Influence of Axial Compression Ratio on Drift Capacity of RC Columns

Terry Y. P. Yuen[(1)], J. S.Kuang[(2)], V. Kobya[(3)]

[(1)] Research Fellow, Department of Civil and Environmental Engineering, the Hong Kong University of Science and Technology, Hong Kong, People's Republic of China, terryyyp@connect.ust.hk
[(2)] Professor, Department of Civil and Environmental Engineering, the Hong Kong University of Science and Technology, Hong Kong, People's Republic of China, cejkuang@ust.hk
[(3)] PhD Candidate, Department of Civil Engineering, Celal Bayar University, Turkey, v.kobya@gmail.com

…

Abstract

RC columns in modern buildings are often subject to high compression force, which can lead to premature compression failure and reinforcement buckling. To ensure adequate drift capacity can be achieved, several design codes limit the allowed axial force to strength ratio (AFR) specific to a ductility class. Nevertheless, there is no consensus on what are the suitable limits to use in preventing the undesirable failures of the vertical members. Some design codes even do not limit the AFR in design. To resolve this issue, this paper presents a scientific revisit on the AFR effect on RC columns, which is conjugated with a comprehensive statistical analysis of 474 sets of experimental data. It was found that the drift capacity of the columns decreases with increasing AFR. In particular after AFR > 40, increasing amount of confining reinforcement is no longer as effective in enhancing the drift capacity as it is in the low AFR level. Yet, the lateral strength increase with increasing AFR. To guarantee desirable seismic performances of RC columns, the AFR should be limited in the design. The results also suggest that different AFR limits may be used for slender and short RC columns due to different failure mechanisms.

Keywords: axial force ratio; shear column; drift capacity; lateral strength; design codes

1. Introduction

Protecting columns from losing the load-carrying capacity is thus of most important in the capacity design of structures to withstand seismic effects [1–3]. Special design and reinforcement detailing are needed for RC columns to avert severe strength and stiffness degradation during earthquakes. The concrete confinement detailing in critical regions, such as the potential plastic hinges, is one of the most effective means to enhance the member ductility. The amount of confining reinforcement required for achieving specific ductility level is strongly dependent on the imposed axial compression. High axial compression can reduce the ductility capacity as a result of premature concrete crushing, low-cycle fatigue or reinforcement buckling. Therefore, the confinement detailing requirement is often set to be more stringent at higher axial compression levels in order to compensate the reduced ductility capacity.

Some RC design codes, e.g. Eurocode 8 [4], Turkish Earthquake Code [5] Chinese Seismic Code [6] and Hong Kong RC Code [7], even stipulate upper limits on axial force ratio (AFR) - a ratio of nominal axial stress to concrete compressive strength of the section. But not all design codes have the same requirements, for instance, New Zealand [8] and ACI codes [9] do not explicitly prescribe any limits for AFR. The two latter codes only require the maximum axial force does not exceed the design axial capacity, but that is generally less restrictive than the AFR limit requirements. In this regard, this paper revisits the scientific background of the AFR effects and attempts to establish its relationships with the drift capacity and lateral strength. The AFR effects are studied with well-established theoretical and empirical models in conjunction with a comprehensive statistical analysis using 474 sets of column test data. The work presented in this paper would provide reference for the use and determination of a suitable limit of axial force ratio (AFR) for achieving target performances of RC columns in earthquakes.

2. Axial compression effects on drift capacity and strength

High axial compression causes early crushing of concrete and lead to low displacement capacity [2]. It can be illustrated by considering the displacement capacities of the RC members under different axial compression levels. Generally, members with length-to-depth ratio $\alpha_L > 3$ display flexural-controlled behaviour, $\alpha_L \leq 3$ display shear- or flexural-shear controlled behaviour [4,10]. By making the following assumptions: (1) the strain penetration length $L_{sp} = 0.022 f_y d_l$, (2) $L_{sp}^2 / L_s^2 << 1$, and (3) the lower bound value of plastic hinge length $L_p = 2L_{sp}$ [11], the ultimate drifts of the flexural-controlled members can be directly calculated by the moment-curvature analysis that gives

$$\frac{\Delta_u}{L_s} \geq \frac{\zeta \varepsilon_y \alpha_L}{3} + 0.044 f_y d_l \left(\frac{\varepsilon_{cu}}{c} - \frac{2\xi \varepsilon_y}{3l_c} \right) \tag{1}$$

where L_s is the length from the critical section to the point of contra-flexure; l_c is the section depth; ζ is a shape factor, which takes a value of 2.1 for rectangular column sections, and 2.25 for circular column sections [11]; ε_y is the yield strain of steel reinforcement; $\alpha_L = L_s / h_c$ is the column aspect ratio; f_y is the yield strength of the longitudinal reinforcement; d_l is the longitudinal bar size; and c is the depth of natural axis. It can be shown that the depth of natural axis *c* is a monotonic increasing function of axial compression. Therefore, the curvature ductility and the displacement ductility calculated by Eq. (1) decrease with the increase of axial compression.

The lateral strength is

$$v_{\mathbf{u}} = \frac{1}{A_g L_s}\left(-\int_c^d \sigma_c b(x)\cdot(d-c+x)dx + \sum_i \min(f_y\,\mathrm{sgn}(\varepsilon_{si}),\ E_s\varepsilon_{si})\cdot A_i \right) \tag{2}$$

where sgn is sign function: $\mathrm{sgn}(a)=1$ if $a \geq 0$ and $\mathrm{sgn}(a)=-1$ if $a < 0$; $b(x)$ is the width of the section at height x from the bottom reinforcement; ε_{si} is the steel strain of the i th longitudinal reinforcing steel bar of area A_i .

