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ABSTRACT

In this paper, the cyclic response of non-ductile reinforced concrete (RC) columns is analytically investigated using a finite element (FE) software. The specimens selected, represent the typical long, medium and short non-ductile RC columns that are mostly found in the buildings of Thailand. These RC columns were tested previously and their experimental results are used here to compare with those of the FE software. Comparisons are made between analytical and experimental results considering the default material constitutive models and behavior mechanisms of FE software to assess its accuracy in predicting the actual response of these specimens. It was found that the FE software can predict the strength, deformation response and failure mode of reinforced concrete columns with good accuracy.

1. Introduction

RC columns when subjected to seismic forces perform differently depending on many factors such as their aspect ratio, longitudinal and transverse reinforcement ratios, loading conditions, etc,. Lack of sufficient transverse reinforcement imparts non-ductile behavior to the RC columns. These columns can sustain wind and gravity loads but fail in a non-ductile behavior when subjected to lateral cyclic loads. At present, in the regions of low to moderate seismicity such as Thailand, there are many buildings that are designed to the old codes considering only the wind and gravity loads [1]. Majority of the RC columns that are found in these buildings have insufficient shear reinforcement and during an earthquake it may lead to the buckling of longitudinal reinforcement or even brittle shear failure in the plastic hinge region [2]. As a result, the lateral strength of existing RC columns could significantly drop soon after the peak load is reached.

At present, the performance-based or displacement-based design approaches are preferred by many seismic design provisions to consider the complete load-deformation response of structures or their components [3]. Contrary to the large ductility requirements of the current seismic codes, the non-ductile RC columns continue to fail in a brittle way. Hence, it is found necessary to investigate the seismic performance of these non-ductile RC columns according to concepts of performance-based approaches. One of these approaches is the smeared crack and average stress-strain concept. The smeared crack approach can be subdivided into fixed smeared crack and rotating smeared crack approaches. In fixed smeared crack approach, the direction of a crack is considered geometrically fixed while in smeared rotating crack approach the direction of crack follows the principal stress and strain vectors. Previous studies showed that both the smeared crack approaches could predict the response of RC elements, however, the post-peak response predicted by smeared rotating crack approach was particularly more reliable [4]. To assess the complete load-deformation response of non-ductile RC columns of this study, the use of smeared rotating crack approach is preferred over the fixed crack approach.

VecTor2, which is a non-linear finite element analysis program, is based on the smeared rotating crack approach. Although, the FE programs usually require more efforts and computer efficiency for modeling and execution of analysis but detailed output information regarding stress and strain distribution, concrete crack pattern, reinforcement buckling etc., make it useful for researchers to investigate in detail the behavior of RC columns. To achieve the objective mentioned above, FE program VecTor2 was used for the modeling and analysis of RC columns. The specimens selected, represent the typical non-ductile RC columns that are mostly found in buildings of Thailand. These columns were tested previously and their experimental results are used here to compare with those of the finite element program VecTor2 [7].

2. The VecTor2 program

VecTor2 is a non-linear FE program developed at the University of Toronto for nonlinear finite element analysis of RC structures. It is based on Disturbed Stress Field Model (DSFM) which is a refinement of Modified Compression Field Theory (MCFT) [8-9]. Vector2 considers smeared rotating crack concept which is based on the displacement-based analytical approach [4]. It also considers the second order effects such as compression softening, tension stiffening, tension softening and tension splitting. It can model concrete expansion and confinement, cyclic loading and hysteresis response, bond slip, crack shear slip deformations, reinforcement buckling and crack allocation processes. It uses a fine mesh of low-powered elements for its models that are computationally efficient and numerically stable. These elements include a three-node triangle, a four-node rectangular and a four-node quadrilateral element for modeling concrete with smeared reinforcement. For discrete reinforcement, a two-node truss-bar element is used and for modeling bond-slip mechanisms a two-node link and a four-node contact element is used [10]. The finite element model is constructed in FormWorks,

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a pre-processor software that generates input files for VecTor2, and the results are visualized in Augustus program which is a post-processor.

3. Details of column specimen

Three column specimens, tested previously, representing the typical short, medium and long non-ductile columns of Thailand are selected for this study [7]. Cross-sectional details of column specimens are shown in Figure 1.

