c''\left[\frac{2\varepsilon_c}{\varepsilon_o} - \left(\frac{2\varepsilon_c}{\varepsilon_o}\right)^2\right]$ (1)

Region BC: $\varepsilon_c \ge \varepsilon_0$: $f_c = f_c''\left[1 - Z\left(\varepsilon_c - \varepsilon_0\right)\right] \ge 0.2 f_c''$ (2)

In which: $f_c'' = Kf_c'$ (3); $\varepsilon_0 = 0.002K$ (4)

$$Z = \frac{0.5}{\dfrac{3+0.29 f_c'}{145 f_c' - 1000} + \dfrac{3}{4}\rho_s\sqrt{\dfrac{b''}{s_h}} - 0.002K}$$ (5); $K = 1 + \rho_s f_{yh} / f_c'$ (6)

3 SELECTION OF GROUND MOTIONS FOR DIFFERENT SEISMIC LEVELS

The Pacific Earthquake Engineering Research Center database software (PEER, 2011) is used for selection of ground motions. The soil profile type S_D (stiff soil profile) and seismic source type A are assumed for the location of the structures in order to select the seismic records. Six different seismic levels or zones of 1, 2A, 2B, 3, 4 Far-Fault (4FF) and 4 Near-Fault (4NF) are presented by the acceleration spectra based on UBC (ICBO, 1997) code. Sets of 14 fault-normal and fault-parallel ground motion records of seven earthquakes represent different seismic levels.

Table 1. Parameters for design spectrum.

Seismic zone factor, Z	N_a	N_v	C_a	C_v
0.075	-	-	0.12	0.18
0.15	-	-	0.22	0.32
0.2	-	-	0.28	0.40
0.3	-	-	0.36	0.54
0.40 (far fault (≥ 10km))	1.0	1.0	0.44	0.64
0.40 (near-fault (≤ 2km))	1.5	2.0	0.66	1.28

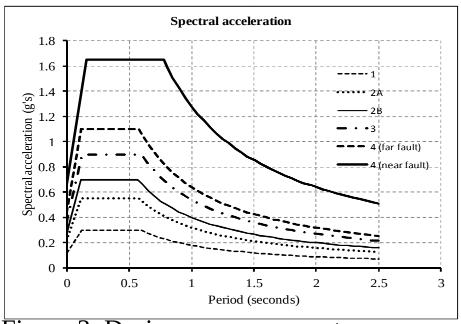


Figure 3. Design response spectrum.

The Joyner-Boore distance (R_JB) and the closest distance (R_rup) to the rupture plane are assumed to vary from 0 to 2 km for zone 4 near-fault and from 20 to 200 km for other zones. The selected records are then scaled to match the design response spectrum based on UBC

(ICBO, 1997) at the fundamental period of structure. Table 1 shows parameters of design spectra based on the UBC code, in which N_a, N_v are near source factors and C_a, C_v are seismic coefficient factors. Figure 3 shows the design response spectra established based on these parameters. Tables 2 shows six sets of earthquakes with scaled factors obtained from the Pacific Earthquake Engineering Research Center database software (PEER, 2011). Each earthquake represented by both fault-normal and fault-parallel components makes the total of 84 records applied for the analyses.

Table 2. Earthquakes for six zones.