For shear-controlled columns, the ultimate drift can be estimated by the following empirical equation [12]:

$$\frac{\Delta_u}{L_s} = \mathbf{4}\rho_s - \frac{1}{40}\frac{v}{\sqrt{f_c}} - \frac{1}{40}\frac{P}{A_g f_c} + \frac{3}{100} \geq \frac{1}{100} \tag{3}$$

where ρ_s is shear reinforcement ratio, v is nominal shear stress, P is axial force; A_g is the gross cross sectional area. The above equation explicitly includes the effect of AFR by the third term in the middle expression. According to this equation, lower ultimate drifts of shear-controlled columns are resulted in higher AFR as of flexure-controlled columns, though it is also recognised that axial compression can increase the friction on the cracked surfaces. The positive effect of axial compression on the shear strength of RC columns is reflected in the Sezen (2002) model [13] as

$$v_{\mathbf{u}} = k\left[\rho_s f_{yt} + \frac{0.4\sqrt{f_c}}{\alpha_L}\sqrt{\frac{P}{0.5\sqrt{f_c}A_g}} \right] \text{ (shear columns)} \tag{5}$$

where the value of the factor k is a function of displacement ductility μ_Δ as follows

$$k = \begin{cases} \mathbf{1.0} & \mu_\Delta < \mathbf{2} \\ -0.075\mu_\Delta + 1.15 & \mathbf{2} \leq \mu_\Delta < \mathbf{6} \\ \mathbf{0.7} & \mu_\Delta \geq \mathbf{6} \end{cases} \tag{6}$$

The AFR effects on the displacement capacities, as depicted by Eqs. (1) and (3), for flexural columns and shear columns of different percentages of confining hoop reinforcement are illustrated in Figs. 1 (a) and (b) respectively. Similarly, the AFR effects on lateral strength for flexural and shear columns are illustrated in Figs. 2 (a) and (b) respectively. Furthermore, the experimental data presented in Structural Performance Database [14] established by PEER and the University of Washington are used to evaluate the deviations of the calculated results from the actual test results. The comparisons are shown in the Figs. 1 (c) and (d) for ultimate displacement drifts, and Figs. 2 (c) and (d) for lateral strength respectively. It can be seen that effectiveness of confinement to enhance the displacement capacity is reduced by increasing AFR, revealing the need of AFR limits to control the ductility of RC columns. Furthermore, the ultimate drifts of flexural columns, failing in tension failure, decrease rapidly with increase of AFR over the low AFR region as shown in Figs. 3 (a). Subjecting to higher AFR, the rate of drop in ultimate drift becomes more gradual as a result of the change of behaviour from tension failure to compression failure. On the other hand, the Elwood and Moehle model [12] depicts a less rapid drop in ultimate drifts of shear columns until a constant level is reached in high AFR (Fig. 1 (b)).

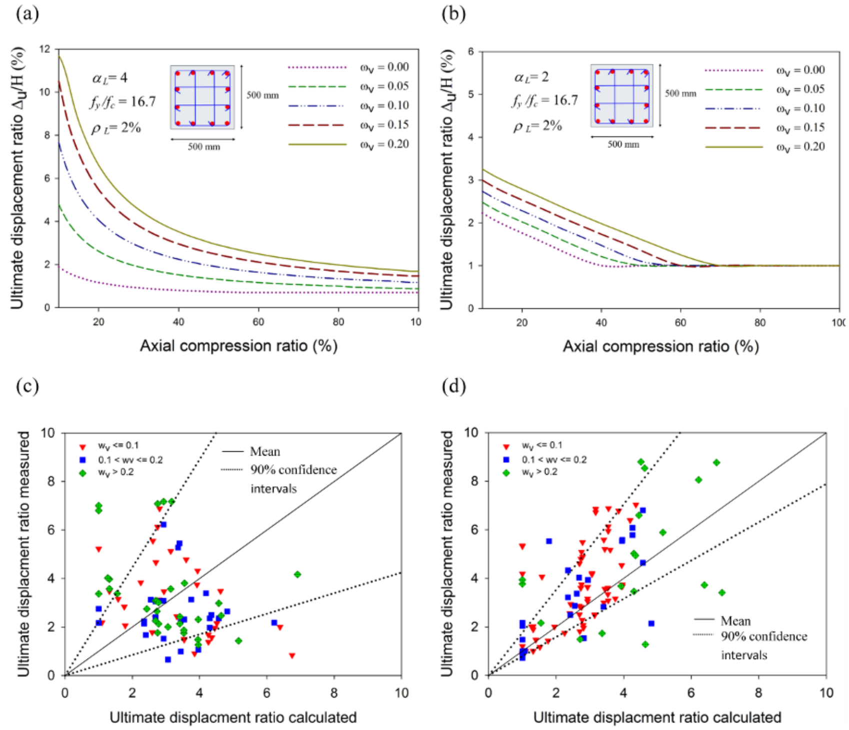


Fig. 1 – Ultimate displacement ratios (UDRs) of RC columns (a) AFR vs UDR of flexural columns; (b) AFR vs UDR of shear columns; (c) calculated results vs experimental results of flexural columns; (d) calculated results vs experimental results of shear columns

