Simplified numerical modelling of reinforced concrete columns with smooth rebars

T. Zhang & I. Prionas
Blue Barn Consulting Limited, Auckland.

E. Opabola
University of Auckland, Auckland.

ABSTRACT

Pre-1970s concrete buildings in New Zealand were commonly constructed with smooth longitudinal reinforcing bars. The NZSEE guidelines for the seismic assessment of existing buildings (NZSEE C5) recognises that the seismic behaviour of concrete structural components or global structures constructed with smooth bars is different from those constructed with deformed bars. Thus, careful considerations and assumptions are required to assess the seismic behaviour of these structures. Where numerical modelling is employed to simulate existing concrete columns reinforced with smooth bars, the bond-slip properties between the smooth bars and the surrounding concrete is the key factor that influences the column behaviour and in particular the post-yield behaviour. It is impracticable and computationally inefficient for the engineers in practice, to explicitly model the bond-slip phenomenon within concrete structures, usually determined from experimental tests, and incorporate it into a comprehensive global model of the existing buildings while utilising sophisticated non-linear time history analysis. In this paper, a simplified numerical model is investigated and finally proposed to simulate the cyclic behaviour of columns reinforced with smooth bars by employing regularisation techniques to the material constitutive models. The proposed model was verified against experimental results available in the literature and the provisions of the NZ seismic assessment guidelines. The results indicate that the model is capable of estimating with good accuracy lateral strength, cyclic strength and stiffness degradation, pinching effect, and deformation capacity of columns with smooth bars.

1 INTRODUCTION

In New Zealand, the use of smooth plain bars as longitudinal reinforcement was a typical construction practice for reinforced concrete (RC) buildings constructed before the 1970s. The 2010/2011 Canterbury Earthquakes highlighted the seismic vulnerabilities of old concrete buildings reinforced with smooth rebars. Columns are generally the most important structural elements for moment-resisting concrete frame structures, whereas the use of smooth reinforcement may limit their seismic performance. Recent experimental studies (Arani et al. 2013, Goksu et al. 2014, Melo et al. 2015) show that the behaviour of columns with smooth bars is dominated by a rocking-like (i.e. fixed-end rotation) mechanism with development of a limited number of cracks. Additionally, the energy dissipation capacity and deformation capacity were limited to a relatively low level compared to columns with deformed bars.

For non-linear time history analysis (NLTHA) of older RC structures, the accuracy of modelling of columns with smooth bars is important as the columns play a key role in the final global response, especially in the case of strong beam-weak column configurations. The hysteresis response (e.g. strength and stiffness degradation) needs to be accurately estimated in order to achieve reasonable force distribution between the columns and beams after yielding. An important factor influencing the behaviour of RC columns with smooth bars is the bond strength between concrete and steel. However, the bond-slip properties are difficult to determine, and it is not computationally efficient for NLTHA. For practical purposes, it is common to model the rocking behaviour of columns by introducing a rotational spring at the column base. This method is valid for pushover analysis, but it is not capable of computing the cyclic stiffness degradation in NLTHA to accurately re-distribute the seismic demands on structural elements. Thus, the development of a simple and efficient numerical model for columns with smooth reinforcement is needed for NLTHA of reinforced concrete structures.

In this paper, a simple numerical model is developed by adopting a regularised stress-strain relationship of reinforcement. The stress-strain curve of reinforcement is regularised based on the rocking model for columns with smooth reinforcement, proposed by Opabola et al. (2019), in the technical revision of Part C5 (Yellow Chapter, 2018) of the NZSEE assessment guidelines. The proposed model adequately captures the deformation capacity and cyclic behaviour in terms of lateral strength, strength and stiffness degradation, and pinching of columns with smooth bars.

2 PROPOSED APPROACH FOR MODELLING COLUMNS WITH SMOOTH REBARS (NZSEE C5 MODEL)

As described in Section 1, the deformation capacity of RC columns with smooth reinforcement is typically governed by the fixed-end rotation mechanism, which means the post-yield response of the column is dominated by rocking at a single crack at the column-foundation interface. The current NZSEE assessment guidelines Part C5 (NZSEE C5) provides detailed calculations on determining the deformation capacity of columns reinforced with plain bars. The development of the provisions in NZSEE C5 (Yellow Chapter) is based on a mechanics-based rocking model (C5 rocking model) characterised by dual-phased 'pseudo-ductile' behaviour including an elastic phase and a rocking phase (Opabola et al. 2019). The elastic response of the column with smooth rebars is similar to that of columns with deformed bars. After yield, the response of the column would be dominated by rocking that continues until the overturning point where the lateral resistance drops to zero. This rocking model was assessed and verified using 65 test columns with smooth reinforcement on estimating the deformation capacity.

