The 11th Canadian Conference on Earthquake Engineering

Canadian Association for Earthquake Engineering

VISUAL SEISMIC DAMAGE STATES FOR REINFORCED CONCRETE FRAME-SHEAR WALL BUILDINGS

SAURABH SHIRADHONKAR
Graduate Student, Indian Institute of Technology Bombay, Mumbai, India
saurabh.coep@gmail.com

Ravi Sinha
Professor, Indian Institute of Technology Bombay, Mumbai, India
rsinha@civil.iitb.ac.in

ABSTRACT: The paper evaluates crack width based visual damage state definitions used for damage inspection of reinforced concrete shear wall and establishes relationship between visual damage states, crack widths and engineering parameters viz. curvature and drift ratio. Quasi-cyclic tests on flexure dominating reinforced concrete shear walls performed by Dazio et al. (2009) and Thomsen and Wallace (1995) are analytically simulated using spread plasticity models to establish the relationship between drift ratios and maximum crack widths. The relationships between observed maximum crack widths and drift ratios derived from Dazio et al., and observed damage states and drift ratios derived from Thomsen and Wallace study are used to examine crack width based visual damage states proposed by Ohkubo (1991), Sinha and Goyal (2007) and Anagnostopoulos and Moretti (2008). It is shown that crack width based damage definition given by Okhubo works reasonably well to capture slight to moderate damage states of shear walls. To define engineering limits for visual damage states in terms of curvature or strain limits, maximum crack widths for analytical simulations are estimated using widely used Gergely and Lutz (1968) crack width formula and compared against experimentally observed crack widths. It is also found that analytical estimation of crack widths matches fairly with the observed crack widths for values less than 2 mm. It is thus concluded that slight to moderate visual damage states of flexure dominated RC shear walls can be defined using crack width limits given by Ohkubo, while severe and collapse damage states are better represented by drift ratios.

1. Introduction

Accurate classification of damage in buildings is an important issue in the aftermath of an earthquake. Damage surveys carried out immediately after an earthquake aims to assess the safety and integrity of the damaged buildings. Visual damage states, with clear definition of damage and failure mechanisms, acts as a tool to differentiate between distinct levels of structural damage and classify buildings into occupiable and unoccupiable damage classes. The post-earthquake damage surveys are also used to evaluate the extent of damage in unoccupiable partially damaged buildings for making repairing or strengthening decisions. Hence, visual damage assessment methods addressing residual seismic capacity are of utmost importance to earthquake engineering community.

Damage assessment methods have traditionally focused on broad qualitative evaluation in order to characterize the size of earthquakes in terms of intensity measures such as MMI (1931), MSK (1964) and EMS-98 (1998). Visual inspection methods classifying damage on discrete scale have been also developed to evaluate safety of buildings for usability and retrofitting. Goretti and DiPasquale (2002) classified the post-earthquake survey methods based on their objective of survey and the time when the data are recollected. Typically, data collected in first few days from reconnaissance survey is used for macro-seismic seismic intensity assessment (such as MSK, EMS-98 etc.). On the other hand, the surveys conducted during the next few months are used to assess the damage and to evaluate the long-term use of building considering losses and financial implications. ATC-13 (1985) subdivided building damage classification used in MMI damage scale into repairable and non-repairable damage categories using damage probability matrices for 78 types of representative California buildings. This also provided the basis for transition from intensity-based measures (such as MMI, MSK-64), to post-earthquake safety evaluation measures in ATC-20 (1989) and performance measures in FEMA 273 (1997). Damage state definitions, developed on the basis of cost-ratio or damage factor, effectively link ground motion parameters to structural and non-structural damage and consequently to the cost of damage; which is useful in estimating economic losses.

Ohkubo (1991) introduced damage rank to classify structural damage based on post-earthquake visual damage survey. The damage rank classification uses observed crack-widths in structural members and weighs them to determine global damage state. Similarly, Sinha and Goyal (2004) proposed crack-width based damage state definitions from observations after Killari (1993), Jabalpur (1999) and Bhuj earthquakes (2001) in India. Anagnostopoulos and Moretti (2008), Tu et al. (2012) and Taskin et al. (2012) used data from past earthquake and expert opinion to propose crack-width based member definitions to develop post-earthquake damage assessment procedures for Greece, Taiwan and Turkey, respectively. These methods have proposed different set of rules to integrate member damage information for damage assessment of the entire building, but do not explicitly address reduction in structural capacity to provide a comprehensive post-earthquake safety assessment. The authors (Shiradhonkar and Sinha, 2014) have studied the significance of crack width based member damage levels using damage observed during experiments reported in the published literature. This study has shown that the crack width based visual damage states proposed by Ohkubo adequately captures slight to moderate damage states of reinforced concrete frame members.