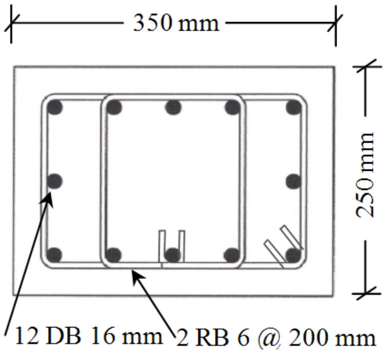

Figure 1 cross-sectional detail of column specimens [7]

Column specimens were tested under a combination of constant axial compressive load to represent the gravity loads, and incremental lateral reverse cyclic load. Details and material properties of column specimens are given in Table 1.

Table 1 Details and material properties of specimens

Specimen	S1L	S2M	S3S
B (mm)	350	350	350
H (mm)	250	250	250
L (mm)	2050	1570	1100
α	5.85	4.50	3.15
S_h (mm)	200	200	200
f'_c (MPa)	30	33	28
ρ_l (%)	2.756	2.756	2.756
ρ_t (%)	0.226	0.226	0.226
f_{yl} (MPa)	547	547	547
f_{yt} (MPa)	396	396	396
P (kN)	400	400	400

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B = width of the section, H = depth of the section, L = shear span, α = aspect ratio, S_h = hoop spacing, f'_c = concrete compressive strength, ρ_l = longitudinal reinforcement ratio, ρ_t = transverse reinforcement ratio, f_{yl} = longitudinal reinforcement yield stress, f_{yt} = transverse reinforcement yield stress, P = axial load.

A constant axial load of 400 kN was applied to all specimens using a 600 kN capacity jack. The lateral load was applied by a 250 kN capacity hydraulic actuator in a displacement-controlled manner. Specimens were subjected to two cycles of incremental percent drifts of ±0.25, ±0.50, ±0.75, ±1.0, ±1.5, ±2.0, ±2.5, ±3.0, etc. and so on until the failure of specimens. All the columns were designed with identical cross-section and material properties except their concrete compressive strengths as given in Table 1. Three different aspect ratios were used to simulate different failure modes: flexure, shear-flexure and shear failures that are associated to long, medium and short columns, respectively.

Experimental results showed that the failure of long specimen S1L was a flexural failure. The maximum load was reached at about 2% lateral drift, while buckling of the longitudinal reinforcement occurred at 3.5% drift. Finally, the gravity load collapse was occurred at 4.5% for this specimen. In medium height column S2M, shear cracks were observed along the height of specimen and resulted in the rapid degradation of lateral strength at 3.5% drift. Buckling of longitudinal reinforcement and slip along the shear plane lead to flexure-shear failure. Specimen S3S failed at 2.7% drift in a shear mode with the formation of a single large shear crack.

4. FE modeling of RC column specimens

Procedure for generating the FE mesh for column specimen S1L is described here as all the three specimens have same sectional and material properties. The finite element mesh was constructed in the FormWorks which is a pre-processor to VecTor2. The finite element mesh is presented in Figure 2.

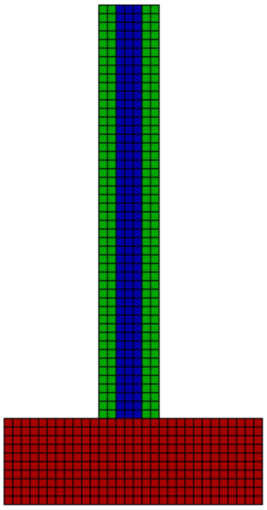

Figure 2 Finite element mesh of S1L

In the experimental setup, the column footing was fixed to the strong floor and no sliding was observed during the test. To achieve this situation, all the nodes at the bottom of the footing were assigned pinned supports. During analysis, the footing remained elastic and no considerable damage was observed.

For reinforced concrete, plane stress rectangular elements were used for modeling. Longitudinal and transverse reinforcements were modeled as smeared reinforcement. The sizes of the concrete elements were selected such that they represent the widths of column section over which the longitudinal reinforcements were smeared. The shaded area in Figure 3 represents the two ends of section where the area of three longitudinal bars was smeared over the section. Within the central portion of the column section, area of six longitudinal bars was smeared over the entire un-shaded portion. Considering the concrete cover and longitudinal bar spacing, the width of the shaded area was assumed to be 100 mm. Three types of concrete were used for each specimen. To make the footing rigid and elastic, high concrete compressive strength and reinforcement ratios were assigned to it. The properties of these types of concrete are given in Table 2.