For columns with smooth reinforcement, the vertical reinforcement is unlikely to develop strain hardening after yielding in tension due to the weak bond strength between concrete and reinforcement. Once the column cracks at the base, bond-slip is likely to occur, and the column deformation will be concentrated at the crack at the column base with no secondary cracks occurring. The tension force carried by the vertical reinforcement will not increase, and the lateral resistance starts to drop. This behaviour is well described by the C5 rocking model. The post-yield stress-strain response of reinforcement is modified in the numerical modelling to capture the column's strength degradation after yielding. After the reinforcement reaches yield strain, the stress then drops until the ultimate strain is reached. The ultimate tensile strain of the modified reinforcement stress-strain curve is determined according to the overturning rotation calculated following C5 guidelines. The ratio of ultimate strain and yield strain is assumed to be same as the ratio of overturning rotation and yield rotation, as shown in Figure 1. Thus, the ultimate strain of the regularised columns with smooth reinforcement can be expressed as Equation 1. The yield rotation and overturning rotation are determined by Equation 2 and 3 as recommended by NZSEE C5 guidelines based on the research by Opabola and Elwood (2019, 2010).

$$\varepsilon_{u\_model} = (\frac{\theta_o+\theta_y}{\theta_y})\varepsilon_y \tag{1}$$

$$\theta_y = \beta\frac{2\varepsilon_y}{h_c}(\frac{L_c}{3}) \tag{2}$$

$$\theta_o = \frac{h_c-c}{2L_c} \tag{3}$$

Where θ_y is the yield rotation, θ_o is the overturning rotation, ε_y is the yield strain of reinforcement, h_c is the column depth in the critical loading direction, L_c is the shear span of the column, β is a factor accounting for the contribution of bar slip and shear deformation, c is the depth of neutral axis which is computed by stress block theory.

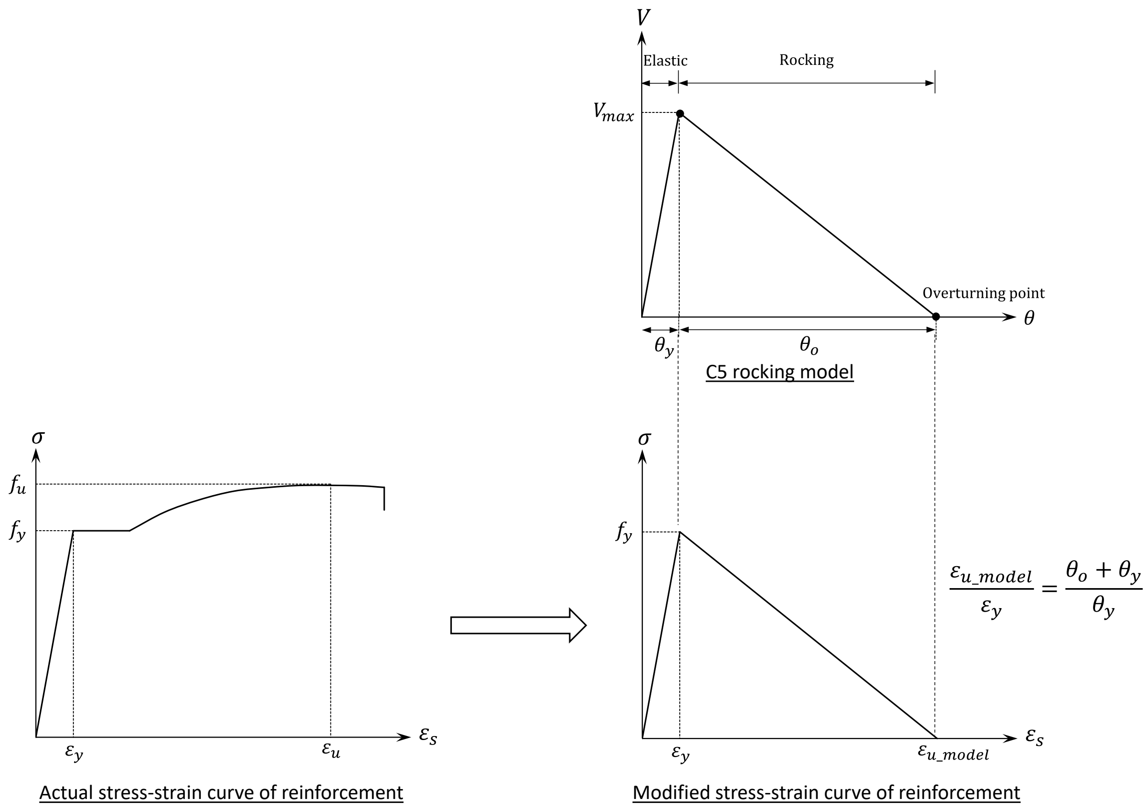


Figure 1: Proposed approach for regularisation of reinforcement stress-strain response for columns reinforced with smooth reinforcement

