LIMITATIONS OF INTER-STOREY DRIFT IN SEISMIC DAMAGE ASSESSMENT OF STRUCTURES

CAO VAN VUI
Ho Chi Minh City University of Technology, Vietnam National University HCMC
Email: cvvui@hcmut.edu.vn

(Received: September 09, 2016; Revised: November 21, 2016; Accepted: December 06, 2016)

ABSTRACT

In this study, inelastic time history of 3-storey frames is performed for three seismic intensities and the inter-storey drifts are obtained. Damage distribution in the frames based on inter-storey drifts is then evaluated in comparison with the experimental and analytical damage. Though the "inter-storey drift" parameter due to its simplicity has been widely accepted in different seismic codes around the world, it is herein found that inter-storey drift suffers a number of limitations in interpreting the damage state of structures subjected to earthquakes.

Keywords: Damage index; Inter-storey drift; Damage assessment; RC frame; Seismic load.

1. Introduction

A large number of buildings in different parts of the world are vulnerable to earthquakes as is evident in the past earthquake events such as Northridge (1994), Kobe (1995), Chi-Chi (1999), Bam (2003), Christchurch (2011), etc. Mitigating the seismic hazards for these deficient structures, instead of replacing, has been increasingly looked at by the engineering community due to economic reasons. It may be the reason that strengthening existing deficient RC structures has become a widespread topic and can be found in several studies such as (Garcia, Hajirasouliha, & Pilakoutas, 2010; Ludovico, Prota, Manfredi, & Cosenza, 2008; Phan, Todd, & Lew, 1993). However, assessment should be performed, in prioritization, in order to identify, locate and quantify the damage potential in the existing structures under anticipated seismic loads as suggested by the current codes, providing information for the strengthening design. This evaluation of structural damage locations and quantifications plays an important role in efficiently design retrofitting solutions.

Evaluating the performance and damage of RC structures subjected to seismic loads has increasingly attracted researchers. Shaking table test seems to be favourable and performed by a number of researchers such as Bracci (1992), Garcia et al. (2010), Sharma et al (2012), etc. Due to the limited capacity of the shaking tables, pseudo-dynamic test is a choice for testing large structures, which was tried by Pinto et al (2002), Corte et al (2006), Ludovico et al (2008), etc. In these experimental studies, apart from the observation of damage, maximum inter-storey drift has been used as a main tool to evaluate the performance and damage of the structures. Also, it has been widely used as a tool to evaluate the structural damage due to its simplicity.

In this study, inelastic time history analyses are performed for a selected 3-storey reinforced concrete frame subjected earthquakes. The obtained inter-storey drift is used to evaluate the damage in the frame. The results of inelastic time history analyses are used to compute the damage indices of the frame using damage model. In comparison with of damage distribution in the frame based on damage model and experiment, the damage distribution based on inter-storey drift shows its limitation which presented in the following sections.

2. Inter-storey drift and damage models

2.1. Inter-storey drift

Inter-storey drift is a widely used parameter to evaluate the damage of structures. It is defined as the ratio of the maximum relative lateral displacement *Δu* of a storey or a building to the height of that particular storey or building *h*. Guidelines based on inter-storey drift to identify the damage states of structures are presented in available current codes or documents such as FEMA 356 (ASCE, 2000).

2.2. Damage models

Damage models can be categorised into two types: non-cumulative and cumulative. Cumulative damage models are more rational to evaluate damage states of structures subjected to cyclic loading or earthquake excitations. Therefore, only cumulative damage models are discussed herein. In a simple way, Banon and Veneziano (1982) used normalised cumulative rotation as a DI which is expressed by the ratio of the sum of inelastic rotations during half cycles to the yield rotation. Some years later, Park and Ang (1985) proposed a DI based on deformation and hysteretic energy due to an earthquake as shown in Equation 1. This is the best known and the most widely used DI (Kim, Lee, Chung, & Shin, 2005), largely due to its general applicability and the clear definition of different damage states provided in terms of DI. However, the following limitations are worth noting – DI > 0 when a structure works within elastic range and DI > 1 when the structure collapses with no specified upper limit for DI.

$$DI = \frac{u_m}{u_u} + \beta \frac{E_h}{F_y u_u} \tag{1}$$

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 SDOF system, F_y is the yield force and β is a parameter to include the effect of repeated loading.

