Appendices for:

Performance evaluation of low-rise infilled reinforced concrete frames designed by considering local effects on column shear demand

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Appendix

The execution of the design method described in Section 2 is presented in this section. The calculation for the C1 column on the 1st story of building B1R was selected as an example. Two iterations, including the opening gap dimension (a) equal to 1d and 5d are demonstrated. The process is initiated by estimating the characteristic stiffness parameter (λH) of the infilled RC frame [36]. In this example, the initial angle of the infill wall equivalent diagonal strut with respect to the horizontal axis can be estimated as $\theta_i = \tan^{-1}(H_w/L_w) = \tan^{-1}(2600/3675) = 35.28°$. Hence,

$$\lambda H = \left[\frac{E_w t_w \sin 2\theta_i}{4E_c I_c H_w}\right]^{1/4} \times H = \left[\frac{7078\times100\times\sin(2\times35.28)}{4\times24{,}870\times9.3\times10^8\times2600}\right]^{1/4} \times 2800 = 3.61. \quad \text{(A1)}$$

Note that this characteristic stiffness parameter is assumed to be constant for this example and is used in all the iterations. The axial capacity of the undamaged infill wall equivalent strut (C_i) is calculated using Eqs. (6) and (8), respectively. The effective width of the strut (w) can be calculated as $w = 0.25(\lambda H)^{-1.15}\, d_w = 0.25 \times (3.61)^{-1.15} \times 4502 = 257$ mm.

For the 1st iteration, the opening gap (a) is assumed to be at 1d (275 mm for this example).

Step 1: Calculate the strut force.

The strut capacity reduction factor (α) is calculated using Eq. (4):

$$\alpha = 1.05 - 1.1(a/H_\mathrm{w}) = 1.05 - 1.1 \times (275/2600) = 0.93. \text{ (A2)}$$

In this example, the compressive strength of the infill prism is 6.5 MPa and the strength factor of 1.9 is used. Hence, the axial strut capacity can be computed as

$$\alpha C_\mathrm{i} = \alpha \psi w t_\mathrm{w} f'_\mathrm{m} = 0.93 \times 1.9 \times 257 \times 100 \times 6.5/1000 \ = 295 \text{ kN. (A3)}$$

Step 2: Calculate the column shear demand.

The process is initiated by estimating the plastic moment (M_p) and checking the failure mechanism of the column using Eqs. (1)–(3), respectively. At this stage, the strut angle is calculated as $\theta_\mathrm{w} = \tan^{-1}$ ((2600-275)/3750) = 32.32°. The column shear demands are calculated using Eqs. (1) and (2) as follows:

$$V_\mathrm{a} = \frac{2M_\mathrm{p}}{H_\mathrm{w}} + \frac{\alpha C_\mathrm{i} \cos\theta_\mathrm{w}(H_\mathrm{w} - a)}{H_\mathrm{w}} = \frac{(53944 + 84799)}{2600} + \frac{295\cos(32.32) \times (2600 - 275)}{2600} = 276 \text{ kN, (A4)}$$

and

$$V_\mathrm{b} = \frac{2M_\mathrm{p}}{a} = \frac{(53944 + 84799)}{275} = 505 \text{ kN. (A5)}$$

The lower value is used as the required shear strength (V_u) of the column; in this case,

$$V_\mathrm{u} = V_\mathrm{a} = 276 \text{ kN. (A6)}$$

Step 3: Check the column shear capacity.

At this stage, the shear-span-to-effective-depth ratio (a/d) = 1. The column shear strength is calculated using the strut-and-tie approach with the formula $V_\mathrm{n} = \lambda_\mathrm{s} f'_\mathrm{c} A_\mathrm{str} \cos\varphi$, which depends on the geometric parameter of the strut-and-tie model. In this study, the shear strength is calculated based on the approach by Li and Hwang [44]. The column shear capacity is found to be $V_\mathrm{n} = \lambda_\mathrm{s} f'_\mathrm{c} A_\mathrm{str} \cos\varphi = (0.59 \times 28 \times 27149 \times \cos(45.60))/1000 = 314$ kN.