Table 1 – Damage classification and crack width limits for shear walls

Damage description	Crack widths and damage states		
	Ohkubo (1991)	**Sinha and Goyal (2004)**	**Anagnostopoulos and Moretti (2008)**
Narrow cracks on surface of concrete	Rank I - crack-width less than 0.2 mm	State 1 - crack-width less than 0.1 mm	I – None –no sign of distress
Visible but narrow cracks on surface of concrete	Rank II - crack-width 0.2 to 1.0 mm	State 2 - crack-width 0.1 to 0.2 mm	II - Slight- crack-width less than 1.00 mm
Local crush of covered concrete, considerably big cracks	Rank III - crack-width 1.0 to 2.0 mm	State 3 - crack-width 0.5 to 3.0 mm	III - Moderate – Heavy-crack-width 1.0 to 3.0 mm
Remarkable crush of concrete with exposed rebars, cover concrete spalled off, diagonal cracks in core	Rank IV - crack-width greater than 2 mm	State 4 - crack-width greater than 3 mm	IV – Severe - crack-width greater than 3.0 mm
Rebars bent, core concrete crushed, visible vertical deformation of column/wall	**Rank V**	**State 5**	

Reinforced concrete shear walls are frequently used in RCC buildings to enhance the lateral stiffness and strength of medium to high-rise buildings. In dual systems, the shear wall resists most seismic demand and helps to delay seismic damage in the columns. Thus, shear wall acts as a principle lateral load-resisting member in frame-shear wall dual system. This paper evaluates correlation between visual damage state definitions, and crack widths using quasi-cyclic test results on flexure dominating reinforced concrete shear walls performed by Dazio et al. (2009). Nonlinear analyses are carried out to simulate the experimental response. The crack widths of the analytical simulations are evaluated using Gergely and Lutz (1968) crack width formulation. The main aim of this work is to verify slight, moderate and extensive damage states of shear wall used in post-earthquake condition assessment. The investigation also helps to verify a relationship between visual damage states and engineering parameters.

2. Damage State Definitions for Visual Damage Assessment

Ohkubo defined damage rank of members using crack widths to classify damage in shear walls. The damage rank of structural walls is defined in terms of total length of structural walls in particular damage states. Global damage state of the frame-shear wall building is obtained using the damage ratio (DR), calculated using summation of weightage assigned to damage class of individual member. On similar ground, Sinha and Goyal proposed the visual damage state definition for frame shear wall buildings, based on loss estimation study after Bhuj Earthquake (2001). Recently, Taskin et al. (2012) proposed a survey form to evaluate post-earthquake damage in reinforced concretes buildings in Turkey using Ohkubo damage rank definition. Anagnostopoulos and Moretti developed a post-earthquake emergency survey method for safety evaluation of damaged building, based on extensive experience from past earthquakes in Greece. The crack width limits for shear-wall damage states, proposed by different researchers are summarized in the Table 1.

3. Damage State Information from Experimental Studies

Relatively limited research results are available in published literature to evaluate deformations at intermediate damage states used in post-earthquake damage assessment. For RC columns, a variety of studies such as Berry and Eberhard (2004) and Jiang et al. (2010) have been carried out to investigate the damage behavior of RC members at different damage states. Pilakoutas and Elnashai (1995) and Krolicki et al. (2011) studied hysteretic response and individual damage states of ductile shear walls during entire loading process until failure. However, these studies did not correlate damage states in terms of visual damage parameters such as crack width. This section discusses relation between intermediate visual damage states and engineering parameters, and verification of visual damage state definitions using published experimental results.

Table 2 –Relationship between drifts and damage observations, Dazio et al. (2009).