Zone-Earthquake	NGA#	Scaled factor	Earthquake	Year	Station	Mag.
1-01	2478	4.15	Chi-Chi, Taiwan-03	1999	CHY056	6.2
1-02	644	1.93	Whittier Narrows-01	1987	LB-Harbor Admin FF	5.99
1-03	3498	3.14	Chi-Chi, Taiwan-06	1999	TCU113	6.3
1-04	1318	1.44	Chi-Chi, Taiwan	1999	ALAO14	7.62
1-05	153	1.57	Coyote Lake	1979	SJB Overpass Bent 5 g.l.	5.74
1-06	1804	13.28	Chi-Chi, Taiwan-04	1999	KAU001	6.2
1-07	1952	27.45	Anza-02	2001	Mill Creek Ranger Station	4.92
2A-01	1096	3.4	Northridge-01	1999	Wrightwood-Jackson Flat	6.69
2A-02	2536	26.68	Chi-Chi Taiwan-03	1999	HWA033	6.2
2A-03	294	8.82	Irpinia, Italy-01	1980	Tricarico	6.9
2A-04	2162	15.1	Chi-Chi Taiwan-02	1999	CHY027	5.9
2A-05	2940	6.38	Chi-Chi Taiwan-05	1999	CHY019	6.2
2A-06	1256	3.71	Chi-Chi Taiwan	1999	HWA002	7.62
2A-07	2384	9.85	Chi-Chi Taiwan-02	1999	TCU068	5.9
2B-01	2209	26.64	Chi-Chi Taiwan-02	1999	CHY107	5.9
2B-02	2698	25.44	Chi-Chi Taiwan-04	1999	CHY022	6.2
2B-03	2921	19.79	Chi-Chi Taiwan-04	1999	TTN027	6.2
2B-04	2162	18.88	Chi-Chi Taiwan-02	1999	CHY027	5.9
2B-05	2240	14.23	Chi-Chi Taiwan-02	1999	HWA033	5.9
2B-06	2752	2.39	Chi-Chi Taiwan-04	1999	CHY101	6.2
2B-07	2536	33.34	Chi-Chi Taiwan-03	1999	HWA033	6.2
3-01	2478	12.44	Chi-Chi, Taiwan-03	1999	CHY056	6.2
3-02	644	5.8	Whittier Narrows-01	1987	LB-Harbor Admin FF	5.99
3-03	3498	9.43	Chi-Chi, Taiwan-06	1999	TCU113	6.3
3-04	1318	4.33	Chi-Chi, Taiwan	1999	ILAO14	7.62
3-05	2804	39.77	Chi-Chi, Taiwan-04	1999	KAU001	6.2
3-06	153	4.7	Coyote Lake	1979	SJB Overpass Bent 5 g.l.	5.74
3-07	1952	82.36	Anza-02	2001	Mill Creek Ranger Station	4.92
4FF-01	1096	6.81	Northridge-01	1994	Wrightwood-Jackson Flat	6.69
4FF-02	2536	53.35	Chi-Chi, Taiwan-03	1999	HWA033	6.2
4FF-03	294	17.64	Irpinia, Italy-01	1980	Tricarico	6.9

4FF-04	2162	30.21	Chi-Chi, Taiwan-02	1999	CHY027	5.9
4FF-05	2940	12.77	Chi-Chi, Taiwan-05	1999	CHY019	6.2
4FF-06	1256	7.42	Chi-Chi, Taiwan	1999	HWA002	7.62
4FF-07	2384	19.71	Chi-Chi, Taiwan-02	1999	TCU068	5.9
4NF-01	171	2.21	Imperial Valley-06	1979	EC Meloland	6.53
4NF-02	181	2.56	Imperial Valley-06	1979	El Centro Array #6	6.53
4NF-03	1120	1.26	Kobe, Japan	1995	Takatori	6.9
4NF-04	1106	1.08	Kobe, Japan	1995	KJMA	6.9
4NF-05	1119	1.2	Kobe, Japan	1995	Takarazuka	6.9
4NF-06	1503	1.43	Chi-Chi, Taiwan	1999	TCU065	7.62
4NF-07	1529	2.82	Chi-Chi, Taiwan	1999	TCU102	7.62

4 DESCRIPTION AND MODELLING OF A TESTED FRAME

The model shown in Figure 4 is a one-third scaled three-storey reinforced concrete frame designed only for the gravity load. Its dimensions (in inches) and reinforcing details are shown in Figure 5. Concrete strength f'_c=20.2 to 34.2 MPa (average f'_c= 27.2 MPa), the average E_c=24200 MPa. Four types of reinforcement were used and their properties are shown Table 3.