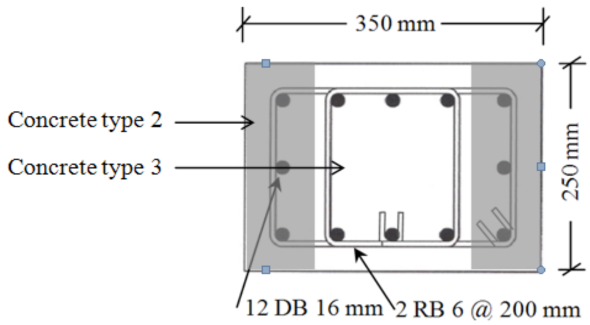

Figure 3 Concrete types in column section

Concrete type 1 was assigned to the footing of specimens. Elastic behavior was considered for the concrete elements because no cracks were observed during the testing. Concrete type 2 and 3 were assigned to the corner and inner concrete of the column specimen as shown in Figure 2, respectively. Confinement caused by the closed ties was provided by introducing out-of-plane reinforcement in Concrete type 2.

In VecTor2, the reinforcement can be modeled either using smeared or discrete reinforcement. In this study, only the smeared reinforcement was used to model both the longitudinal and transverse reinforcements.

In experiment, a constant axial compressive load of 400 kN was applied uniformly on the top of the column. This situation was achieved by applying 50 kN of vertical downward point load on each eight nodes at the top of the column as shown in Figure 3(a). Lateral loading in the form of displacement was applied at a height of 2050 mm for specimen S1L. Reverse cyclic loading was assigned to each specimen with 1 mm increments, as shown in Figure 3(b).

5. Concrete and reinforcement analytical models

Selection of appropriate analytical models for materials and behavior mechanism is very important for the computation of actual response. It is suggested to choose the default analytical models unless the use of some other models is justified [10]. The models selected for concrete elements, reinforcement elements and concrete bond are given in Table 3. Analytical models with asterisks were not the default models of VecTor2.

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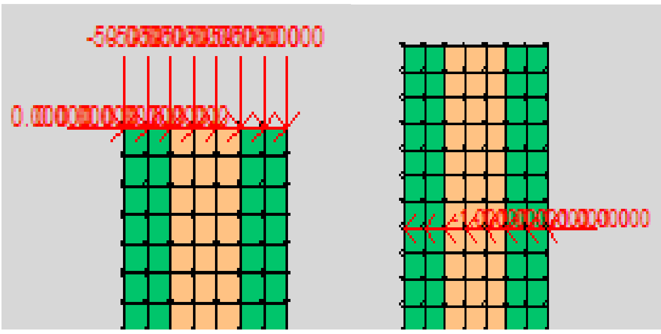
Table 2 Concrete element type

Concrete type	Location	Thickness (mm)	Reinforcement			
			Size (mm)	Direction	Ratio	Type
1	Footing	1500	DB 16 DB 16	90° 0°	2% 2%	Smeared
2	Column (Corner)	250	DB 16 DB 6 DB 6	90° 0° Out-of-plane	2.4% 0.11% 0.14%	Smeared Smeared Smeared
3	Column (Inner)	250	DB 16 DB 6	90° 0°	1.071% 0.226%	Smeared Smeared



(a) Axial loading (b) Lateral loading

Figure 3 Column axial and lateral loading

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Table 3 Analytical models used in FE modeling

Material property	Model
Concrete Models	
Compression pre-peak response	Hognested Parabola
Compression post-peak response	Modified Park Kent
Compression softening	Vecchio 1992-A (e1/e2-Form)
Tension stiffening	Modified Bentz 2003
Tension softening	Linear
FRC Tension	Not considered
Confined strength	Kupfer/Richard Model
Dilation	Variable-Kupfer
Cracking criterion	Mohr-Coulomb (Stress)
Crack stress calculation	Basic (DSFM/MCFT)
Crack slip check	Vecchio-Lai (Cyclic)*
Crack width check	Agg/2.5 Max. crack width
Hysteretic response	Palermo 2002 (w/Decay)*
Reinforcement Models	
Hysteretic response	Seckin Model (Bauschinger)
Dowel action	Tassios Model (Crack slip)
Buckling	Refined-Dhakal Maekawa
Bond models	
Concrete bond	Eligehausen