	% Drift at initiation of spalling	% Drift at Significant spalling	Remarks
WSH1	0.68	-	Poor ductility of longitudinal reinforcement
WSH2	0.68	-	Poor ductility of longitudinal reinforcement
WSH3	1.02	1.71	Ductile shear wall
WSH4	1.02	1.71	Ductile shear wall
WSH5	0.55	1.35	Poor ductility of web reinforcement
WSH6	1.02	1.71	Ductile shear wall

Dazio et al. (2009) presented results of quasi-cyclic tests on six reinforced concrete walls performed at ETH Zurich. The experimental program was conducted to investigate the effect of percentage of vertical reinforcement and reinforcement ductility properties on the deformation behavior of slender RC walls. The six wall half-sized specimens, simulating the lower half of a wall of a 6-storey prototype building were designed according to NZS 3101 for different ductility classes defined in Eurocode 8. All six specimens were tested under quasi-cyclic loading. Reinforcement details, stress-strain relationship of reinforcing bars, local and global force-deformation data and maximum crack widths at each loading points are made available online. Table 2 summarizes the relationship between drifts and damage observations presented by Dazio et al.

Table 3 – Relationship between drifts and damage observations, Thomsen and Wallace (1995).

% Drift	Observations
0.50	flexural cracks at the bottom
0.75	first yielding of longitudinal reinforcement
1.00	vertical splitting and minor crushing (initiation of spalling)
1.50	concrete in boundary element crack extensively, (initiation of significant spalling)
2.00	crushing and significant spalling, bar buckling
3.00	complete failure of shear wall

Similarly, Thomsen and Wallace (1995) tested slender rectangular and T-shaped shear walls, to evaluate the effectiveness of using a displacement based design procedure. Thomsen and Wallace tested two ductile rectangular shear walls, with same percentage of longitudinal and shear reinforcements but different hoop spacing, under quasi-cyclic loading. Table 3 summerises relationship between damage states and drift ratios established from Thomsen and Wallace.

From tables 2 and 3, it is concluded that 1% drift value indicates initiation of spalling and drift value around 1.5% indicates initiation of significant spalling for ductile slender shear walls. Initiation of cover concrete spalling occurs in the next stage of damage reduces shear resistance of concrete due to destruction of bond between tensile reinforcement and surrounding concrete. The destruction of bond compromises effectiveness of dowel action. Which consequently leads to reduction in strength and stiffness under repeated lateral loading (Penelis and Kappos, 1997). Even though sufficient flexural deformation capacity against collapse may be available at this damage stage, a detailed engineering evaluation is required for determining suitable repairing techniques. Therefore, initiation of spalling of cover concrete represents moderate damage to the member. On the other hand, significant spalling leads to severe reduction in shear resistance under cyclic loading due to grinding and gradual smoothening of crack interface, which eventually leads to buckling of compression longitudinal reinforcement bars. Therefore, significant spalling of cover concrete represents initial stage of severe damage state, and crushing of core and buckling of reinforcement bars indicates extensive damage states of the member.

Once the concrete has spalled off it will remain that way and therefore crack width limits are not required to identify moderate, severe and collapse damage states. However to identify slight, light and moderate damage states, crack width based visual damage states have been verified by comparing measured crack widths against the drift ratios.

3.1. Relationship Between Maximum Crack Width and Maximum Drift

Maximum flexural crack widths measured during experimentation are plotted against the maximum drift in the figure 1. It can be observed in that there exists a nearly linear relationship between maximum crack width and maximum drift experienced by the shear wall. Mean ± std. dev. lines are also marked in the figure 1.

Drift ratio of 0.75 % indicates yielding of longitudinal reinforcement and classified as light damage state. From figure 1, the mean value of maximum crack widths for drifts 0.5% 0.75% and 1% are 0.78, 1.16 and 1.53 mm, respectively. Ohkubo defined crack width limits as 0.2 to 1 mm and 1 to 2 mm for light and moderate damage states, respectively. Therefore, crack width definition given by Okhubo works reasonably well to capture slight, light and moderate damage states of the flexural dominant slender shear walls. The mean drift ratios for crack widths 0.1, 0.2 and 0.5 mm crack widths are 0.25, 0.30 and 0.50, respectively. Thus, the crack widths limits proposed by Sinha-Goyal are found conservative for slight, light and moderate damage states. On the other hand, crack widths limits proposed by Anagnostopoulos and Moretti are found non-conservative for moderate and heavy damage states.

