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Stiffness Demand for Pin-supported Walls in RC Moment Resisting Frames

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ABSTRACT
A simple criterion for the stiffness demand of pin-supported walls in multi-story reinforced concrete moment-resisting frames is proposed. A stiffness ratio is defined as the ratio of the stiffness of the walls as analogs of simply-supported beams under uniformly distributed loads to that of the first modal stiffness of the moment frame. Since the structural height is implicitly included in the stiffness ratio, a height-independent value of two is suggested for multi-story moment frames of various number of stories up to 12 stories. Incremental dynamic analysis of 4-, 8- and 12-story architype moment-resisting frames equipped with pin-supported walls of various stiffness show that the recommended stiffness ratio can ensure a global plastic mechanism in a moment-resisting frame no matter if it follows the capacity design concept of strong column-weak beam or not. In spreading the lateral drift among stories, the pin-supported walls also increase the lateral strength of the structure by motivating more elements in the structure to contribute to the earthquake resistance. The strength increase is more significant for weak column-story beam frames than strong column-weak beam ones.

KEYWORDS: *pin-supported wall, reinforced concrete frame, stiffness ratio, higher mode vibration*

1. INTRODUCTION

Past earthquakes have witnessed the vulnerability of reinforced concrete (RC) moment frames to weak story collapse [1, 2]. This has continued to be a problem even if the strong column-weak beam design concept was implemented in proportioning the frame elements. For example, recently built RC frames were found collapsed in weak story mechanisms in the 2008 M8.0 Wenchuan earthquake [3].

Pin-supported walls (referred to as PS walls hereinafter) provide a robust means to avoid story drift concentration in moment frames. As a seismic retrofit effort of an 11-story steel reinforced concrete frame that was built in 1970s, six post-tensioned walls that rest on mechanical pin bearings were attached along the perimeter of the existing buildings to spread the lateral drift among stories. Steel dampers were also installed along both sides of each wall to dissipate energy and thus reduce the seismic response [4, 5].

In such a PS wall-frame system, the walls work as balancers that transfer redundant strength somewhere in the system to where the strength is inadequate. Figure 1.1 demonstrates a simplified model of a PS wall and two single-story subassemblies, which are connected to the PS wall by horizontal links at different levels. In this model, the pin-supported wall provides no additional lateral resistance.

Assume that the i^{th} story alone is not adequate in strength to resist the earthquake action $F_{EQ,i}$ and is expected to go deep into the plastic regime and sustain large deformation. With the existence of the PS wall, the lateral drift of this story is forced to be similar to those of other stories by an additional resisting force of F_w provided by the wall. At the same time, the j^{th} story where the strength is redundant must resist an additional force F_w that is imposed by the wall. Thus, the deformation of this story is increased. In other words, weaker stories use strength from stronger ones so that they do not fail prematurely.

At the same time, the PS wall must sustain the resultant shear force and moment so that this mechanism can be retained even during strong earthquakes. Therefore, proper selection of the dimensions of PS walls is essential in the preliminary design of the system. For the aforementioned retrofit project of the 11-story building, which is the first application of this system, the proportion of the PS walls relied heavily on engineering judgement and extensive nonlinear dynamic analysis. To provide a rational basis for proportioning the PS walls in preliminary design, extensive nonlinear analysis of six architype moment frames is conducted to investigate the relationship between a newly-defined stiffness ratio and how well a global plastic mechanism is formed.

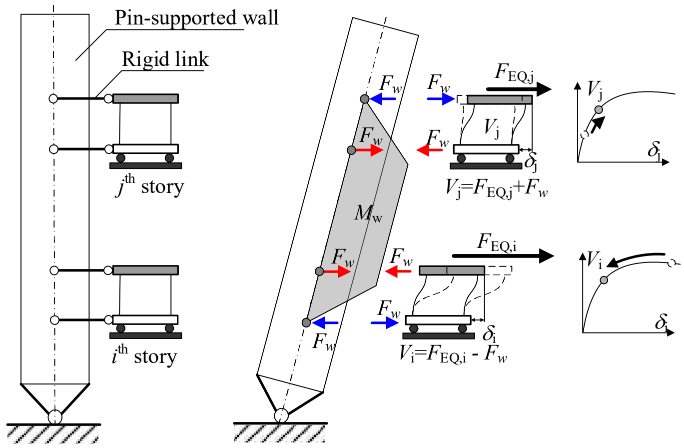

Figure 1.1 Mechanism of deformation pattern control by pin-supported walls [4]