*non-default model

The details of all material and behavior models mentioned above are available in Manual of VecTor2 [10]. The crack slip model proposed by Vecchio and Lai was used instead of the default one to better consider the effect of reverse cyclic loading [11]. The effects of strength decay after the peak strength was reached were observed more accurately with hysteretic response model of Palermo 2002 (w/Decay) [12].

6. Discussion

Comparison of experimental and analytical load-displacement response for column specimen S1L is given in Figure 4. In experiment, the buckling of the longitudinal reinforcement was observed at a lateral drift of 3% and after that the complete collapse was occurred at 4.5% drift. The final failure mode of the column specimen S1L was a flexural failure.

The results obtained from the VecTor2 were in good accordance with the experimental results. The peak lateral load calculated by the analytical work was 89.1 kN which is close to the experimental value of 86.83 kN. Buckling of the

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longitudinal reinforcement was also captured with good accuracy at 3% lateral drift. Overall, the assessment of the hysteretic response of column specimen S1L was reliable up to 4% drift. After that, due to the excessive damage in the plastic hinge zone the solution did not converged.

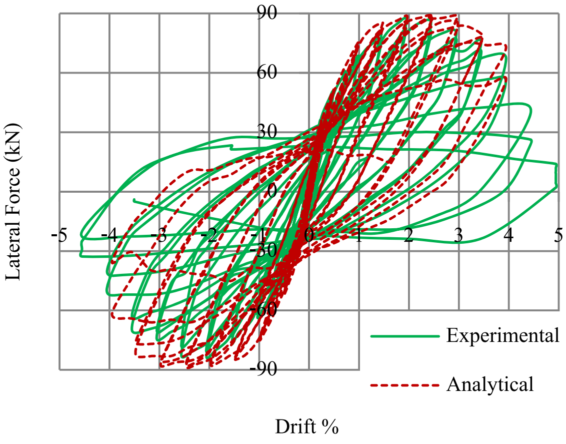
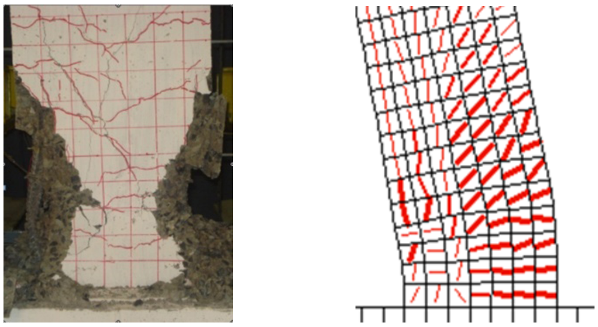


Figure 4 Load-displacement response of S1L

Figure 5 shows the comparison between the failure mode of experimental and analytical works. It was observed that the flexural failure that was concentrated in the plastic hinge zone was successfully captured by the analytical work.



(a) Experimental (b) Analytical

Figure 5 Comparison of final failure mode of specimen S1L [7]

For column specimen S2M, shear cracks were found distributed along the length of specimen and the lateral strength was degraded at a faster rate as compared to the specimen S1L. The axial load carrying capacity was sudden lost at lateral drift of 3.5% due to the buckling of compression reinforcement and slip along the shear plane as shown in Figure 6. The peak lateral load that was observed in experiment was 121.44 kN at the lateral drift of 2.5% during the reverse half

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cycle. In comparison, the analytical peak lateral load was found to be 118 kN at the same lateral drift during the forward half cycle. The buckling of compression reinforcement and the loss of axial load carrying capacity was also observed analytically with good accuracy. A comparison of final failure modes of experimental and analytical works is shown in Figure 7. The formation of flexural cracks in the plastic hinge zone and the shear cracks along the height of column is validating the capability of VecTor2 in assessing the post peak response up to failure with good accuracy.