4. Simulation of Dazio et al. Experimental Data

Responses of experiments presented in Dazio et al. are simulated in IDARC-2D version 7.0. Stress-strain relations of confined concrete are calculated separately for boundary and web portion using confinement model proposed by Park et al. (1982) and EC-8 (2004), based on the arrangement of transverse reinforcement in the section. Trilinear moment curvature relationship has been used to simulate the experimental response. In the calculation of moment curvature relationship it is assumed that, plane section remains plane during bending. The initial slope of the moment-curvature relationship has been adjusted to match the initial slope of analytical force-displacement response with the experimental force-displacement response. The inelastic behavior of components is monitored using a new non- symmetric tri-linear hysteretic model, proposed by Park et al. (1987). Different values of hysteretic modeling parameters are tried during simulation to match the hysteretic properties of experimental force deflection relationship. Shear-deformation characteristics of the shear wall are modelled using concentrated shear hinge. The nonlinear shear-deformation properties of the shear hinge are estimated as per Krolicki et al. (2011). Table 4 compares first yield and nominal yield displacements observed during the experiments with those estimated from the simulations. From this table it is observed that, both yield displacements from simulated responses are closely related with those observed during experimental studies.

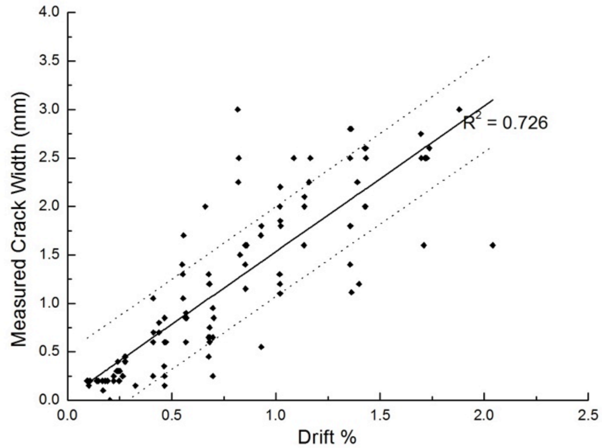


Fig. 1 – Comparison of Measured Crack Width against Maximum Drift

Table 4 – Comparison of simulated and measured yield displacements for Dazio et al. (2009).

	At first yield of outer longitudinal reinforcement (mm)		**Nominal yield displacement based on strain limits according to Priestley et al. (2007) (mm)**	
	Simulated	**Measured**	**Simulated**	**Measured**
WSH1	8.06	8.40	12.31	11.00
WSH2	9.98	7.80	12.68	10.50
WSH3	11.24	11.30	17.60	16.50
WSH4	11.18	11.40	16.61	15.50
WSH5	7.89	7.80	10.57	9.30
WSH6	10.35	9.90	15.74	12.70

Figures 2(a-c) compare experimental and simulated force deflection relationship for WSH2, WSH3 and WSH 6, respectively. A reasonable match is observed between experimental and simulated force deflection relationship from figures 2(a-c).

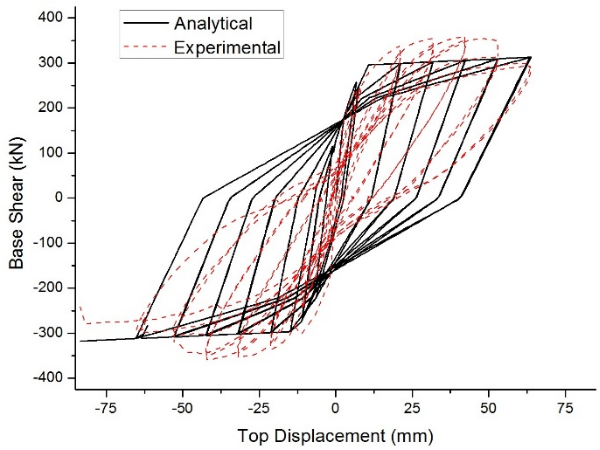
4.1. Estimation of Maximum Crack Widths

In order to determine the damage states of analytical model as per Ohkubo definition, the maximum crack width in the member has been estimated using Gergely and Lutz (1968) formulation. This formulation is based on a regression analysis of a large number of tests from different sources. The crack-width for given moment is given by,