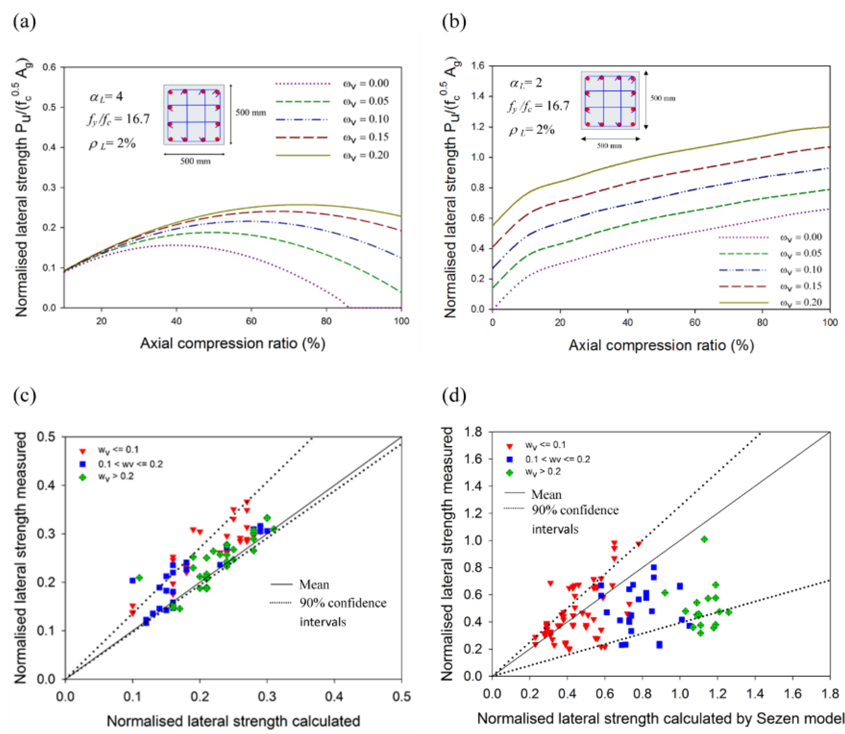


Fig. 2 – Normalised lateral strength (NLS) of RC columns (a) AFR vs NLS of flexural columns; (b) AFR vs NLS of shear columns; (c) calculated results vs experimental results of flexural columns; (d) calculated results vs experimental results of shear columns

As shown in Fig. 2 (a), the lateral strength-AFR relationship of flexural columns shows the typical behaviour of M-N interaction behaviour. In low AFR region, where columns fail by tension failure, the moment capacity and the lateral strength are enhanced by the increase of axial force. However, after the balanced-failure point, compression-failure dictates the column behaviour and further increase of AFR cause reduction in lateral strength. In the case of shear columns, as shown in Fig. 2 (b), the shear strength is directly proportional to the amount of transverse reinforcement provided as predicted by Sezen (2000) shear models. As shown in Fig. 2 (c), a very accurate estimation of the lateral strength of flexural columns can be obtained by simple moment curvature analysis. For the shear columns, as shown in Fig. 2 (d), the Sezen model can give good estimations for lightly reinforced columns but tends to underestimate the shear strength of heavily reinforced columns.

3. Statistical analysis of the axial force effect

The actual effects of axial compression on RC columns are more complicated than the modelled behaviour. Beside the reduction of curvature ductility, high axial compression can also (1) lead to early spalling of the concrete cover at relatively low displacement, which in turn increases the buckling risk of the exposed longitudinal bars [3], (2) exacerbate the P-Δ effect on the columns in particular the slender ones, which is characterised by "crawling" phenomenon occurring in the hysteretic behaviour and leading to collapse [2], and (3) increase of the risk of low cycle fatigue of the columns under seismic motions [15]. In view of this complication behaviour, the quantification of the concrete confinement and axial compression effects has been very much relying on experimental data and this is normally served as a basis for developing code provisions for practical design. A statistical analysis is conducted with a large database of 474 experimental data sets of RC columns, which is composed of those experimental data presented in the Structural Performance Database [14], and also some latest test results by [16–20].

3.1 Ultimate displacement capacity

Displacement capacity is particularly important in determining the seismic performance and collapse probability of RC columns in devastating earthquakes. In this regard, the relationships between ultimate displacement ratio (UDR) against axial compression ratio (AFR) for the tested circular and rectangular RC columns are plotted in Figs. 3 and 4 respectively. The data sets are further divided into three groups of lightly confined columns with mechanical volumetric reinforcing ratio $\omega_v = \rho_v f_y / f_c^{'} \leq 0.1$, moderately confined columns ($0.1 < \omega_v \leq 0.2$), and heavily confined columns ($\omega_v > 0.2$). Figs. 3 and 4 provide solid evidence on how the axial compression and confinement influence the displacement capacities of RC columns. There is a trend of deteriorating displacement ductility and capacity with the increase of axial compression. Below a relatively low level of axial force ratio (≤ 20%), considerable flexural columns can achieve moderate to high UDR (>6%) with moderate confinement ($0.1 < \omega_v \leq 0.2$). And the effect of confinement on improving the structural performances is clearly demonstrated again. At higher axial force ratio (> 30%), the columns can hardly maintain the same level of ductility as in under low axial force ratio and large amount of confinement is needed in order to achieve moderate to high UDRs. Above axial force ratio of 40%, the UDRs of most columns cannot even be higher than 2%. There is a similar trend for shear columns that the UDRs are diminishing with increasing AFR, regardless of the amount of confinement reinforcement being provided.

Nevertheless, the ultimate drift of short columns is less influenced by AFR and shows a more graduate decline with increasing AFR when compared with the case in flexural columns. The Elwood and Moehle model [12] sets a lower bound for the ultimate displacement ratio of shear columns to be 1%, but it can be seen from Fig. 4 (a) that some columns have values of as low as 0.71%. It is therefore suggested that the lower bound for ultimate displacement ratio of shear columns should be set at 0.7%. Furthermore, it is known that the axial compression has both adverse and beneficial effects on the shear-controlled RC members. The benefits of axial compression become predominant for RC members with very low aspect ratios. It was shown in [21] for very short RC walls with aspect ratio less than 1.5, the trend for the ultimate displacement-AFR relationship is even completely reversed, due to the fact that the shear transfer by arch action and sliding resistance of cracks in the shear-controlled RC members can be enhanced by axial compression.