Hence, the column shear demand (V_u) and capacity (V_n) can be compared. V_u = 276 kN < V_n = 314 kN.

Based on this result, the column exhibits sufficient shear strength. The shear demand-to-capacity ratio is 0.87.

For the next iteration, the column shear demand to capacity can be checked by increasing the opening gap (a) and repeating all the steps until the a/d ratio reaches 4. When the a/d ratio exceeds 4, the procedure is the same, except for Step 3, where the strength is calculated using ACI 318-14. The procedure is as follows:

For the nth iteration, a/d > 4.

Step 1: In this example, if a/d = 5 (a = 5d or 1375 mm) is selected. For the column, the strut force, strut capacity reduction factor (α), axial capacity, and angle are recalculated as follows:

$$\alpha = 1.05 - 1.1\left(\frac{a}{H_\mathrm{w}}\right) = 1.05 - 1.1 \times \left(\frac{1375}{2600}\right) = 0.47, \text{ (A7)}$$

$$\alpha C_\mathrm{i} = \alpha \psi w t_\mathrm{w} f'_\mathrm{m} = 0.47 \times 1.9 \times 257 \times 100 \times 6.5/1000 = 149 \text{ kN, (A8)}$$

$$\theta_\mathrm{w} = \tan^{-1}((2600\text{-}1375)/3675) = 18.43°. \text{ (A9)}$$

Step 2: The column shear demand is calculated using Eqs. (1) and (2) as follows:

$$V_a = \frac{2M_p}{H_w} + \frac{\alpha C_i \cos\theta_w (H_w - a)}{H_w} = \frac{(53{,}944 + 84{,}799)}{2600} + \frac{149\cos(23.53) \times (2{,}600 - 1{,}375)}{2600} = 118 \text{ kN}, \text{(A10)}$$

$$V_b = \frac{2M_p}{a} = \frac{(53{,}944 + 84{,}799)}{1375} = 101 \text{ kN}. \text{(A11)}$$

Therefore, the lower value is used as the required shear strength (V_u) of the column; in this case,

$$V_u = V_b = 101 \text{ kN}. \text{(A12)}$$

Step 3: In this case, $a/d = 5$. Column shear capacity is calculated using a code-based formula. In this study, the formula adopted in ACI 318-14 is used. Hence, the column shear capacity is a combination of the concrete and rebar strengths.

$$V_n = 0.17\left(1 + \frac{N_u}{14A_g}\right)\lambda_c\sqrt{f_c'}\,b_w d + \frac{A_{sv} f_{yv} d}{s}, \text{(A13)}$$

$$V_n = 0.17 \times \left(1 + \frac{24{,}460}{14(325 \times 325)}\right) \times 1.0 \times \sqrt{28} \times (325 \times 275) + \frac{157 \times 400 \times 275}{125} = 220 \text{ kN}, \text{(A14)}$$

where A_g is the gross cross-sectional area; N_u is the axial force; b_w is the column width; λ_c is the modification factor for lightweight concrete; and A_{sv}, f_{yv}, and s are the transverse reinforcement area, yield strength, and spacing, respectively.

Hence, the column shear demand (V_u) and capacity (V_n) are $V_u = 101$ kN < $V_n = 220$ kN.

Based on this result, the column has sufficient strength.

For other iterations, the column shear demand and capacity can be checked by varying the opening gap (a) and repeating all steps. The shear-demand-to-capacity ratios for different values of a/d are shown in Fig. A1.

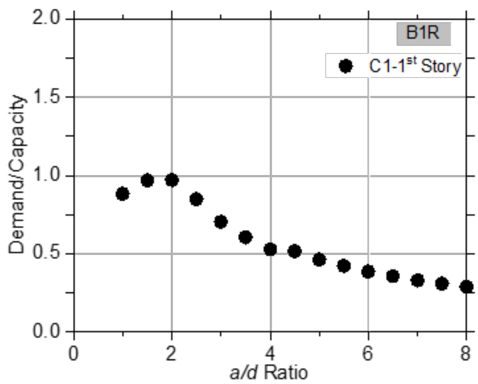

Fig. A1 C1 column shear demand-to-capacity ratio for different opening gaps.