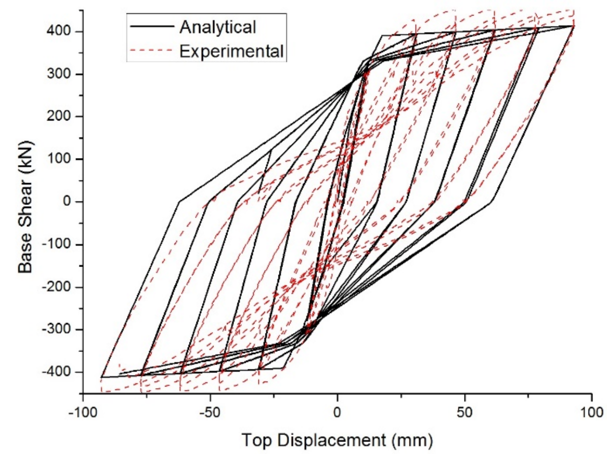
$$W_{\max} = 0.076\beta f_s \sqrt[3]{d_c A_e} \times 10^{-3} \text{ inches} \quad (1)$$

where β is ratio of distance between neutral axis and tension face to distance between neutral axis and centroid of reinforcing steel, f_s is stress in steel, d_c is distance from center of bar to extreme tension fiber (in inches) and A_e is the effective stretched concrete area (in^2). This formulation is valid up to yield point of steel. The same expression is used to evaluate the crack widths corresponding to post-yield moments, as the maximum crack width of the member is directly proportional to the strain in the steel once the minimum crack spacing is reached (Forsch, 1999).

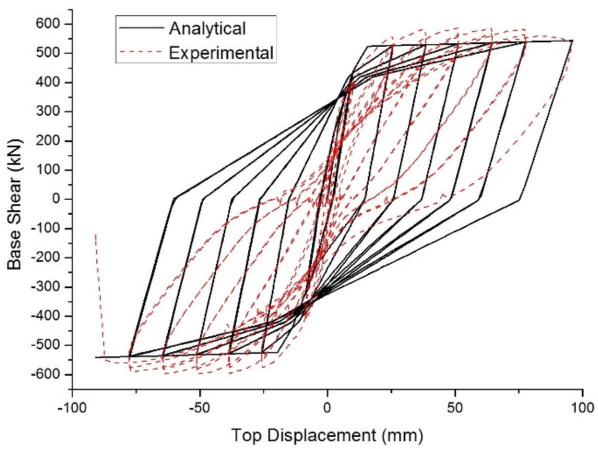
The authors (Shiradhonkar and Sinha, 2014) used extended Gergely and Lutz crack widths formula to estimate curvature ductilities for the crack widths based visual damage states of the frame members. They found that mean curvature ductility at the observed initiation of spalling is same as the mean curvature ductility corresponding to 2 mm crack width. Thus, curvature ductility corresponding to 2 mm crack width represents moderate damage state of the member. To define engineering limits for visual damage states in terms of curvature, maximum crack widths for analytical simulations are estimated using widely used Gergely and Lutz crack width formula.