2. ARCHETYPE BUILDINGS

Six RC moment-resisting frame models of 4-, 8- and 12-story are established. All six frames have the same plan layout and are assumed to locate on a stiff soil site of a 0.4g design-level spectral pseudo acceleration, that is, in Intensity VIII region in the Chinese seismic design code [6]. The seismic code also requires that the design of RC frames taller than 40 m in Intensity VIII regions needs special panel review. Therefore, tall RC bare frames without structural walls are rarely seem in seismic prone regions in China. For all frames, the story height is 4.1m for the bottom story, 3.8m for the second story, and 3.6m for all other stories. C40 concrete of 40 MPa nominal compressive strength and HRB400 rebars of 400 MPa nominal yield strength are used. The cast-in-situ RC slabs are 120 mm thick and are reinforced by two layers of 10 mm diameter rebars distributed at 150mm spacing in both directions.

The six frames fall into two categories according to their column-to-beam strength ratios. In one category, the frame members are proportioned in accordance with the Chinese seismic design codes [6]. In particular, the capacity design concept is enforced by specified strong column-weak beam factors, η, that amplify the flexural demands for the columns to satisfy Equation 2.1. The frames in this category are referred to as strong column-weak beam frames (SC frames), hereinafter.

$$\sum M_{\mathrm{c}} = \eta \sum M_{\mathrm{b}} \tag{2.1}$$

where ΣM_{c} and ΣM_{b} are the sum of the design moment of columns and that of the beams framing into the same joint. η is the strong column-weak beam factor. For frames located in regions of Intensity VIII, $\eta = 1.7$ for frames taller than 24 m, and $\eta = 1.5$ for other frames [6].

The resultant beam and column reinforcement ratios, ρ, are given in Figure 2.1 together with the column-to-beam strength ratio, η_{a}, which is calculated by Equation 2.2. In Figure 2.1, the reinforcement ratio, ρ, is the ratio of the area of longitudinal tensile rebars to that of the effective cross section for beams, and the ratio of the total longitudinal rebar area to the total cross section area for columns.

$$\eta_{\mathrm{a}} = \sum M_{\mathrm{cua}} / \sum M_{\mathrm{bua}} \tag{2.2}$$

where ΣM_{cua} and ΣM_{bua} are the sum of the actual flexural strengths of columns and that of the beams framing into the same joint.

It should be noted that η_{a} can be quite different from the strong column-weak beam factor, η, in the design procedure. This is because M_{cua} and M_{bua} are evaluated based on the actual cross section dimensions and reinforcement. Both of them frequently exceed the design moment demands, M_c or M_b.

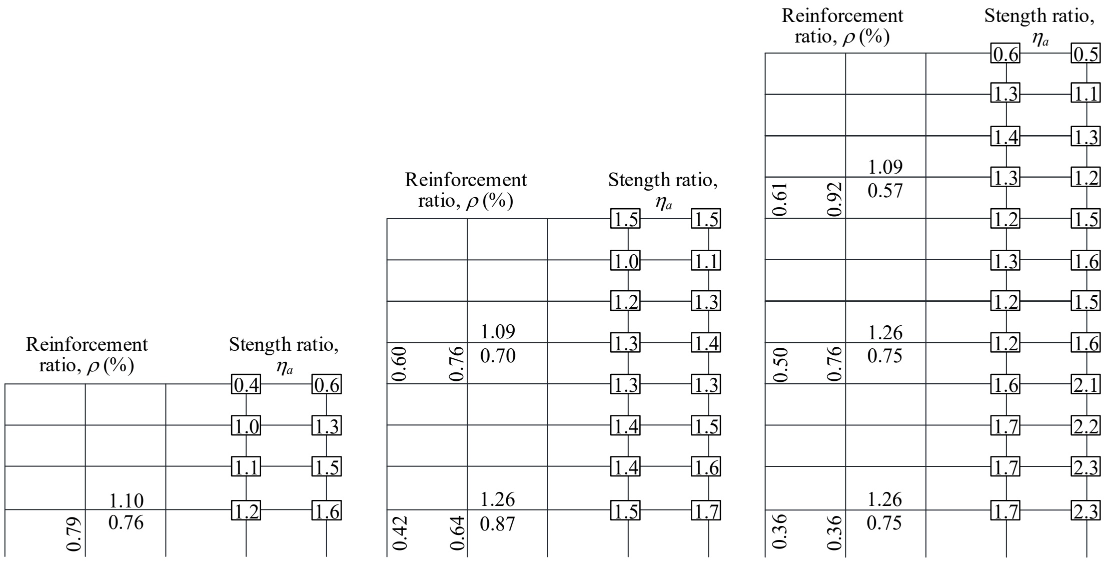

Figure 2.1 Properties of strong column (SC) frames

In the other category, the code-stipulated strong column-weak beam requirement is waived. Meanwhile, larger beam and smaller column cross sections are chosen intendedly to reduce the column-to-beam stiffness ratios. The resultant design is summarized in Figure 2.2. These frames are referred to as weak column-strong beam frames (WC frames), hereinafter.