3 PROPOSED FINITE ELEMENT MODEL

In this paper, DIANA FEA software was selected for the numerical analysis. DIANA is an extensive, multi-purpose finite element software with robust functionality which includes extensive material models, element libraries and analysis procedures. For modelling of concrete structures, detailed reinforcement and concrete material models are incorporated in the program that take into account compression softening, tension stiffening and softening of concrete as well as dowel action and buckling of reinforcement. Additionally, a sophisticated cracking criterion is developed in the program to calculate stress, crack rotation and shear slip. Thus, the program is able to provide an accurate estimation on the behaviour of concrete structures including load-deformation response, cracking, damage progression and post-peak behaviour. Moreover, a well calibrated subassembly model with nonlinearity can be easily incorporated into a global structure model for non-linear time history analysis.

3.1 Model description

In DIANA, a reinforced concrete structure can be modelled by plain concrete elements (e.g. line element, shell element and solid element) and steel reinforcing bars. For the consideration of computational efficiency in non-linear time history analysis, the line element is preferred for modelling the concrete section of columns and hence it is used in this study. A total of 9 integration points through the cross-section is assigned to the concrete element to achieve accurate modelling results with non-linear material. For columns with smooth bars, the behaviour of the reinforcement is critical. Thus, the vertical reinforcement location and strains are discretely modelled. Two-node truss elements with uniform cross-sectional area were employed to represent each individual vertical reinforcement. Non-linear material properties are assigned to the concrete elements and steel reinforcement. The concrete material model accounts for cracking failure in tension and crushing failure at compressive and shear stresses. The steel reinforcement is modelled with Von Mises type elasto-plastic material models. The reinforcement is fully embedded in the concrete elements in which they are located and are therefore fully coupled.

3.2 Steel constitutive model

The stress-strain response of the reinforcing steel implemented in the column model used the non-linear hysteric model is as shown in Figure 2. The backbone of the model is a regularised curve following the approach proposed in Section 2, which includes an initial linear-elastic response and a linear stress degradation phase until rupture. The model includes Bauschinger effect in which the reinforcement exhibits premature yield upon load reversal after plastic pre-straining due to stress change.

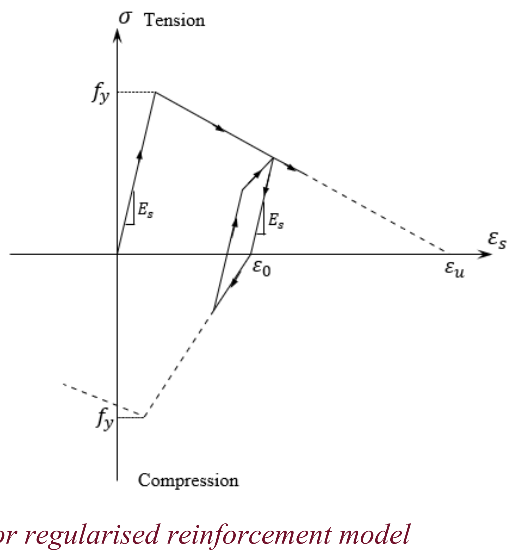


Figure 2: Hysteresis behaviour for regularised reinforcement model

3.3 Concrete constitutive model

A non-linear rotating crack model is adopted for the cracked concrete element in DIANA. The constitutive model is based on total strain and was developed following the Modified Compression Field Theory (MCFT), originally proposed by Vecchio & Collins (1986). The total strain-based crack model follows a smeared approach for the fracture energy, and the cracks can re-orientate to align with the changing direction of the principal concrete compressive stress field. The three-dimensional extension to this theory is proposed by Selby & Vecchio (1993), which is the basis of the total strain crack model implemented in DIANA. The concrete stress-strain relationship of the total strain concrete model utilised in the modelling is shown in Figure 3. It is noted that unloading is not to the origin (secant unloading), the sophisticated Maekawa cracked concrete model incorporates damage memory which effectively introduces elemental elongation during hinging cycles.