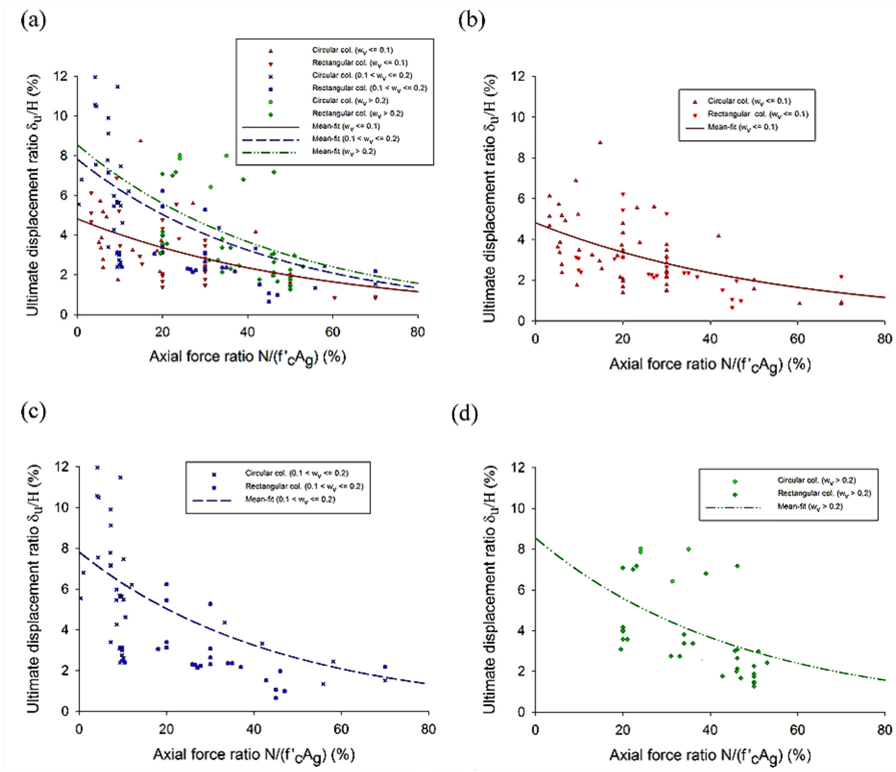


Fig. 3 – AFR vs UDR of flexural columns ($\alpha_L > 3$) (a) all flexural columns; (b) lightly confined columns ($\omega_v \leq 0.1$); (c) moderately confined ($0.1 < \omega_v \leq 0.2$); (d) heavily confined ($\omega_v > 0.2$)

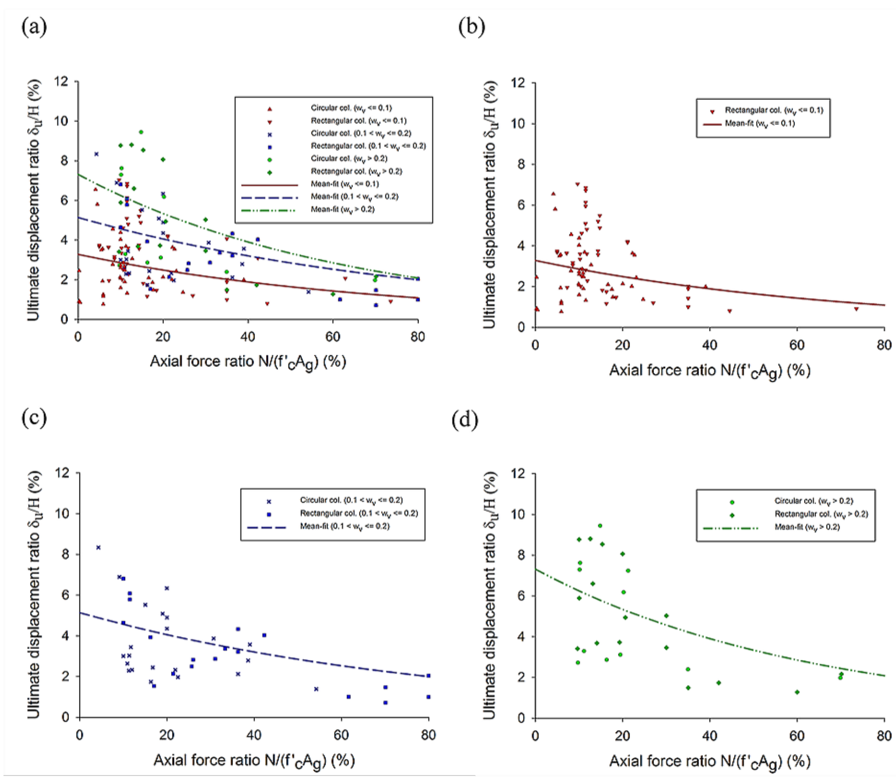


Fig. 4 – UDR vs AFR of short columns ($\alpha_L \leq 3$) (a) all short columns; (b) lightly confined columns ($\omega_v \leq 0.1$); (c) moderately confined ($0.1 < \omega_v \leq 0.2$); (d) heavily confined ($\omega_v > 0.2$)