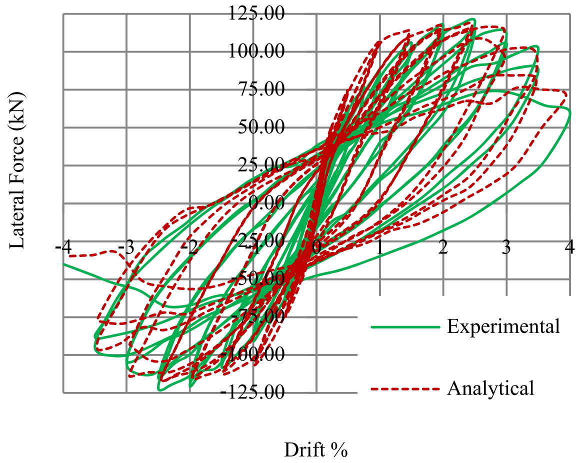

Figure 6 Load-displacement response of S2M

The failure mode for column specimen S3S, in experiment, was a shear failure with the formation of a large shear crack. The sudden drop in lateral strength was observed due to the reduction of force transferred along the shear crack as shown in Figure 8. The specimen was capable of carrying axial load up to 2.7% lateral drift. In analytical study, the peak lateral load was calculated to be 160.4 kN which is very close to the experimental value of 162.75 kN. The sudden drop in the lateral strength during the forward half cycle at 1.5% drift was not assessed precisely, as the degradation was observed analytically at about 2.5% drift. The final shear failure mode observed by the analytical study was not showing the formation of a large shear crack, but numerous shear cracks could be observed on the shear face as shown in Figure 9. Overall, the peak lateral strength and the maximum lateral drift corresponding to the gravity load capacity failure are in good accuracy with the experimental values.

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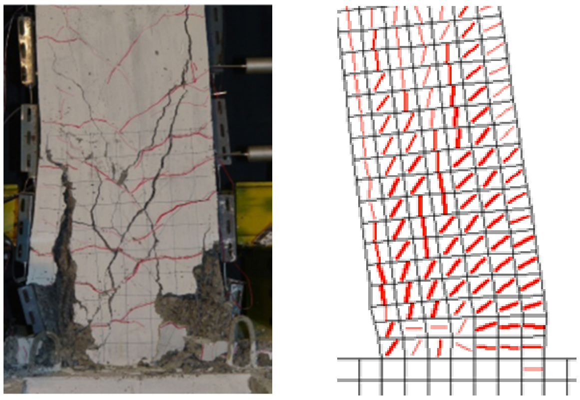

(a) Experimental (b) Analytical

Figure 7 Comparison of final failure mode of specimen S2M [7]

A comparison of analytical and experimental response is given in Table 4. It can be seen that there is a good agreement between the calculated and measured peak lateral forces for all the three RC column specimens.

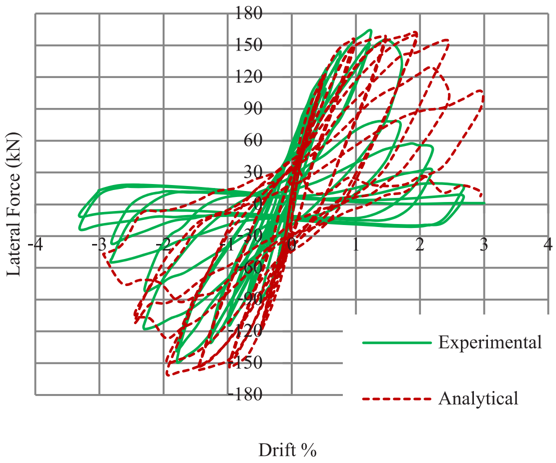

Figure 8 Load-displacement responses of S3S

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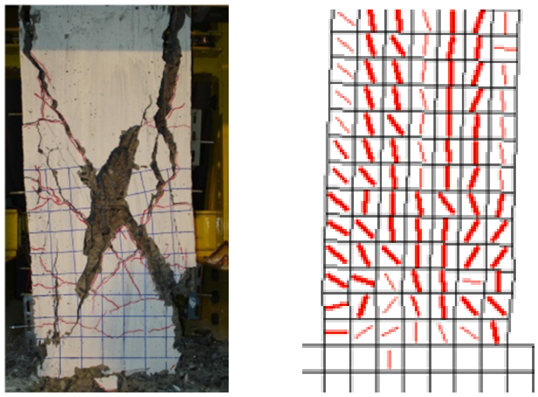

a) Experimental (b) Analytical

Figure 9 Comparison of final failure mode of specimen S3S [7]