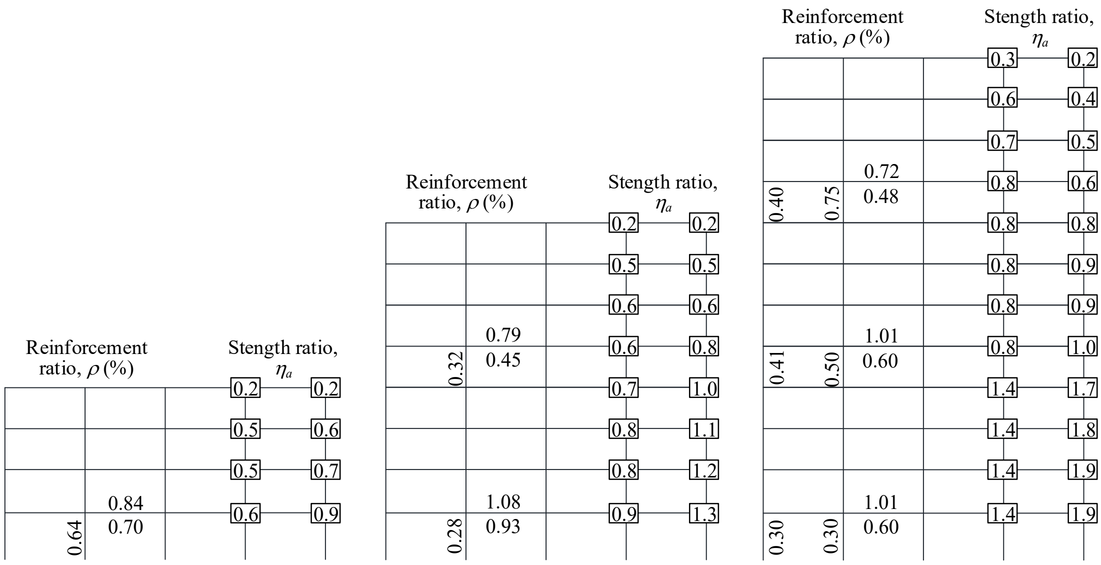

Figure 2.2 Properties of weak column (WC) frames

3. STIFFNESS RATIO AND PLASTICITY RATIO

3.1. Stiffness ratio

The stiffness of PS walls relative to that of the frame is quantified by a stiffness ratio in Equation 3.1, which is the ratio of the stiffness of the wall as analogs of simply-supported beams under uniformly distributed loads, K_w, to that of the first modal stiffness of the moment frame, K_f. Both K_w and K_f represent a global property of either the wall or the frame, rather than story-by-story properties.

$$\beta = \frac{K_w}{K_f} \tag{3.1}$$

$$K_w = \frac{\pi^4 E_w I_w}{2H_w^3} \tag{3.2}$$

$$K_f = (\frac{2\pi}{T_1})^2 \cdot M_1 \tag{3.3}$$

where E_w, I_w and H_w are the Young's modulus, sectional moment of inertia and height of the PS wall, respectively; M_1 and T_1 are the first mode period and participating mass of the bare frame, respectively. The first mode participating mass is calculated by Equation 3.4. For preliminary design practice, M_1 can be approximately taken as 85% of the total mass of the bare frame.

$$M_1 = (\frac{\{\phi_1\}^T[M]\{1\}}{\{\phi_1\}^T[M]\{\phi_1\}})^2 \{\phi_1\}^T[M]\{\phi_1\} \tag{3.4}$$

where $[M]$ and $\{\phi_1\}$ are the mass matrix and the first mode shape of the bare frame, respectively.

3.2. Plasticity ratio

In previous studies, the drift concentration factor (*DCF*) has been used to evaluate the effect of continuous stiffness in spreading lateral drifts, which is defined as the ratio of the maximum to average inter-story drifts [7]. Although *DCF* is a straightforward measure to qualitatively demonstrate the effect of PS walls in creating uniform lateral drift distribution, a desired extent of uniformness needs to be determined before *DCF* can be use as a quantitative design criterion.