3.2 Lateral strength

Relationships between the shear strength normalized with respect to $f_c^{0.5} A_g$ and AFR are plotted in Figs. 5 and 6 for flexural and shear columns respectively. Although the analytical behaviour for flexural columns (Fig. 2 (a)) cannot be clearly observed in the statistical results as shown in Figure 7, the calculated normalised shear strength by the moment-curvature analysis (Eq. (2)) is bounded below the average experimental results i.e. $\mathbf{V}_u / f_c^{0.5} A_g = 0.16\text{-}0.2$ when AFR is less than 40% and thus can gives reasonable estimation over this region of low to moderate AFR. Higher amount of confining reinforcement does not result in significant increase in the lateral strength of the columns. This is because the strength of flexural columns is largely dependent on the amount of longitudinal reinforcement. Confining reinforcement primary enhance the concrete ductility but only increase slightly the concrete strength. Hence, the lateral strength is not largely affected by the amount of confining reinforcement used in the columns. On the contrary, higher AFRs lead to noticeable increase in ultimate lateral strength of short columns as shown in Fig. 6, which is as much as double the attainable lateral strength of the flexural columns at the same level of AFR. High axial compression can enhance the friction on the shear crack surfaces, which in turn enhance the overall lateral strength of the short columns controlled by shear deformation. The Sezen (2002) model is able to predict this behaviour correctly (Fig. 4 (b)). Unlike the flexural columns, the strength of shear columns is largely dependent on the amount of transverse reinforcement. Therefore, it can be seen that with greater amount of confining reinforcement, the lateral strength of the shear columns is also noticeably higher. Nevertheless, such increase in lateral strength generally cannot compensate the adverse effect on reduction in displacement capacity due to high axial compression, which generally dictates the collapse probability of buildings under seismic loading.

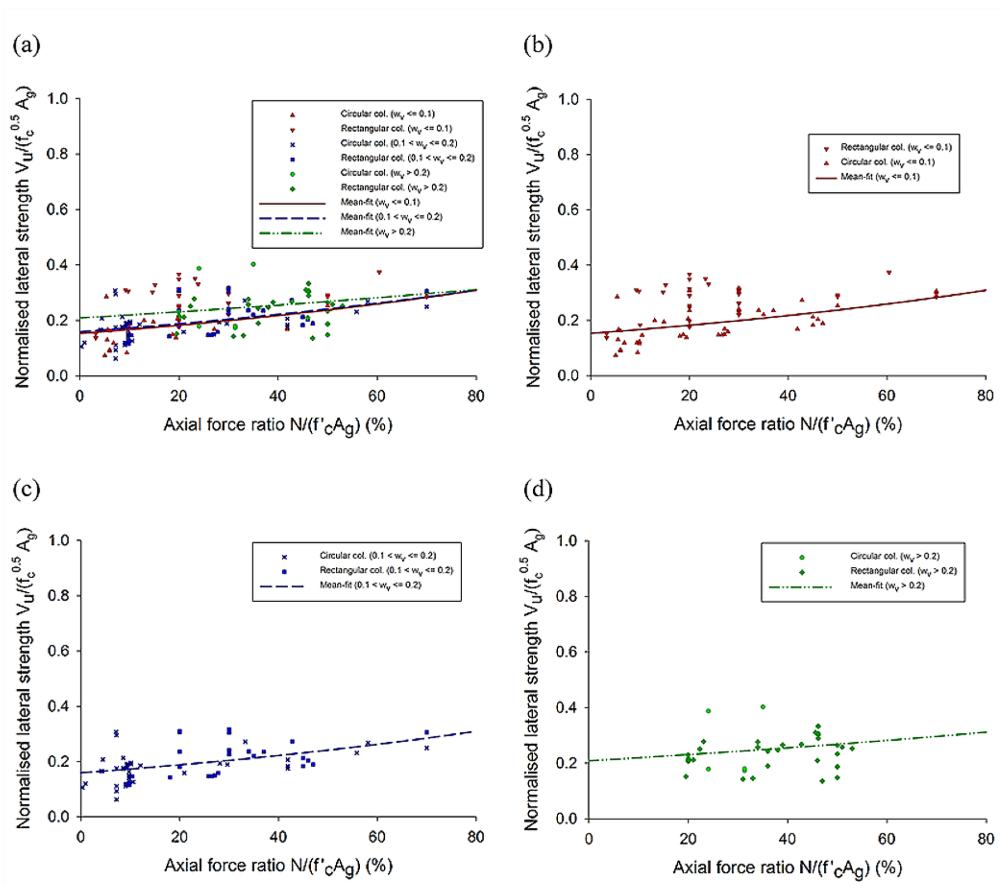


Fig. 5 – NLS vs AFR of flexural columns ($\alpha_L > 3$) (a) all flexural columns; (b) lightly confined columns ($\omega_v \leq 0.1$); (c) moderately confined ($0.1 < \omega_v \leq 0.2$); (d) heavily confined ($\omega_v > 0.2$)