Park and Ang (1985) classified damage states into the following five levels:

DI < 0.1: No damage or localized minor cracking.

$0.1 \le DI < 0.25$: Minor damage: light cracking throughout.

$0.25 \le DI < 0.40$: Moderate damage: severe cracking, localized spalling.

$0.4 \le DI < 1.00$: Severe damage: concrete crushing, reinforcement exposed.

DI ≥ 1.00: Collapse.

DI ≥ 0.8 has been suggested to represent collapse (Tabeshpour, Bakhshi, & Golafshani, 2004). Park and Ang (1985) also proposed DI for an individual storey and for an overall structure using the weighting factor based on the amount of hysteretic energy (E_i) absorbed by the element or the component.

Park and Ang's (1985) concept has been widely adopted and modified by researchers such as Fardis et al. (1993), Ghobarah and Aly (1998) and Bozorgnia and Bertero (2001). However, the most significant modification was made by Kunnath et al. (1992) who used the moment-rotation behaviour to replace the deformation terms used by Park and Ang (1985) and subtracted the recoverable rotation as shown in Equation 2, where, θ_m is the maximum rotation in loading history, θ_u is the ultimate rotation capacity, θ_r is the recoverable rotation when unloading and M_y is the yield moment. The merit of this modification is that DI will be 0 when structures work within elastic range. The major limitation to this proposal is, however, that the DI > 1 when the structure fails.

$$DI = \frac{\theta_m - \theta_r}{\theta_u - \theta_r} + \beta \frac{E_h}{M_y \theta_u} \tag{2}$$

The amount of energy absorbed by a structure is closely related to its corresponding damage state. Hence, DI may be expressed as the ratio of the hysteretic energy demand E_h to the absorbed energy capacity of a structure under monotonic loading $E_{h,u}$ (Cosenza, Manfredi, & Ramasco, 1993; Fajfar, 1992; Rodriguez & Padilla, 2009). However, this proposed DI has no specific upper limit to define the state of collapse.

It is obvious that the damage states of structures are closely related to residual deformation (ASCE, 2000). This concept was developed and a damage model was proposed by Cao et al. (2011). It is herein modified as shown in Equations 3 to 5.

$$DI = \left[\frac{E_h}{E_h + E_{rec}} \right]^{\alpha(N-i)} \qquad (3)$$

$$N = \frac{E_{h,1collapse}}{E_{h,1y}} \qquad (4)$$

$$i = \frac{E_h}{E_{h,1y}} \qquad (5)$$

where $E_{h,1collapse}$ and $E_{h,1y}$ are the hysteretic energy of one complete ultimate and yielding cycle, respectively. Equations 4 and 5 define the proposed parameters *N* and *i*. *N* is the equivalent number of yielding cycles to collapse whilst *i* is the equivalent number of yielding cycles at the current time of loading ($i \leq N$). α is a modification factor and is proposed as 0.06 and the damage levels are shown in Table 1, in which the legends in the first column corresponding damage levels are used to express the damage presented in Sections 5.

Table 1

Damage levels

Legend	Damage index	Description
.	>0 - 0.05	No or minor
+	0.05 - 0.25	Light
x	0.25 - 0.50	Moderate
▲	0.50 - 0.75	Severe
●	0.75 - 1.00	Collapse

3. Description of a tested three-storey frame (bracci, 1992)

The frame shown in Figure 1 is a one-third scale three-storey RC frame designed only for gravity load. Its dimensions (in inches) and reinforcing details are presented in Figure 2. Concrete strength varied from 20.2 to 34.2 MPa (the average can be taken as f_c' = 27.2 MPa), and the average modulus of elasticity was taken as $E_c = 24200$ MPa. Four types of reinforcement were used, and their properties are shown Table 2.

Table 2

Properties of reinforcement

Reinforcement	Diameter (mm)	Yield strength (MPa)	Ultimate strength (MPa)	Modulus (MPa)	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 dead loads were calculated from the self-weight of beams, columns, slabs and additional weights attached to the frame, as shown in Figure 1. The total weight of each floor was found to be approximately 120 kN. Further details of this frame can be found in (Bracci, 1992) and (Bracci, Reinhorn, & Mander, 1995). The seismic record selected for simulation was the N21E ground acceleration component of Taft earthquake occurred on 21 July 1952 at the Lincoln School Tunnel site in California. The peak ground accelerations (PGA) are 0.05g, 0.20g and 0.30g representing minor, moderate and severe shaking, respectively.