a. WSH2

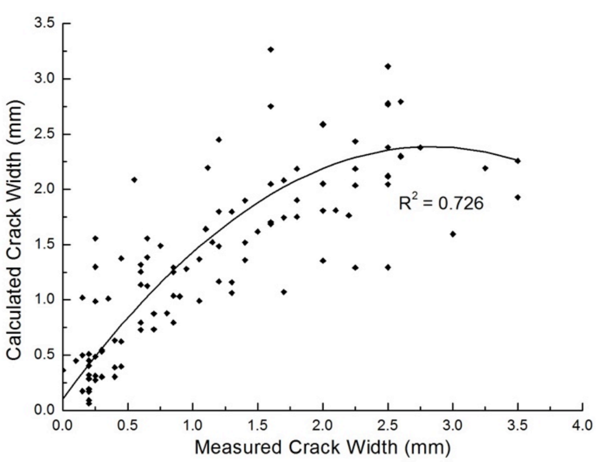


b. WSH3

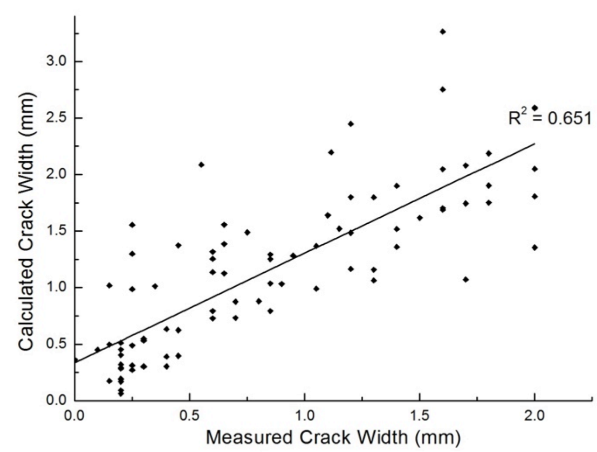


c. WSH6

Fig. 2 – Comparison of Experimental and Simulated Force Deflection Relationship



a. All data



b. Upto crack width of 2 mm

Fig. 3 – Comparison of Calculated and Measured Crack Width

4.2. Comparison of Estimated and Measured Maximum Crack Width

Maximum curvatures corresponding to the peak drift values in the experiments are estimated using simulated response. For each curvature value, analytical crack width is calculated using Gergely and Lutz formula. Figure 3 compares crack widths estimated using Gergely and Lutz formula against measured crack widths at peak drifts.

From figure 3a it is observed that, for complete dataset analytical estimation of crack widths differs significantly with the observed crack widths. However, analytical estimation of crack widths varies almost linearly with the observed crack widths, for values less than 2 mm, as shown in the Figure 3b. Maximum drift corresponding to 2 mm crack width determined from figure 1 is 1.35 %, which indicates extreme end of moderate damage state between initiation of spalling and significant spalling. Therefore Gergely and Lutz crack width formula fairly predicts crack widths to determine slight to moderate damage states in analytical simulations of flexural dominant slender shear wall.

5. Conclusions

The paper evaluates the crack width limits used in the visual damage state definitions using experimental data presented by Dazio et al. (2009) and Thomsen and Wallace (1995). The maximum crack widths in the analytical simulations are estimated using widely used Gergely and Lutz (1968) crack width formula and compared against observed crack widths in order to evaluate visual damage states of the analytical simulations. It has been found that Gergely and Lutz crack width formula is able to predict the crack widths in slender shear walls to determine slight to moderate damage states in analytical simulations. From the relationships developed between the observed maximum crack widths and drift ratios and the observed damage states and drift ratios, it has been found that crack width definition given by Ohkubo is able to capture slight to moderate damage states of the member. It is also seen that the Ohkubo damage state definitions work reasonably well for slender reinforced concrete shear wall, where damage is predominantly due to flexural cracking. The Sinha-Goyal damage state definition is found to be conservative, while Anagnostopoulos and Moretti definition is found to be non-conservative to classify moderate damage in slender shear walls.

6. References

ANAGNOSTOPOULOS, Stavros S, MORETTI Marina, “Post-Earthquake Emergency Assessment of Building Damage, Safety and Usability—Part 1: Technical Issues”, *Soil Dynamics and Earthquake Engineering*, Vol. 28, No. 3, 2008, pp. 223–232.

APPLIED TECHNOLOGY COUNCIL (ATC-13), *Earthquake Damage Evaluation Data for California,* Technical report, Redwood City, California, USA, 1985.

APPLIED TECHNOLOGY COUNCIL (ATC-20), *Procedures for Post-Earthquake Safety Evaluation of Buildings,* Technical report, Redwood City, California, USA, 1985.

BERRY, Michael, EBERHARD, Marc, *Performance Models for Flexural Damage in Reinforced Concrete Columns*, Pacific Earthquake Engineering Research Center, College of Engineering, University of California, Berkeley, USA, 2004.

DAZIO, Alessandro, BEYER, Katrin, BACHMANN, Hugo, "Quasi-Static Cyclic Tests and Plastic Hinge Analysis of RC Structural Walls", *Engineering Structures*, Vol. 31, No. 7, 2009, pp. 1556-1571.

EUROCODE 8, *Design of Structures for Earthquake Resistance—Part 1: General Rules, Seismic Actions and Rules for Buildings*, EN, 1, 2004.

FEDERAL EMERGENCY MANAGEMENT AGENCY (FEMA 273), *NEHRP Guidelines for The Seismic Rehabilitation of Buildings*, Technical report, Washington DC, USA, 1997.

FROSCH, Robert, "Another Look at Cracking and Crack Control in Reinforced Concrete", *ACI Structural Journal*, Vol. 96, No. 3, 1999, pp. 437-442.