Table 3. Properties of reinforcement.

Reinforcement	Diameter	Yield strength	Ultimate strength	Modulus	Ultimate strain
D4	5.715	468.86	503.34	214089.8	0.15
D5	6.401	262.01	372.33	214089.8	0.15
12 ga.	2.770	399.91	441.28	206160.5	0.13
11 ga.	3.048	386.12	482.65	205471	0.13

The total weight of each floor was found to be approximately 120 kN. Further details of this model can be found in (Bracci, 1992) and (Bracci *et al*, 1995). The seismic record selected for simulation was the N21E ground acceleration component of Taft earthquake with peak ground accelerations (PGA) of 0.05g, 0.20g and 0.30g representing minor, moderate and severe shaking, respectively. The axial loads in columns are assumed to be constant during excitations. Figure 6 shows the model with LINK elements in SAP2000. The Moment-Rotation of a LINK element is determined using fiber model and a plastic hinge length L_p. In this study, L_p=4d, where d is the dimension of cross section, as proposed by Sheikh and Khoury (1993) is used. The structural frequencies of the first three mode shapes are determined in Table 4 in comparison with the experimental results. These are very close in the first and second modes, but are slightly different in the third mode. Table 5 presents a comparison between the experimental (Bracci *et al*, 1995) and the analytical results in terms of maximum inter-storey drift and maximum storey displacement. Though not an exact match, the model provides an overall good approximation.

5 DAMAGE ANALYSIS

5.1 Selection of the damage model

Damage index can be classified into two types: non-cumulative and cumulative. Cumulative damage models are more rational for evaluating the damage states of structures, especially for those that experience cyclic loading or earthquake excitation because the damage of structures depends not only on the response magnitude but also on the number of load cycles (Colombo and Negro, 2005). Park and Ang (1985) proposed a cumulative DI based on deformation and hysteretic energy resulted from an earthquake as shown in Equation 7.

$$DI = u_m / u_u + \beta E_h / F_y u_u \qquad (7)$$

where, u_m is the maximum displacement of a single-degree-of-freedom (SDOF) system subjected to earthquake, u_u is the ultimate displacement under monotonic loading, E_h is the hysteretic energy dissipated by the system, F_y is the yield force and β is a parameter to include the effect of repeated loading.

Figure 4. Model of three storey frame (Bracci *et al*, 1995).

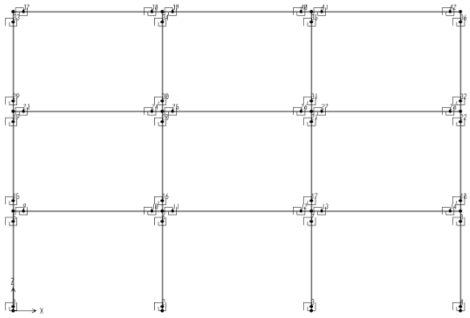


Figure 6. Modelling of the 3-storey frame with LINK elements.