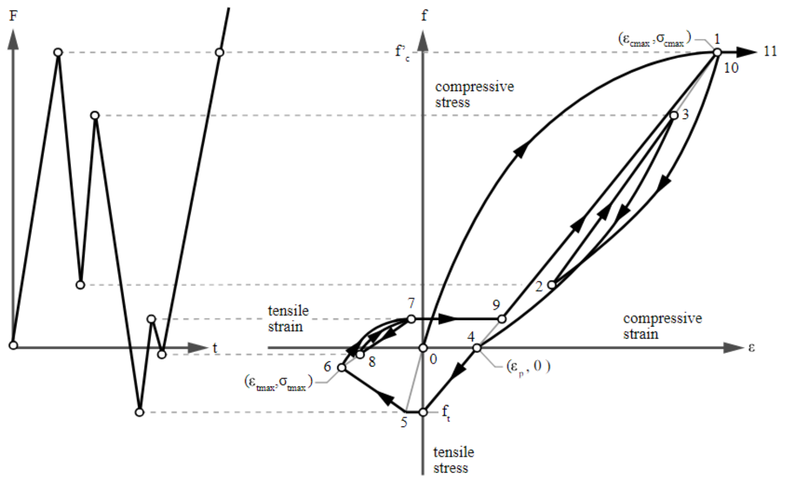
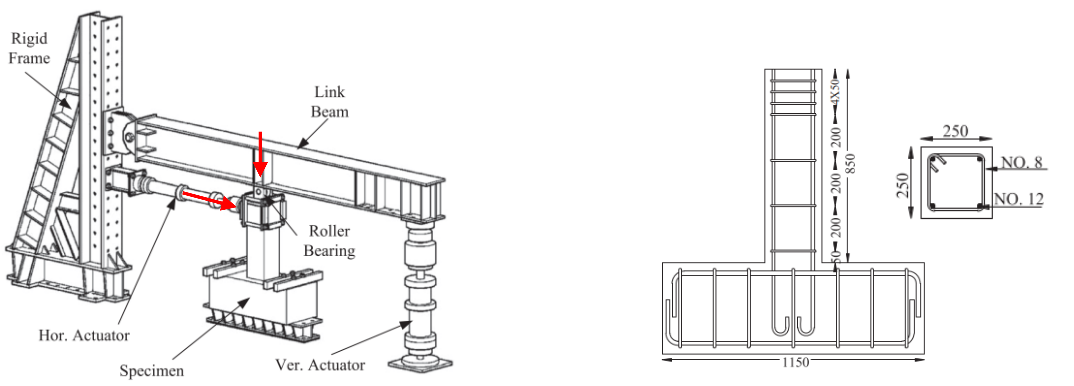


Figure 3: Hysteresis behaviour for Maekawa cracked concrete model (after: Diana FEA manual)

4 MODEL VERIFICATION

4.1 Test column description

The experimental program conducted by Arani et al. (2013) on columns with smooth reinforcement was used to validate the proposed simplified modelling approach in our study. A total of four concrete column specimens with identical dimensions but reinforced with various types of splices were tested in the experimental program. All the column specimens are designed with a square cross-section (250 mm by 250 mm) and are reinforced with four 12 mm diameter longitudinal reinforcement. The shear span of the specimens is 750 mm. The concrete compressive strength and reinforcement yield strength are 22.5 MPa and 370 MPa, respectively. The specimens were fixed at base and loaded as cantilevers with cyclic lateral load applied at the top, as shown in Figure 4(a). The lateral loading protocol consists of 0.35%, 0.50%, 0.75%, 1.00%, 1.40%, 1.75%, 2.20%, 2.75%, 3.50%, 4.50%, 6.00% and 7.50% drift ratios with three repeated cycles at each drift level. A constant axial load of $0.15A_g f_c'$ was applied throughout the testing. Our developed FE model uses the exact same loading protocols.



(a) Test setup (b) Details of column specimen WOS-C

Figure 4: Illustration of the experimental equipment and specimen (after: Arani et al. 2013)

The observed results of the experiments indicate that the response of the columns is dominated by a limited number of flexural cracks at the base of the columns with the lower crack to be the widest. The hysteresis behaviour of all the tested columns with different types of splice detailing is similar in terms of strength degradation and cyclic stiffness. The hysteresis curve is characterised by a high pinching effect and a low residual displacement (Figure 5). Thus, only one of the column specimens (WOS-C) is selected for the model verification in this study as shown in Figure 4(b).