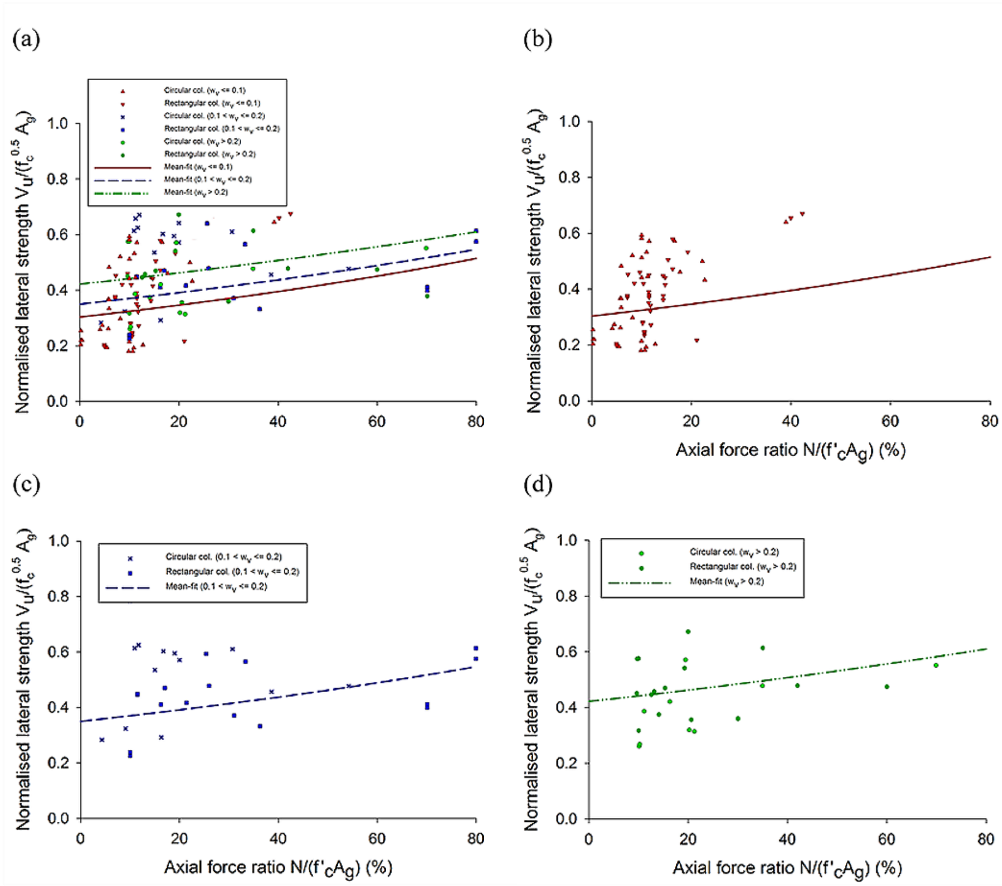


Fig. 6 – NLS vs AFR of shear columns ($\alpha_L \leq 3$) (a) all shear columns; (b) lightly confined columns ($\omega_v \leq 0.1$); (c) moderately confined ($0.1 < \omega_v \leq 0.2$); (d) heavily confined ($\omega_v > 0.2$)

4. Comparisons of the code stipulated AFR limits

The stipulated limits on the axial compression ratios in various design codes are compared in adjacent to the statistical analysis presented before, which has related the displacement capacity and axial compression ratio. Since the denominators in the axial compression ratios defined by various design codes are generally different, they are firstly re-normalised with respect to the specified cylindrical compressive strength of concrete as presented in Table 1, and the renormalized limits are plotted together with the statistical data in Fig. 7. Fig. 7(a) presents the plot of displacement ductility of flexural columns against AFR, while Fig. 7(b) presents the plot of ultimate displacement ratio of shear columns against AFR. Two different types of plots are used because, unlike flexural columns, the displacement ductility or the yield drift of shear columns is difficult to define properly and the ultimate displacement ratio (UDR) is thus more widely accepted to represent their structural performances.

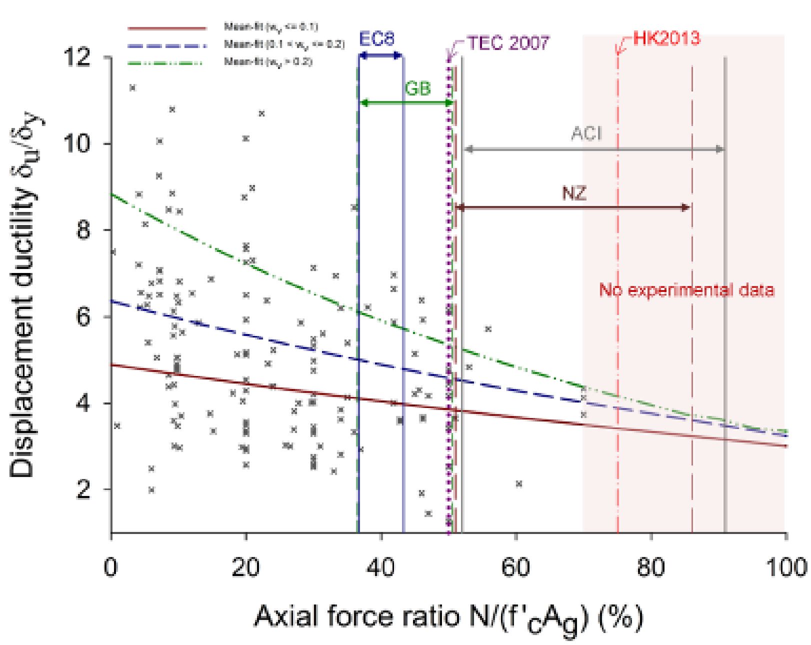
As shown in Fig. 7, the upper bounds of the NZ and ACI codes limits and the HK code's apparent limit on axial compression ratio, though not explicitly stipulated, fall in a region where the predicted column behaviour is unsupported by any experimental data. Therefore, these limits cannot really guarantee desirable column behaviour as designed during earthquakes. In EC8, for multi-storey frames with fundamental period T_1 larger than the corner period T_c, ductility factor μ_δ is equal to the behaviour factor q, of which basic values are 3.6 and 5.4 for DCM and DCH respectively. As seen in Fig. 7, EC8 provisions satisfy the target levels of ductility for flexural columns, and also ensure that the ultimate drift ratio of shear columns would not be lower than 2%, provided that column hinges are properly detailed. The TEC 2007 and GB codes also give reasonable limits on AFR.