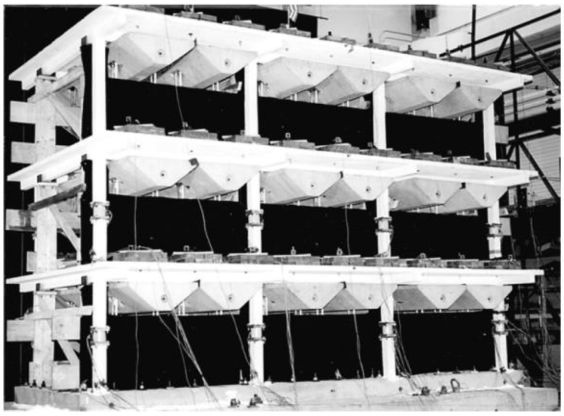

Figure 1. Three storey frame (Bracci, et al., 1995)

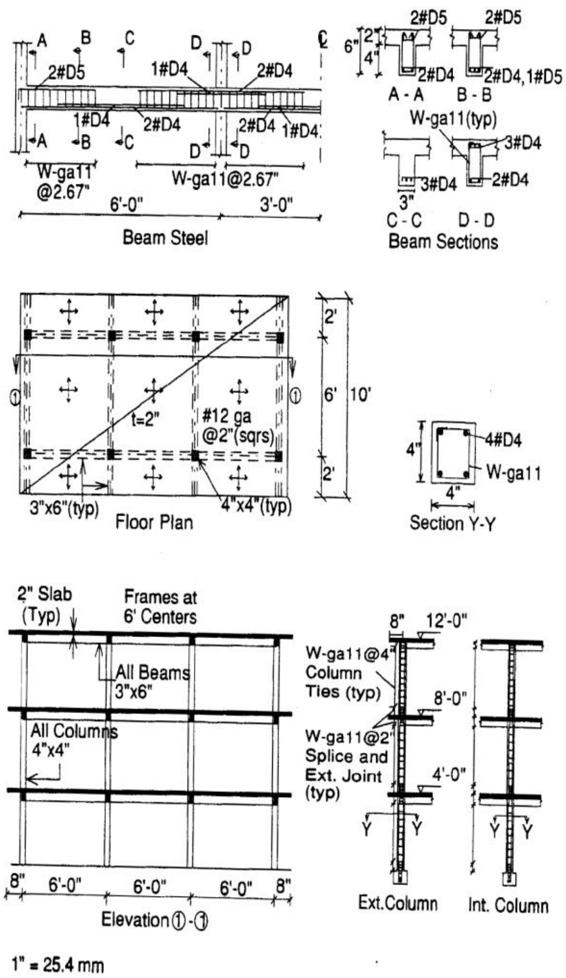

Figure 2. Dimensions and reinforcement arrangement of three storey frame (Bracci, et al., 1995)

4. Modelling and verification

The axial loads in columns are assumed to be constant during excitations and are shown in Table 3. Moment-rotations for every beams and columns are computed. Axial loads on columns are taken into account; however, the effect of confinement is ignored due to relatively large stirrup spacing. Figure 3 shows the model with nonlinear Link elements in SAP2000. The hysteretic behaviour of these nonlinear elements follow Takeda model (Takeda, Sozen, & Nielsen, 1970). The structural frequencies of the first three mode shapes are determined in Table 4 in comparison with the experimental results. They are very close in the first and second modes, but different in the third mode; however, the first mode is the most important.