4.2 Validation of the proposed FEA model to the experimental results

The computed lateral load-displacement response of the numerical column model developed following the simplified regularisation approach in this study is shown alongside the measured experimental results in Figure 5. A comparison of the key simulated response to the experimental results is provided in Table 1. The yield drift Δ_y was estimated from the lateral load-displacement backbone curve at the 75% of the maximum lateral strength, factored by 1.33, based on the reduced stiffness equivalent elasto-plastic yield procedure proposed by Park (1988). The corresponding strength at yield drift was defined as yield strength V_y and the initial stiffness K_y was determined by the secant stiffness at the yield point. The ultimate drift capacity Δ_u was defined as the drift level at which the column exhibits a drop of 20% maximum lateral load resistance (i.e. lateral failure).

Overall, the simulated response of the developed model shows good correlation with the experimental results, well capturing the strength, cyclic degradation of lateral strength and stiffness as well as the overall hysteresis behaviour. Specifically, as indicated in Table 1, the yield, maximum lateral strength and the ultimate drift capacity were adequately estimated with the mean ratios of the simulated to the observed results ranging from 0.87 to 1.13. However, a lower simulated yield drift than the measured result was predicted by the model. The mean ratio of the yield drift was 0.46. The lower yield drift was strongly associated with the higher initial stiffness predicted by the models with a mean ratio around 2.88. The inability of MCFT models to capture the initial stiffness was also concluded by Pugh (2013). A model with 'pre-cracked' consideration to simulate the shrinkage cracking was proposed by Palermo and Vecchio (2007) to improve the accuracy of predicting the initial stiffness and can be used for the further development of the numerical models in DIANA. Other observations indicate that the deviations between the boundary conditions and the application of loading between the developed model and the experimental specimen could result in differences to the initial stifness.

As shown in Figure 5, a pinching of the simulated hysteresis response was clearly observed, which matches the experimental results. This pinching behaviour of the developed models results in a low cyclic residual drift that correlates well with the testing observations.

Table 1: Key results of the developed model Vs experimental results

Loading direction	$\frac{\Delta_{y\_model}}{\Delta_{y\_test}}$	$\frac{K_{y\_model}}{K_{y\_test}}$	$\frac{V_{y\_model}}{V_{y\_test}}$	$\frac{V_{max\_model}}{V_{max\_test}}$	$\frac{\Delta_{u\_model}}{\Delta_{u\_test}}$
Positive	0.60	1.73	1.04	1.07	0.72
Negative	0.32	4.03	1.17	1.18	1.02
Mean	0.46	2.88	1.10	1.13	0.87

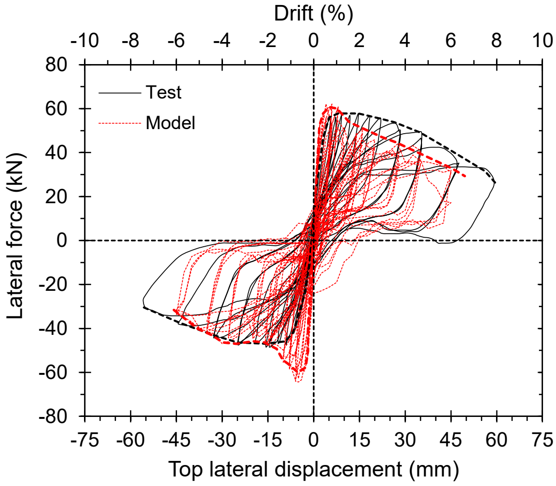


Figure 5: Comparison of experimental and simulated hysteresis response

4.3 Comparison with NZSEE C5 rocking model

Figure 6 below, compares the column response calculated by C5 rocking model and the proposed FE model with material regularisation of reinforcement. As recommended by Opabola et al. (2019), the lateral strength for the C5 rocking model can be reasonably taken as the full theoretical flexural strength. The yield rotation can be computed by Equation 2. The plastic rotation capacity can be calculated as $0.2K$ of the overturning rotation (Equation 3), where coefficient K accounts for the effect of aspect ratio on the post-yield behaviour of concrete columns with smooth bars. The ultimate rotation capacity is determined by summing up the yield and plastic rotation. As shown in Figure 6, the FE model predicts the rocking-like response with clear lateral strength degradation after yield. The ultimate drift capacity is estimated with adequate accuracy compared to the estimated drift by the NZSEE C5 rocking model.