GERGELY, Peter, LUTZ Lutz, “Maximum Crack Width in Reinforced Concrete Flexural Members - Causes, Mechanism, and Control of Cracking in Concrete”, *American Concrete Institute*, Vol. SP-20, 1968, pp. 87-117.

GORETTI, Agostino, DI PASQUALE, Giacomo, “An Overview of Post-Earthquake Damage Assessment In Italy”, *EERI Invitational Workshop an Action Plan to Develop Earthquake Damage and Loss Data Protocols,* 19-20, 2002.

GRUNTHAL, Gottfried, *European Macroseismic Scale 1998*, European Seismological Commission, Luxembourg, 1998.

JIANG, Huanjun, LU, Xilin, KUBO, Tetsuo, “Damage Displacement Estimation of Flexure-Dominant RC Columns” *Advances in Structural Engineering,* Vol. 13, No. 2, 2010, pp. 357-368.

KROLICKI, J, MAFFEI, J, CALVI, Gian, "Shear Strength of Reinforced Concrete Walls Subjected To Cyclic Loading," *Journal of Earthquake Engineering*, Vol. 15, No. S1, 2011, pp. 30-71.

MEDVEDEV, Sergeĭ V, SPONHEUER, Wilhelm, KARNIK, V, “Seismic Intensity Scale MSK 1964”, *Akad. Nauk SSSR, Geofiz. Kom.*, Moscow, 1965.

OHKUBO, Masamichi, *Current Japanese System on Seismic Capacity And Retrofit Techniques for Existing Reinforced Concrete Buildings and Post-Earthquake Damage Inspection and Restoration Techniques,* Technical report, SSRP-91/02, Department of Applied Mechanics and Engineering Sciences, University of California, San Diego, 1989.

PARK, Robert, PRIESTLEY, Michael, GILL, Wayne, “Ductility of Square-Confined Concrete Columns”, *Journal of Structural Engineering, ASCE,* Vol. 108, No. 4, 1982, pp. 929-950.

PARK, Young, REINHORN, Andrei, KUNNATH, Sashi, “IDARC: Inelastic Damage Analysis of RC Frame-Shear Wall Structures”, National Center for Earthquake Engineering Research, State University of New York, Buffalo NY, 1987.

PENELIS, George, KAPPOS, Andreas, *Earthquake resistant concrete structures*, E & FN Spon, 1997.

PILAKOUTAS, Kypros, ELNASHAI, Amr, "Cyclic Behavior of Reinforced Concrete Cantilever Walls I: Experimental Results", *ACI Structural Journal*, Vol. 92, No. 3, 1995, pp. 271-281.

SHIRADHONKAR, Saurabh, SINHA, Ravi, “Validation of Seismic Damage States for Reinforced Concrete Buildings”, *Proceedings of 10th US National Conference on Earthquake Engineering*, Anchorage, Alaska, 2014

SINHA, Ravi, GOYAL, Alok, “A National Policy of Seismic Vulnerability Assessment of Buildings and Procedure for Rapid Visual Screening of Buildings for Potential Seismic Vulnerability”, *Technical report*, Department of Civil Engineering, IIT Bombay, 2004.

TASKIN, B, GULER, K, TUGSAL, U, M, GENCOGLU, M, CELIK, M, HASGUR, Z, AYDOGAN, M, SAYGUN, A, I, “A Novel Post-Earthquake Damage Survey Sheet: Part I-RC Buildings”, *Proceedings of 15th World Conference on Earthquake Engineering*, Lisbon, Portugal, 2012.

THOMSEN, John, WALLACE, John, *Displacement based Design of Reinforced Concrete Structural Walls: An Experimental Investigation of Walls with Rectangular and T-Shaped Cross-Sections*, Dissertation, Clarkson University, 1995.

Tu, Y, Ao, L, Jean, W, “Study on the Earthquake Damage Evaluation Procedure for RC and Confined Masonry Buildings”, *Proceedings of 15th World Conference on Earthquake Engineering*, Lisbon, Portugal, 2012.

WOOD, Harry O, NEUMANN, Frank, “Modified Mercalli Intensity Scale of 1931”, *Bulletin of the Seismological Society of America,* Vol. 21, No. 4, 1931, pp. 277-283.