Discussion of "Rotation-Based Shear Failure Model for Lightly Confined RC Columns" by Wassim M. Ghannoum and Jack P. Moehle

October 2012, Vol. 138, No. 10, pp. 1267–1278.
DOI: 10.1061/(ASCE)ST.1943-541X.0000555

Li-Wei Liu[1]; Wei-Jian Yi, Ph.D.[2]; and Yun Zhou, Ph.D.[3]

[1]Ph.D. Student, College of Civil Engineering, Hunan Univ., Changsha 410082, P.R. China (corresponding author). E-mail: hnuliuliwei@yahoo.cn

[2]Professor, College of Civil Engineering, Hunan Univ., Changsha 410082, P.R. China. E-mail: wjyi_hnu@163.com

[3]Associate Professor, College of Civil Engineering, Hunan Univ., Changsha 410082, P.R. China. E-mail: zhouyun05@gmail.com

The discussers appreciate the authors' innovative and comprehensive work in proposing a rotation-based shear failure model for lightly confined columns that sustain flexural yielding prior to shear failure [or flexure-shear-critical (FSC) columns]. The method that rotation limits can be used for performance-based assessment of existing buildings is first proposed in this paper, and the authors indicated the restraint provided by framing members may vary from one end of the column to the other and that a significant portion of later drift is attributable to joint and beam deformations, which implies that it is unreliable to utilize lateral drift as the critical parameter for failure criterion judgment. It is a highlighted problem in practical application, and it can be solved by the proposed rotation-based shear failure model. It is the opinion of the discussers that in this paper, some details require clarification as follows:

1. The effects of longitudinal bar-slip from joint or footing anchorage regions were modeled using a zero-length fiber-section model in this paper. To validate the feasibility of choosing the bond-slip model, the error between analytical and experimental elastic stiffness of the column was defined as Eq. (1). The choice of $u_e = 0.9\sqrt{f'c}$ MPa produced a mean error in elastic stiffness across all database columns of 0.01 with a standard deviation of 0.39. The precision of the drift directly impacts the statistical representation of rotations, which is much more important than the elastic stiffness. Regarding the point of maximum shear, the authors only provided calculating results for one column to validate the correctness of choosing this method (Fig. 4). In the discussers' opinion, the drift at maximum shear for all 56 tests in the database should be compared with analytical results, and the mean error and standard deviation should be provided

$$e_s = \frac{K_{y\text{-analysis}} - K_{y\text{-test}}}{K_{y\text{-test}}} \tag{1}$$

2. In the section titled "Sensitivity of Extracted Rotation Values to Bond-Stress Values," the point of maximum shear $\delta_{\max}$ would change with different bond stress, the insensitiveness found in total rotation $\theta_{\text{Tot max}}$ to bond-stress, and the relationship between rotation components in Eq. (2) and Eq. (3) should be further verified. The authors indicated that bar-slip-induced rotations can constitute more than 50% of column end rotation at shear failure initiation, and four related studies were referred to support the analyzing results. From Ghannoum and Moehle's (2012) work, the main focus of the discussion was the shaking table test. In this test, the rotations along height h of columns were measured, but there were no specific data or explanations about the separated bond-slip-induced rotation that would support the analyzing results. Kowalsky et al.'s (1999) work is mainly discussing the lightweight concrete column, and the differences in materials should be noted. In the work by Saatcioglu et al. (1992) and Sezen and Moehle (2006), none of the recorded slip rotations exceeded 50% of the total deformation

$$\theta_{\text{BSTot max}} = 0.55\theta_{\text{Tot max}} \tag{2}$$

$$\theta_{f\text{Tot max}} = 0.45\theta_{\text{Tot max}} \tag{3}$$

3. In Table 3, the value of $\theta_{\text{Tot }Pl\text{ max}}/\theta_{\text{Tot max}}$ varied across different columns, among which the length of plastic hinge length h would have large influences on the final analytical results. The discussers feel that the explanation and verification for this point in the paper were not sufficient.
4. In this paper, only elastic shear deformation was considered in the analytical representation of the columns. The mentioned limited cases, in which shear cracking occurred prior to reaching $V_{\max}$, can be excluded from the database to minimize the dispersion of the results.
5. In addition, shear deformations were modeled using a shear spring with elastic stiffness given by $(5/6GA_g)/(L/2)$, in which L means the length of element, but no specific element was listed.

References

Ghannoum, W. M., and Moehle, J. P. (2012). "Shake-table tests of a concrete frame sustaining column axial failures." *ACI Struct. J.*, 109(3), 393–402.

Kowalsky, M. J., Priestley, M. J. N., and Seible, F. (1999). "Shear and flexural behavior of lightweight concrete bridge columns in seismic regions." *ACI Struct. J.*, 96(1), 136–148.

Saatcioglu, M., Alsiwat, J. M., and Ozcebe, G. (1992). "Hysteretic behavior of Anchorage slip in R/C members." *J. Struct. Eng.*, 10.1061/(ASCE)0733-9445(1992)118:9(2439), 2439–2458.

Sezen, H., and Moehle, J. P. (2006). "Seismic tests of concrete columns with light transverse reinforcement." *ACI Struct. J.*, 103(6), 842–849.