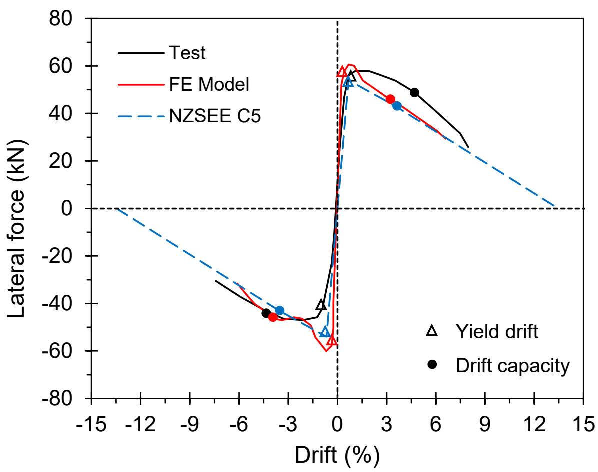


Figure 6: Comparison of column drifts estimated by the developed FE model to the NZSEE C5 model

5 CONCLUSIONS

This paper proposes a simplified approach for modelling concrete columns reinforced with smooth plain bars for use in NLTHA. The constitutive model of the reinforcement is regularised based on the NZSEE C5 rocking model and it is incorporated in a finite element (FE) model developed in DIANA software. The modelling results using the proposed regularisation approach was verified against experimental results and the NZSEE-C5 rocking model to establish the capabilities and limitations of the approach to capture the cyclic behaviour of columns reinforced with plain bars. Overall, the proposed approach enables the FE model to reproduce the cyclic response with good accuracy. The key conclusions drawn from this study included:

- By employing regularisation on the stress-strain relationship of reinforcement, the FE model is able to predict the strength degradation after yield.
- The hysteresis response including stiffness degradation, pinching effect and low residual deformation is well captured by the FE model with regularised reinforcement.
- The deformation capacity of the columns can be well estimated by the FE model compared to the C5 rocking model.
- The initial stiffness is overestimated by the FE model and further study is recommended by considering 'pre-cracked' behaviour in FE models.

6 REFERENCES

Arani, K. K., Marefat, M. S., Amrollahi-Biucky, A., & Khanmohammadi, M. (2013). Experimental seismic evaluation of old concrete columns reinforced by plain bars. The Structural Design of Tall and Special Buildings, 22(3), 267-290.

DIANA FEA. (2020). User's Manual 10.4.

Goksu, C., Yilmaz, H., Chowdhury, S. R., Orakcal, K., & Ilki, A. (2014). The effect of lap splice length on the cyclic lateral load behavior of RC members with low-strength concrete and plain bars. Advances in Structural Engineering, 17(5), 639-658.

Kam, W. Y., Pampanin, S., & Elwood, K. (2011). Seismic performance of reinforced concrete buildings in the 22 February Christchurch (Lyttelton) earthquake. Bulletin of the New Zealand Society for Earthquake Engineering, 44(4), 239-278.

Melo, J., Varum, H., & Rossetto, T. (2015). Experimental cyclic behavior of RC columns with plain bars and proposal for Eurocode 8 formula improvement. Engineering Structures, 88, 22-36.

NZSEE Assessment Guidelines. (2018). Part C5 – Technical proposal to revise the engineering assessment guidelines of concrete buildings.

Opabola, E. A., Elwood, K. J., & Oliver, S. (2019). Deformation capacity of reinforced concrete columns with smooth reinforcement. Bulletin of Earthquake Engineering, 17(5), 2509-2532.

Opabola, E. A., & Elwood, K. J. (2020). Simplified Approaches for Estimating Yield Rotation of Reinforced Concrete Beam-Column Components. ACI Structural Journal, 117(4), 279-291.

Palermo, D., & Vecchio, F. J. (2007). Simulation of cyclically loaded concrete structures based on the finite-element method. Journal of Structural Engineering, 133(5), 728-738.

Park, R. (1988). Ductility evaluation from laboratory and analytical testing. Paper presented at the Proceedings of the 9th world conference on earthquake engineering, Tokyo-Kyoto, Japan.

Pugh, J. S. (2013). Numerical simulation of walls and seismic design recommendations for walled buildings.

Selby, R. G. (1995). Three-dimensional constitutive relations for reinforced concrete.

Vecchio, F. J., & Collins, M. P. (1986). The modified compression-field theory for reinforced concrete elements subjected to shear. ACI J., 83(2), 219-231.