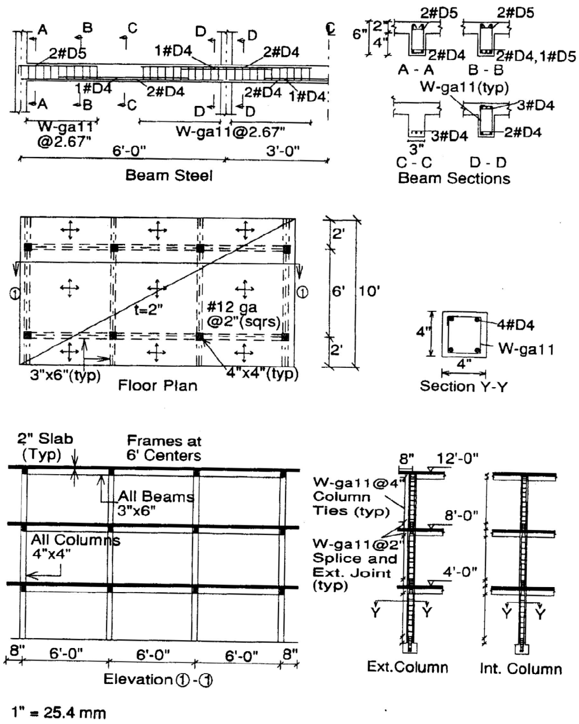


Figure 5. Dimensions and reinforcement arrangement of three storey frame model (Bracci *et al*, 1995).

Table 4. Modal frequencies (Hz).

Mode	Experiment (Bracci *et al*, 1995)	Model
1	1.78	1.70
2	5.32	5.30
3	7.89	9.03

Table 5. Comparison between experimental (Bracci *et al*, 1995) and analytical results.

PGA	Storey	Maximum inter-storey drift (%)		Maximum storey displacement (mm)	
		Experiment	Model	Experiment	Model
0.05g	3	0.23	0.21	7.6	7.9
	2	0.24	0.25	5.6	5.6
	1	0.28	0.23	3.6	2.8
0.20g	3	0.54	0.83	33.5	38.9
	2	1.07	1.17	29.0	30.7
	1	1.33	1.31	16.3	16.0
0.3g	3	0.89	1.18	59.7	58.4
	2	2.24	1.91	52.1	46.1
	1	2.03	1.96	24.6	23.9

Park and Ang (1985)also proposed a definition for different damage states: DI<0.1: No damage; 0.1≤DI<0.25: Minor damage; 0.25≤DI<0.40: Moderate damage; 0.4≤DI<1.00: Severe damage; DI≥1.00: Collapse. DI≥0.8 has been suggested to represent collapse (Tabeshpour *et al*, 2004). It is worth noting that Park and Ang model is widely used. Bassam *et al*(2011), Ghosh *et al*(2011), and Yüksel and Sürmeli (2010) are examples of recent use. This model is also selected in the current study.

5.2 Results

Damage analyses are conducted for the frame with four stirrup spacing of 0.25d, 0.5d, 0.75d and d, where d is the width of the columns' cross section. This range covers the stirrup spacing regulated in the building code (ACI, 2008), namely, close to 0.5d for intermediate moment frame (min{$8d_{rebar}$, $24d_{hoop}$, 0.5d}) and 0.25d for special moment frame (min{0.25d, $8d_{rebar}$, $24d_{hoop}$, 300mm}), in which d_{rebar} and d_{hoop} are the diameters of the rebar and hoop, respectively.

It is worth noting that the stirrup spacing d of the model was designed to represent a deficiency in the existing frames designed based on the older codes (Bracci *et al*, 1995).

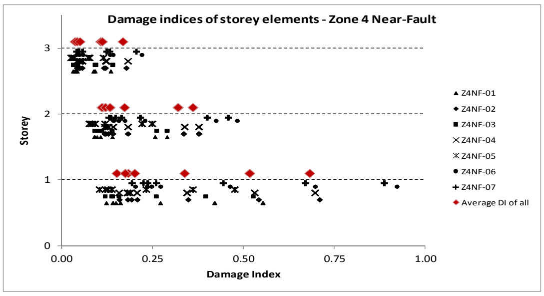
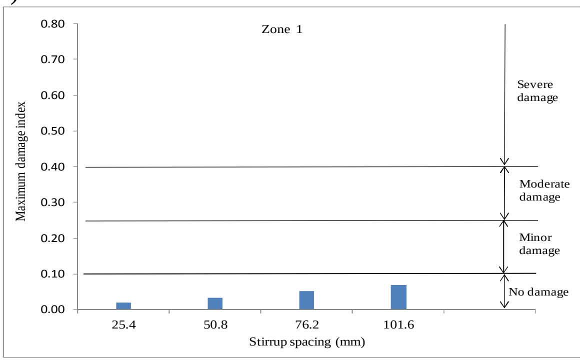


Figure 7. Damage analysis of the frame.