Table 3
Axial load in columns

Storey	Axial load (kN)	
	External column	Internal column
1	30	60
2	20	40
3	10	20

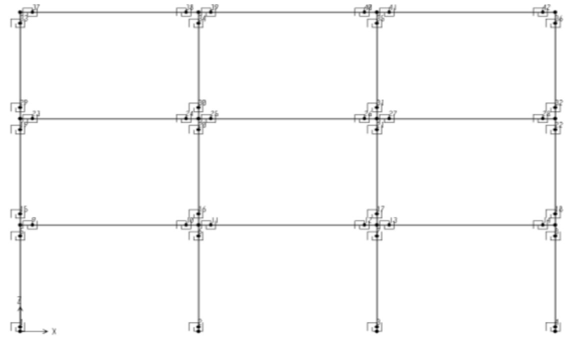

Figure 3. Modelling of the three-storey frame with nonlinear Link elements

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

5. Damage analyses and comparison

Inelastic time history analyses are performed for the frame subjected different seismic intensities. Thus, inter-storey drifts are obtained and plotted in Figure 4a, 5a, 6a for 0.05g, 0.20g and 0.30g, respectively. The results of inelastic time history analyses are used to compute the damage indices of the frame using the selected damage model to identify locate and quantify the damage imparted to the structure during earthquake. Figure 4b, 5b and 6b present the experimental damage states taken from Ref (Bracci, 1992) while Figure 4c, 5c and 6c show the analytical damage states for Taft PGAs of 0.05g, 0.20g and 0.30g, respectively. It should be noted that the analytical damage states are plotted for different damage index levels as described in Table 1. The damage states obtained from analyses are close to those obtained from experiment.

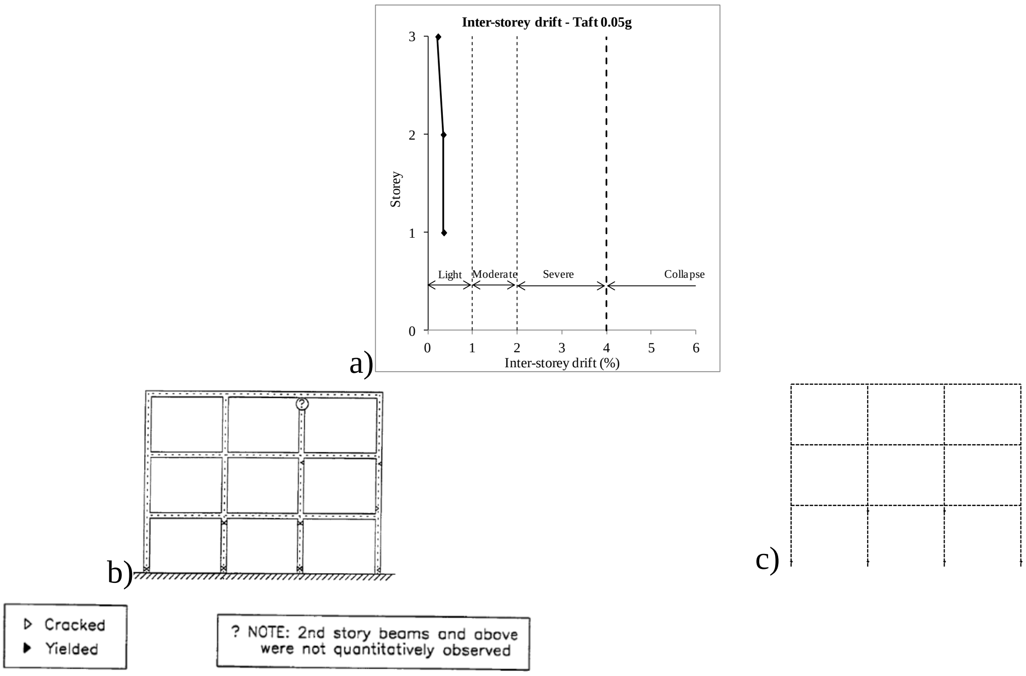

Figure 4. Damage state – Taft 0.05g

a) Inter-storey drift; b) Experiment (Bracci, 1992); c) Analysis.