Table 4 Comparison of analytical and experimental response

Specimen	Peak lateral force (kN)		Analytical/ Experimental
	Experimental	Analytical	
S1L	86.83	89.1	1.026
S2M	121.44	118	0.971
S3S	162.75	160.4	0.985

7. Conclusions

Cyclic response of non-ductile RC columns was analytically investigated using non-linear finite element software, VecTor2. A total of 3 column specimens, previously tested, representing the typical non-ductile long, medium and short columns in Thailand were modeled with smeared reinforcement. All the columns were subjected to same loading conditions. From the results following conclusions can be made:

1. Cyclic response of non-ductile columns can be assessed with good accuracy and reliability using finite element software VecTor2.

2. Both the longitudinal and transverse reinforcements can be modeled as smeared reinforcement to yield accurate and reliable cyclic response.

3. The default material and analytical models should be used unless the use of any other model is justified.

4. Post peak response and strength degradation for S1L and S2M were successfully captured by VecTor2, however, for S3S the sudden drop in the lateral strength was not assessed accurately. It may be due to the modeling of shear reinforcement as smeared reinforcement.

The seismic response of the non-ductile RC columns can be assessed with good accuracy using non-linear FE analysis programs such as VecTor2. After the assessment of complete load-deformation response the seismic deficiency of these columns can be improved in time by considering the appropriate retrofitting techniques.

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References

[1] Warnitchai, P., "Development of seismic design requirements for buildings in Bangkok against the effect of distant large earthquakes", 13th World Conference on Earthquake Engineering, Vancouver, B.C., Canada, 2004 Paper No. 744

[2] SUESUTTAJIT, C., "A Survey of Configuration Irregularities in Typical Multi-Story Concrete Buildings in Thailand", *Asian Institute of Technology*, Master thesis, 2007.

[4] Mostafaei H, Vecchio FJ, Kabeyasawa T. Nonlinear displacement-based response prediction of reinforced concrete columns. Engineering Structures 2008; 30(2008): 2436-2447

[3] Sagbas, G., Vecchio, F.J. and Christopoulos, C. (2011) 'Computational Modeling of the seismic Performance of Beam-Column Subassemblies', Journal of Earthquake Engineering, 15: 4, 640-663

[6] Cofer W.F., Zhang Y. and McLean D.I. (2002) 'A Comparison of Current Computer Analysis methods for Seismic Performance of Reinforced Concrete Members', Finite Elements in Analysis and Design, 38 (2002): 835-861

[7] Rodsin, K., Warnitchai, P., Awan, T., "Ultimate drift at gravity load collapse of RC non-ductile column", The 5th Civil Engineering Conference in the Asian Region and Australasian Structural Engineering Conference 2010, Sydney, Australia, 2010, paper 178.

[8] Vecchio F.J. Disturbed stress field model for reinforced concrete: Formulation. ASCE Journal of Structural Engineering 2000; 126(8): 1070-7.

[9] Vecchio F.J., Collins MP. The modified compression field theory for reinforced concrete elements subjected to shear. ACI Structural Journal 1986;83(2):219-31.

[10] Wong, P.S., Vecchio, F.J. and Trommels, H. [2013] "VecTor2 and FormWorks User's Manual," Second Edition, Technical Report, Dept. of Civil Engineering, University of Toronto, Toronto.

[11] Vecchio, F.J., and Lai, D., 2004. "Crack Shear-Slip in Reinforced Concrete Elements", journal of AdvancedConcrete Technology, Vol. 2, No. 3, pp. 289-300

[12] Palermo, D., and Vecchio, F.J., 2002, "Behavior and Analysis of Reinforced Concrete Walls Subjected to Reversed Cyclic Loading", Publication No. 2002-01, Department of Civil Engineering, University of Toronto, 351 pp.