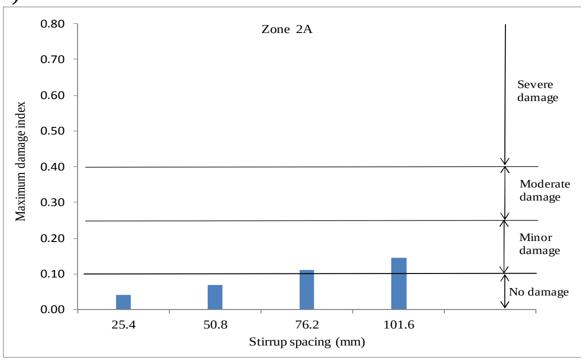
Table 6. Effect of stirrup spacing on damage.

Seismic zone	d/(stirrup spacing) 4	2	1.3
1	3.4	2.1	1.3
2A	3.5	2.1	1.3
2B	3.5	2.1	1.3
3	3.4	2.1	1.3
4FF	3.4	2.1	1.3
4NF	3.8	2.1	1.3
Average	3.5	2.1	1.3

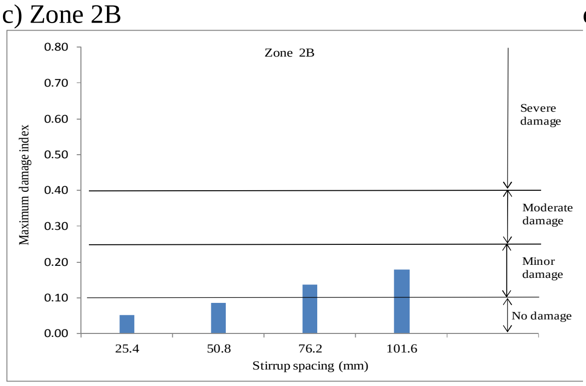
a) Zone 1



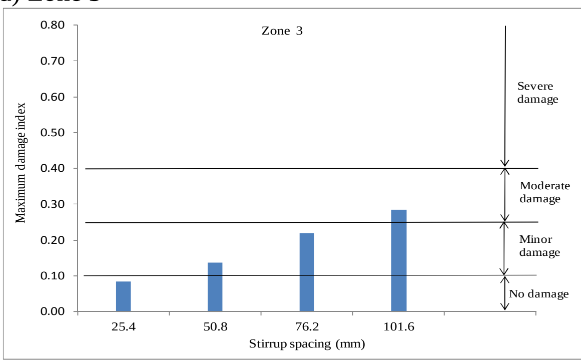
b) Zone 2A



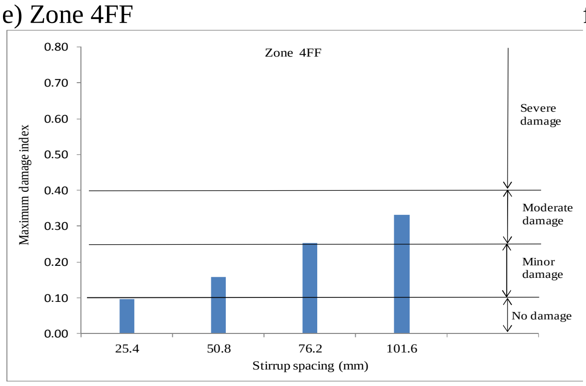
c) Zone 2B



d) Zone 3



e) Zone 4FF



f) Zone 4NF

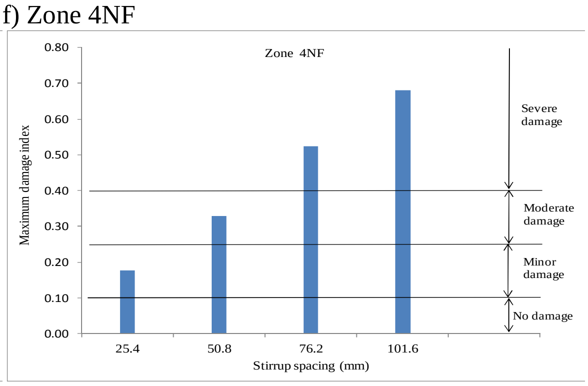


Figure 8. Maximum damage indices with the different seismic zones.