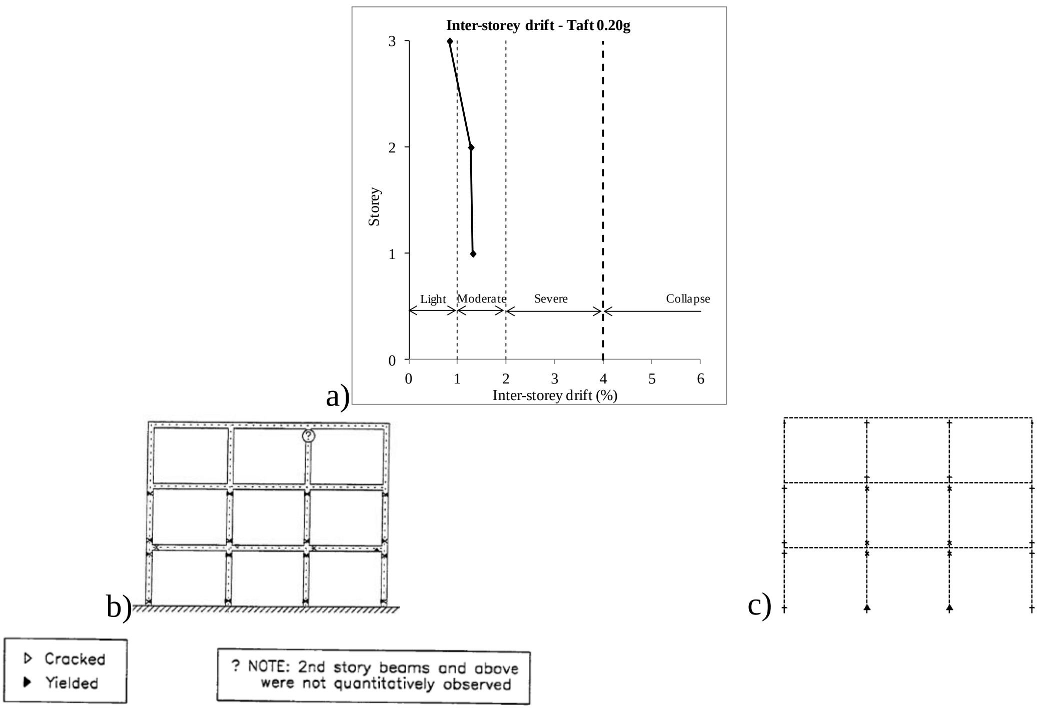

Figure 5. Damage state – Taft 0.20g

a) Inter-storey drift; b) Experiment (Bracci, 1992); c) Analysis.

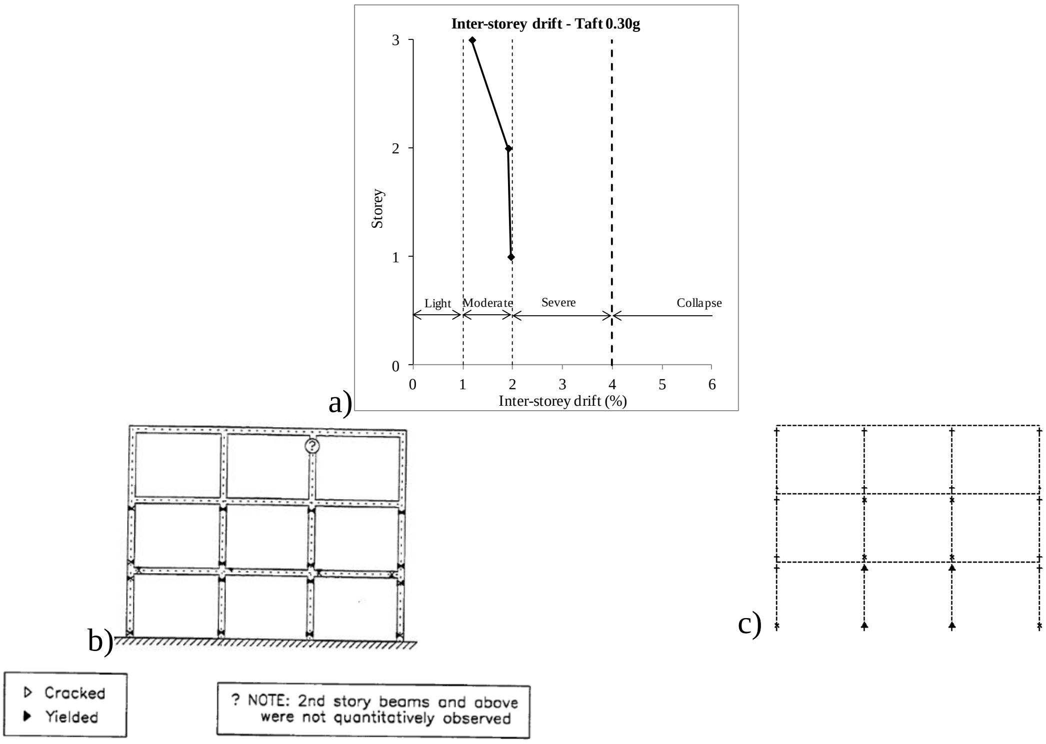

Figure 6. Damage state – Taft 0.30g
a) Inter-storey drift; b) Experiment (Bracci, 1992); c) Analysis.