To avoid such a difficulty, a plasticity ratio, η_p, with clearer physical significance is proposed as a measure to evaluate the effect of PS walls. It is the ratio of the number of plastic hinges in a moment frame, N, to that of all column and beam ends where potential hinges can possibly form under earthquakes, N_p (Equation 3.5).

$$\eta_p = N_p / N \tag{3.5}$$

The formation of a plastic hinge is judged by the ratio of the curvature at the beam or column end, ϕ, to the yield curvature of the section, ϕ_y. For reinforced concrete beam and column members, the yield curvature, ϕ_y, can be calculated by Equation 3.6. In this study, it is assumed that a plastic hinge forms when $\phi \geq 1.5\phi_y$.

$$\phi_y = (1.7 + 20\rho_l)(1 - \lambda^2)\frac{\varepsilon_{sy}}{h_0} \tag{3.6}$$

where ρ_l is the longitudinal reinforcement ratio, λ is the axial force ratio, ε_{sy} is the yield strain of the longitudinal rebars, h_0 is the effective depth of the member.

4. STIFFNESS DEMANDS

Incremental dynamic analysis is conducted for the architype moment frames with PS walls of various stiffness ratios. The structures are subjected to two suites of ground motions selected from the PEER ground motion database[8], that is, a pulse-like ground motion suite of 26 records and a non-pulse like suite of another 26 records. The spectral acceleration of 5% damping corresponding to the fundamental period of a frame, $S_a(T_1, 5\%)$, is taken as the intensity measure (IM) and the maximum inter-story drift ratio as the damage measure (DM) in the analysis. All ground motion records are normalized by PGV before they are collectively scaled by IM for the incremental analysis.

The fishbone model [9] is used in the analysis in OpenSEES [10]. Fiber-sectioned beam elements are used for the beams and column. The built-in Concrete02 hysteretic model in OpenSees is used for concrete fibers, which adopts the Kent-Scott-Park model for the skeleton curve and is capable of modelling both the strength and stiffness deterioration of concrete. For steel fibers that represent the reinforcing bars, the built-in Hysteretic model of a bilinear skeleton curve is adopted. The PS walls are assumed to remain elastic.

To serve as a basis for preliminary design, an inter-story drift ratio of 2% is taken as the performance target in the incremental dynamic analysis. The Hunt&Fill algorithm [11] is used to search for intensity corresponding to the performance target. The allowable relative error of the IM in searching for the performance target is 1%.

The responses of the architype structures at the performance target, that is, when the maximum inter-story drift reaches 2%, are summarized in Figure 3.1-3.3. All the results are the average of the IDA analysis results for the 26 ground motion records in a suite.

For all the architype structures, the *DCF*s decrease significantly as the stiffness ratio increases. This effect is more significant for weak column-strong beam frames which generally exhibit much higher *DCF*s, that is, severer drift concentration, when there is not a PS wall. The characteristics of the ground motions have an effect on the *DCF*s. For 4- and 8-story structures, pulse like ground motions tend to create larger DCFs in the structures, while it is the opposite for 12-story structures (Figure 3.1).

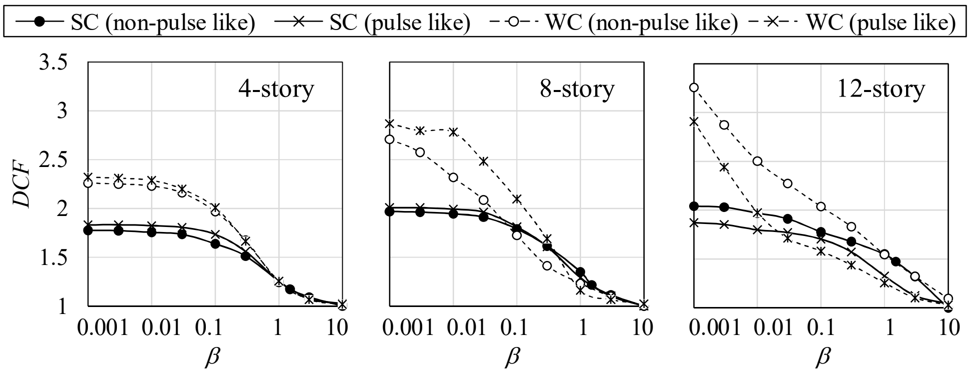

Figure 3.1 *DCF*-stiffness ratio relationship

On the other hand, the plasticity ratio exhibits sudden increases and approaches to 0.5 or above for strong-column (SC) frames as β increases. Weak-column (WC) specimens exhibit much lower plasticity ratios if there is no PS walls, indicating a higher likelihood of localized damage which is usually related to weak-story failure. For both SC and WC frames, pulse like ground motions tend to produce lower plasticity ratios (Figure 3.2).