Table 1 – Codes provisions on AFR limits for RC column design

	NZ	HK(1)	GB50011	EC8	TEC 2007	ACI
Definition of AFR	$\frac{N_o^*}{\phi N_{n,\max}}$ (2)	$\frac{N}{f_{cu} A_c}$	$\frac{N_{C,C}}{f_c A_g}$	$\frac{N_{ED,EC}}{f_{cd} A_c}$	$\frac{N_{d\max}}{f_{ck} A_g}$	$\frac{P_u}{P_o}$ (2)
Upper limit(s)	0.7	0.6	$\begin{cases} 0.65 & \text{(Gr I)} \\ 0.75 & \text{(Gr II)} \\ 0.85 & \text{(Gr III)} \\ 0.90 & \text{(Gr IV)} \end{cases}$	$\begin{cases} 0.55 & \text{(DCH)} \\ 0.65 & \text{(DCM)} \\ - & \text{(DCL)} \end{cases}$	0.5	$\begin{cases} 0.80\phi & \text{(Ties)} \\ 0.85\phi & \text{(Spirals)} \end{cases}$ (3)
Dependent on longitudinal reinforcement	○	×	×	×	×	○
Re-normalised AFR limit(s) vs f_c'	$\frac{N_o^*}{f_c' A_g}$	$\frac{N}{f_c' A_c}$	$\frac{N_{C,C}}{f_c' A_g}$	$\frac{N_{ED,EC}}{f_c' A_g}$	$\frac{N_{d\max}}{f_c' A_g}$	$\frac{P_u}{f_c' A_g}$
Re-normalised limit(s)	$0.7\phi[(\alpha_1 + m)\rho_L + \alpha_1]$ (4)	0.75	$\begin{cases} 0.37 & \text{(Gr I)} \\ 0.42 & \text{(Gr II)} \\ 0.48 & \text{(Gr III)} \\ 0.51 & \text{(Gr IV)} \end{cases}$	$\begin{cases} 0.37 & \text{(DCH)} \\ 0.43 & \text{(DCM)} \\ - & \text{(DCL)} \end{cases}$	0.5	$\begin{cases} 0.80\phi[(0.85+m)\rho_L + 0.85] & \text{(Ties)} \\ 0.85\phi[(0.85+m)\rho_L + 0.85] & \text{(Spirals)} \end{cases}$ (4)

(1) Assumed limit not explicitly stipulated by HKCocrete2013.
(2) $P_o = 0.85 f_c'(A_g - A_{st}) + f_y A_{st}$ and $N_{n,max} = \alpha_1 f_c'(A_g - A_{st}) + f_y A_{st}$ are the nominal axial strength of columns specified by ACI 318-14 and NZ 3101:2006 respectively.
(3) The ACI limits are typical axial load-and-resistance design requirement and have no additional requirements for seismically designed columns.
(4) Limits in ACI and NZ are dependent on the values of strength reduction factor ϕ, longitudinal reinforcement ratio ρ_L, and steel yield strength to concrete compressive strength ratio $m = f_y / f_c'$. If the typical values of 0.85 and 14 are assumed for α_1 and m respectively, the limits are ranged from 0.51-0.86 corresponding to the allowable range of longitudinal reinforcement ratio (0.8-4%) with ϕ = 0.85 for NZ, and 0.52-0.91 (ties) and 0.60-1.04 (spirals) corresponding to the allowable range of longitudinal reinforcement ratio (1-6%) with ϕ = 0.65 (ties) or 0.75 (spirals) for ACI.

(a)



(b)

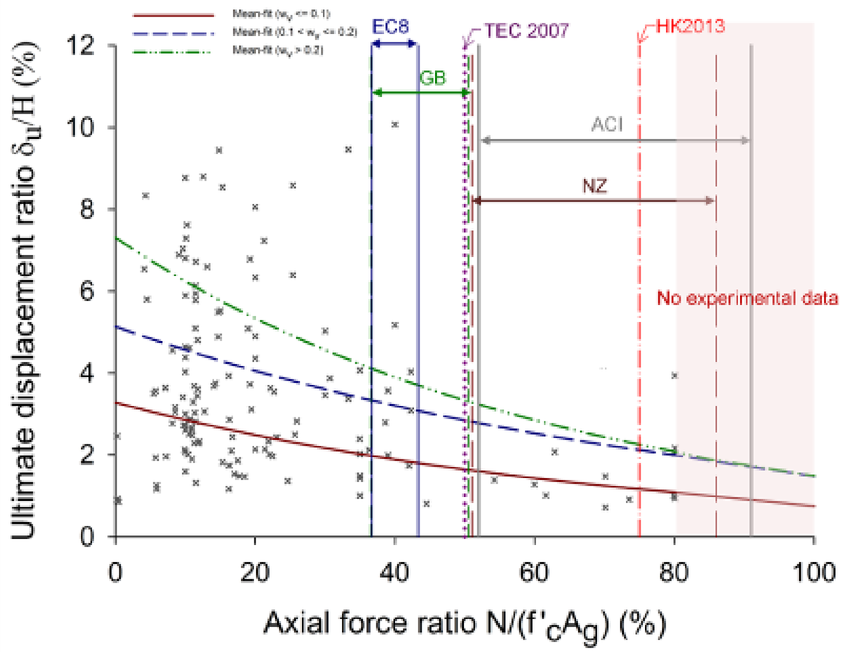


Fig. 7 – Codes specified axial compression ratio limits (a) displacement ductility μ_Δ of flexural columns ($\alpha_L > 3$); (b) ultimate displacement ratio (UDR) of shear columns ($\alpha_L \leq 3$)

5. Conclusion

Structural columns, as a primary and critical load-carrying member, should be designed and detailed to sufficient structural ductility in order to reduce the risk of the catastrophic collapse of the whole structure. In modern seismic design paradigm, the ductility and energy dissipation capacity of RC columns are enhanced by concrete confinement in the potential plastic hinge regions with hoop reinforcement. The required quantity of confining reinforcement is largely dependent on the level of axial compression induced onto the columns. The effectiveness of typical detailing for concrete confinement is often deteriorating with increase of axial compression and therefore many modern seismic design codes also stipulate upper limits on axial compression ratio.