For the Taft 0.05g, both the maximum DI of around 0.03 and the maximum inter-storey drift of 0.28% well represent the state of no damage. For the Taft 0.20g, the maximum DI of around 0.55 represents severe damage while the maximum inter-storey drift of 1.33% represents the state of moderate damage. It should be pointed out that the two lower ends of inner columns in the first storey have DIs exceeding 0.50. This state of damage is not captured well by the inter-storey drift which is more a measure for the whole storey. For the Taft 0.30g, both the maximum DI of around 0.68 and the maximum inter-storey drift exceeding 2%, well represent the state of severe damage. Figure 4b, 5b and 6b show that the damage distributed in the structure can be identified, located and quantified by the damage index. Damage index provides a clear picture and is closer to the experimental damage states (Figure 4b, 5b and 6b) than the inter-storey drift. Inter-storey drifts of storey 1 and 2 are almost similar, which inappropriately interpret the more severe damage in storey 1 comparing to storey 2 as shown in the experiment.

The above results demonstrate the limitations of inter-storey drift in damage assessment of structures, which can be explained as follows. Being an absolute maximum value throughout the seismic events experienced by a structure in its lifetime, inter-storey drift cannot adequately capture the cyclic fluctuating effects of the seismic loading. For instance, in an RC column subjected to constant displacement magnitude loading cycles on the top, the damage in i^{th} cycle is obviously larger than that in the previous one. However, the inter-storey drift remains unchanged, thus incorrectly describes the damage of the column. To overcome this shortcoming, many new design methods have recently been developed based on cumulative parameters such as energy (Surahman, 2007; Teran-Gilmore & Jirsa, 2007) and damage (Cruz & López, 2004; Moustafa, 2011; Prakash & Belarbi, 2010). In spite of the above mentioned limitation, inter-storey drift, with its dominant simplicity characteristic, is widely adopted in current seismic codes.

6. Conclusion

Inelastic time history and damage analyses of the previously tested 3-storey frame were performed for different seismic intensities. A comparison between the damage states of the frames based on inter-storey drift, experiment and damage analysis is conducted. It shows that drift cannot provide an insight into the damage states and damage distribution in the frames while the damage model is able to. Inter-storey drift is also found to be a very unreliable indicator of structural damage because it does not take into account a number of important parameters such as number of cycles, force, deformation, axial load, ductility, etc. Furthermore, inter-storey drift cannot capture the damage distributed in the critical zones such as plastic hinges in structures■

References

ASCE. (2000). Prestandard and commentary for the seismic rehabilitation of buildings *Prepared for Federal Emergency Management Agency, FEMA Publication No. 356*. Washington, D.C.: Federal Emergency Management Agency.

Banon, H., & Veneziano, D. (1982). Seismic safety of reinforced members and structures. *Earthquake Engineering & Structural Dynamics, 10*(2), 179-193.

Bozorgnia, Y., & Bertero, V. V. (2001). Evaluation of damage potential of recorded earthquake ground motion. *Seismological Research Letters, 72*(2), 233.

Bracci, J. M. (1992). *Experimental and analytical study of seismic damage and retrofit of lightly reinforced concrete structures in low seismicity zones.* Ph.D. Thesis, State University of New York at Buffalo.

Bracci, J. M., Reinhorn, A. M., & Mander, J. B. (1995). Seismic resistance of reinforced concrete frame structures designed for gravity loads: performance of structural system. *ACI Structural Journal, 92*(5), 597-608.

Cao, V. V., Ronagh, H., Ashraf, M., & Hassan, B. (2011). A new damage index for reinforced concrete structures subjected to seismic loads. *The First International Postgraduate Conference on Engineering, Designing and Developing the Built Environment for Sustainable Wellbeing -eddBE2011, The Queensland University of Technology, Queensland, Australia, 27-29 April*, 194-199.