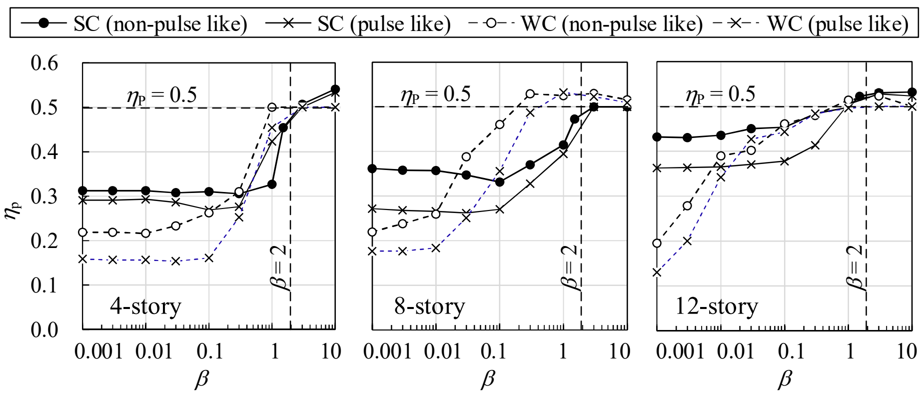

Figure 3.2 Plasticity ratio-stiffness ratio relationship

The plasticity ratio, η_p, is approximately 0.5 no matter if an ideal column hinge or a beam hinge forms in a moment frame. Therefore, η_p of 0.5 is taken as a target for PS wall stiffness so as to ensure a global mechanism to form. According to the results in Figure 3.2, η_p is quite close to 0.5 or even exceeds it when β is 2 for all the architype structures. This can be taken as a reference value when proportioning PS walls in the preliminary design for a multistory moment frame no matter if it is a strong column or a weak column frame. It is also worth noting that the recommended β of 2 is independent of the number of stories. This suggests that the stiffness ratio, β, by its definition, has already taken into account the influence of structural height on the stiffness demands of PS walls.

By creating a more uniform distribution of lateral drift in a moment frame, PS walls motivate more structural elements to contribute to the resistance to earthquakes. In such a manner, the lateral strength of the structure is also increase by PS walls, although the walls themselves do not sustain any lateral resistance if there is not the frame.

This effect is demonstrated in Figure 3.3, in which the lateral strength is represented by the IM, $S_a(T_1, 5\%)$, at the performance target of 2% maximum inter-story drift. In general, the existence of PS walls always increase the lateral strength of the frame, and the increase of lateral strength because of PS walls is much more significant for weak column frames than for strong column ones. Among weak column frames of different number of stories, the increase of lateral strength because of PS walls is more significant for taller buildings.

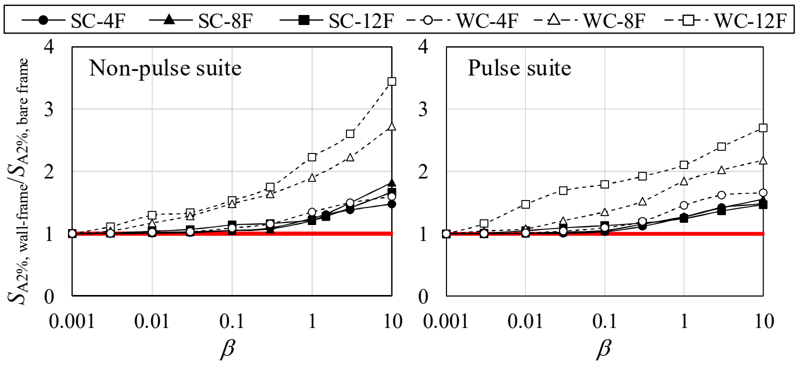

Figure 3.3 Increase of lateral strength as a result of PS walls

5. CONCLUSIONS

To assist the proportioning of pin-supported walls in RC frames in the preliminary design, the relationship between a stiffness ratio for the PS walls and a plasticity ratio for the frames is investigated by incremental dynamic analysis on six multi-story architype frames equipped with PS walls of various stiffness. The following conclusions can be drawn from the analysis results.

(1) The newly-defined stiffness ratio of β provides a consistent measure of the relative stiffness of PS walls to RC frames and inherently takes into account the influence of structural overall height on the stiffness demands for PS walls.
(2) A stiffness ratio, β, of 2 can generally ensure a global plastic mechanism by making the plasticity ratio close to or even exceed 50%. This is valid for both strong column and weak column frames of various numbers of stories.
(3) PS walls can increase the lateral strength of a moment frame, although the walls themselves do not sustain any lateral resistance if there is not the frame.

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