Nevertheless, there considerable dissimilarities in the provisions on the confinement detailing and axial compression limits stipulated in various design codes. In view of this issue, this paper revisited the scientific background of confinement detailing for concrete structures, and then presents a comprehensive statistical analysis is conducted using the 474 experimental data sets of various types of RC columns. It is found that the drift capacity of the RC columns under cyclic loading can be deprived by increasing axial compression ratio, though the lateral strength can be enhanced by increasing axial compression. To prevent undesirable failure mechanisms such as premature compression failure and reinforcement buckling, axial compression ratio should be limited in the columns design. EC8 set reasonable AFR limits for RC columns designed to different ductility classes, yet it was shown that different limits may be used for slender and short columns as they are controlled by different failure mechanisms. The NZ and ACI code requirements result in a large range of the axial force limits dependent on the longitudinal reinforcing ratio, and are much less stringent in comparison with the EC8, TEC 2007 and GB codes. The NZ and ACI codes allow AFR as high as 0.9 for heavily reinforced columns but there is even no experimental data to guarantee the predicted behaviour.

In a region of moderate to high seismicity, the structural member should be designed to at least with ductility $\mu_{\Delta} \geq 4.0$. For typical flexural-controlled columns with confinement $\omega_v > 0.1$, a suitable AFR limit can be taken as ≤ 0.50. Given the evidently poor seismic performances, shear-controlled columns should be avoided. If shear columns are inevitable, the ultimate drift capacity shall not be less than 0.02% of the column height and the AFR limit is tightened up to ≤ 0.40 in order to guarantee life-safe performance level.

6. Acknowledgement

The supports of the Scientific and Technological Research Council of Turkey (TÜBİTAK) under project number 214M236 and the Hong Kong Research Grand Council (HK-RGC) under grant number 614011 are gratefully acknowledged.

7. References

[1] Browning J, Li Y (2000): Performance assessment for a reinforced concrete frame building. *Earthquake Spectra*, **16** (3), 541–555.

[2] Paulay T, Priestley MJN (1992): *Seismic Design of Reinforced Concrete and Masonry Buildings*. John Wiley & Sons, New York.

[3] Kappos A, Penelis G (1996): *Earthquake Resistant Concrete Structures*. CRC Press, Boca Raton.

[4] CEN (2004): *Eurocode 8: Design of Structures for Earthquake Resistance. Part 1: General Rules, Seismic Actions and Rules for Buildings* (EN 1998–1:2004). European Committee for Standardization, Brussels.

[5] TEC (2007): *Specifications for Structures to be Built in Disaster Areas*. Ministry of Public Works and Settlement, Government of the Republic of Turkey.

[6] National Standard of the People’s Republic of China (2010): *Code for Seismic Design of Buildings* (GB 50011-2010). Beijing: China Architecture & Building Press.

[7] HKBD (2013): *Code of Practice for Structural Use of Concrete 2013*. Hong Kong: The HKSAR Government.

[8] SNZ (2006): *Concrete Structure Standard-The Design of Concrete Structures Incorporating Amendment No.1 & 2* (NZS 3101:Part 1:2006-A1&A2). vol. 2. Wellington, New Zealand: SNZ.

[9] American Concrete Institute (2014). *Building code requirements for structural concrete (ACI 318–14) and Commentary*. Farmington Hills, MI: ACI.

[10] Elwood KJ, Matamoros AB, Wallace JW, Lehman DE, Heintz JA, Mitchell AD, et al (2007): Update to ASCE/SEI 41 Concrete Provisions. *Earthquake Spectra*, 23, 493–523.

[11] Priestley M, Calvi G, Kowalsky M (2007): *Displacement-Based Seismic Design of Structures*. IUSS Press, Pavia, Italy.

[12] Elwood KJ, Moehle JP (2008): Dynamic collapse analysis for a reinforced concrete frame sustaining shear and axial failures. *Earthquake Engineering & Structural Dynamics*, **37** (7), 991–1012.

[13] Sezen H (2002): *Seismic Response and Modeling of Lightly Reinforced Concrete Building Columns*. University of California, Berkeley.

[14] Berry M, Parrish M, Eberhard M (2004): *PEER Structural Performance Database User’s Manual*. PEER Research Report. Berkeley, CA: University of California.

[15] Borg RC, Rossetto T (2012): Low Cycle Fatigue Tests of Reinforced Concrete Columns and Joints, Built with Ribbed Reinforcement and Plain Stirrups. *15th World Conference on Earthquake Engineering*, Lisbon Portugal.

[16] Ahn J. M., Shin S. W (2007): An evaluation of ductility of high-strength reinforced concrete columns subjected to reverse cyclic loads under axial compression. *Magazine of Concrete Research*, **59** (1), 29–44.

[17] Rodrigues H, Arêde A, Varum H, Costa AG (2013): Experimental evaluation of rectangular reinforced concrete column behaviour under biaxial cyclic loading. *Earthquake Engineering & Structural Dynamics*, **42** (2), 239–259.

[18] Kim T, Kim Y, Shin HM (2007): Seismic performance assessment of reinforced concrete bridge columns under variable axial load. *Magazine of Concrete Researc*h, **59** (2), 87–96.

[19] Sinha S, Roy L (2004): Fundamental tests of reinforced concrete columns subjected to seismic loading. *Proceedings of the ICE - Structures and Buildings*, **157** (3), 185–199.

[20] Lee TK, Chen CC, Pan ADE, Hsiue KY, Tsai WM, Hwa K (2013): Experimental evaluation of large circular RC columns under pure compression. *Structural Concrete*, **14** (1): 60–68.

[21] Yuen YP, Kuang JS (2015): Effect of axial compression on ductility design of RC walls. *Proceedings of the ICE - Structures and Buildings*, **168** (8), 554–569.