Corte, G. D., Barecchia, E., & Mazzolani, F. M. (2006). Seismic upgrading of RC buildings by FRP: full-scale tests of a real structure. *Journal of Materials in Civil Engineering, 18*(5), 659–669. doi: 10.1061/(ASCE)0899-1561(2006)18:5(659)

Cosenza, E., Manfredi, G., & Ramasco, R. (1993). The use of damage functionals in earthquake engineering: A comparison between different methods. *Earthquake Engineering & Structural Dynamics, 22*(10), 855-868.

Cruz, M. F., & López, O. A. (2004). Design of reinforced concrete frames with damage control. *Engineering Structures, 26*(14), 2037-2045. doi: http://dx.doi.org/10.1016/j.engstruct.2004.01.002

Fajfar, P. (1992). Equivalent ductility factors, taking into account low-cycle fatigue. *Earthquake Engineering & Structural Dynamics, 21*, 837-848.

Fardis, M. N., Economu, S. N., Antoniou, A. N., Komodromos, P. J., & Sfakianakis, M. G. (1993). Damage measures and failure criteria - Part I, Contribution of University of Patras *Final Report of Cooperative research on the seismic response of reinforced concrete structures - 2nd Phase*.

Garcia, R., Hajirasouliha, I., & Pilakoutas, K. (2010). Seismic behaviour of deficient RC frames strengthened with CFRP composites. *Engineering Structures, 32*, 3075 - 3085.

Ghobarah, A., & Aly, N. M. (1998). Seismic reliability assessment of existing reinforced concrete buildings. *Journal of Earthquake Engineering, 2*(4), 569-592.

Kim, T.-H., Lee, K.-M., Chung, Y.-S., & Shin, H. M. (2005). Seismic damage assessment of reinforced concrete bridge columns. *Engineering Structures, 27*, 576–592.

Kunnath, S. K., Reinhorn, A. M., & Lobo, R. F. (1992). IDARC Version 3.0: A Program for the Inelastic Damage Analysis of Reinforced Concrete Structures. *Report No. NCEER-92-0022*, National Center for Earthquake Engineering Research, State University of New York at Buffalo.

Ludovico, M. D., Prota, A., Manfredi, G., & Cosenza, E. (2008). Seismic strengthening of an under-designed RC structure with FRP. *Earthquake Engineering and Structural Dynamics, 37*, 141–162.

Moustafa, A. (2011). Damage-based design earthquake loads for Single-Degree-Of-Freedom inelastic structures. *Journal of Structural Engineering, 137*(3), 456-467.

Park, Y.-J., & Ang, A. H.-S. (1985). Mechanistic seismic damage model for reinforced concrete. *Journal of Structural Engineering, 111*(4), 722-739.

Phan, L. T., Todd, D. R., & Lew, H. S. (1993). Strengthening methodology for lightly reinforced concrete frames-I. *Report NISTIR 5128*.

Pinto, A. V., Verzeletti, G., Molina, J., Varum, H., Pinho, R., & Coelho, E. (2002). Pseudo-Dynamic Tests on Non-Seismic Resisting RC Frames (Bare and Selective Retrofit). Ispra: Joint Research Centre.

Prakash, S. S., & Belarbi, A. (2010). Towards damage-based design approach for RC bridge columns under combined loadings using damage index models. *Journal of Earthquake Engineering, 14*(26), 363-389.

Rodriguez, M. E., & Padilla, D. (2009). A damage index for the seismic analysis of reinforced concrete members. *Journal of Earthquake Engineering, 13*(3), 364-383.

Sharma, A., Reddy, G. R., & Vaze, K. K. (2012). Shake table tests on a non-seismically detailed RC frame structure. *Structural Engineering and Mechanics, 41*(1), 1-24.

Surahman, A. (2007). Earthquake-resistant structural design through energy demand and capacity. *Earthquake Engineering & Structural Dynamics, 36*(14), 2099-2117. doi: 10.1002/eqe.718

Tabeshpour, M. R., Bakhshi, A., & Golafshani, A. A. (2004). Vulnerability and damage analyses of existing buildings. *13th World Conference on Earthquake Engineering, Paper No. 1261*.

Takeda, T., Sozen, M. A., & Nielsen, N. N. (1970). Reinforced concrete response to simulated earthquakes. *Journal of the Structural Division, 96*, 2557-2573.

Teran-Gilmore, A., & Jirsa, J. O. (2007). Energy demands for seismic design against low-cycle fatigue. *Earthquake Engineering & Structural Dynamics, 36*, 383–404.