Time history and damage analyses are performed for 336 combinations of 14 seismic records, 6 seismic zones and 4 stirrup spacings. Figure 7 shows one representative example of 24 combinations of seismic zones and stirrup spacings, namely, damage analyses for the frame with a particular stirrup spacing of d subjected to 14 near-fault earthquake records in zone 4. The damage indices at each location of the frame is plotted under the grid lines corresponding to the storey levels while the average damage indices from 14 records are plotted above the grid lines. Maximum of the average damage indices are compared in the following sections. Not presented in the Figure, however, it should be noted that the two inner columns of the first storey suffered the most severe damage in comparison with the others. This indicates that the damage of the frame depends heavily on the damage of these columns.

In order to represent the effectiveness of the confinement on reduction of damage, a comparison of the damage of the frame with the stirrup spacing of d to that of others (0.75d, 0.5d and 0.25d) is made using the damage reduction (DR), which is defined as the ratio of maxDI_d to maxDI_s as shown in Equation 8, where maxDI_d is the maximum damage index in corresponding to the stirrup spacing of d, maxDI_s is the maximum damage index in corresponding to the stirrup spacing s = 0.75d, 0.5d and 0.25d.

$$DR = \max DI_d / \max DI_s \tag{8}$$

Table 6 shows the DR for different seismic zones and d/s ratios. The average DRs are 1.3, 2.1 and 3.5 corresponding to the ratio d/s of 1.3, 2 and 4. For a certain ratio of d/s, the DRs for different zones are close to one another and close to the average value. This means the effect of confinement on damage reduction is almost similar for different zones. However, the effect of confinement on the damage states is different. Figure 8 shows the maximum damage indices for different seismic zones, which decreases as the stirrup spacing decreases. The frame does not suffer any damage as shown in Figure 8a if it located in seismic zone 1. The frame located in seismic zone 2A and 2 will suffer minor damage if its column stirrup spacing is larger than 0.75d. On the contrary, no damage occurs when the stirrup spacing is less than 0.75d. In seismic zones 3 and 4FF, the results demonstrate the effect of confinement in reducing the damage from moderate to no damage if the stirrup spacing is reduced 4 times from d to 0.25d which is close to the stirrup spacing in special moment frames. In addition, the effect of confinement can bring the moderate damage levels down to minor damage if the stirrup spacing is chosen closer to the one for intermediate moment frames. In seismic zone 4, the frame with the stirrup spacing larger than 0.5d will suffer severe damage. Intermediate frame will suffer moderate damage while special moment frame will suffer minor damage.

6 CONCLUSIONS

The important effect of confinement on reduction of potential seismic damage in reinforced concrete frames is presented in this paper. A previously tested three-storey frame is first analysed and the results compared with the experimental results. Upon agreement of the results, the same frame was subjected to 84 different seismic records representing 6 seismic levels. The effect of confinement on damage of the RC frame was investigated for four cases of stirrup spacing corresponding to d, 0.75d, 0.5d and 0.25d, making the total number of time history analysis equal to 336. The results show that stirrup spacing and maximum damage index are linearly dependent and that the frame located in seismic zones 1, 2A, 2B, suffers from minor to no damage. For the frame located in seismic zones 3 and 4, increasing the effect of confinement can be considered as a retrofitting method, which can reduce the damage of the frame from severe to minor or no damage. The effect of confinement is specifically beneficial to structures located in seismic zones 4, both near fault and far fault.

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