Evaluation of New Zealand code requirements related to instability failure of structural walls

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ABSTRACT: Observations of wall instability in the 2010 Chile earthquake and in the 2011 Christchurch earthquake resulted in concerns about the current design provisions of structural walls. This mode of failure was previously observed in the experimental response of some wall specimens subjected to in-plane loading. Therefore, the postulations proposed for prediction of the limit states corresponding to out-of-plane instability of rectangular walls are generally based on stability analysis under in-plane loading only. These approaches address stability of a cracked wall section when subjected to compression, thereby considering the level of residual strain developed in the reinforcement as the parameter that prevents timely crack closure of the wall section and induces stability failure. The New Zealand code requirements addressing the out-of-plane instability of structural walls are based on the assumptions used in the literature and the analytical methods proposed for mathematical determination of the critical strain values, such as the height of the wall involved in development of the out-of-plane deformation. In this study, the analytical methods proposed for prediction of this critical level of strain are compared with experimental measurements of a wall specimen that exhibited out-of-plane instability as the only and main failure pattern. The specimen was half-scale model of the first story of a four-story prototype wall which was designed according to the latest version of the New Zealand concrete design standard (NZS3101:2006) and was subjected to in-plane quasi-static cyclic loading.

1 INTRODUCTION

According to the observations of the recent earthquakes in Chile and New Zealand, lateral instability of a large portion of a wall section (out-of-plane buckling) was one of the failure patterns that raised concerns about performance of buildings designed using modern codes (Figure 1). Prior to the Chile earthquake, this failure mechanism had only been primarily observed in laboratory tests (Oesterle 1979, Vallenas et al. 1979, Goodsir 1985, Thomsen IV and Wallace 2004, Johnson 2010) (Figure 2).

Paulay and Priestley (1993) made recommendations for the prediction of the onset of out-of-plane instability, based on the observed response in tests of rectangular structural walls and theoretical considerations of fundamental structural behaviour. Because of the very limited available experimental evidences, engineering judgement was relied on extensively. The major source of the instability was postulated to be the previously experienced (maximum) tensile strain rather than the maximum compression strain.

The out-of-plane instability of rectangular reinforced concrete walls under in-plane loading has been mainly investigated by idealizing the boundary region of the wall as an axially loaded column. For this purpose, RC prism units were subjected to tension and compression cyclic loading. This type of research on out-of-plane instability failure was first conducted by Goodsir (1985) and the main finding was the effect of the maximum tensile strain reached in the reinforcement on development of out-of-plane deformations. Chai and Elayer (1999) also conducted an experimental study to examine the out-of-plane instability of several RC columns that were designed to represent the end-regions of a ductile planar RC wall under large amplitude reversed cyclic tension and compression.
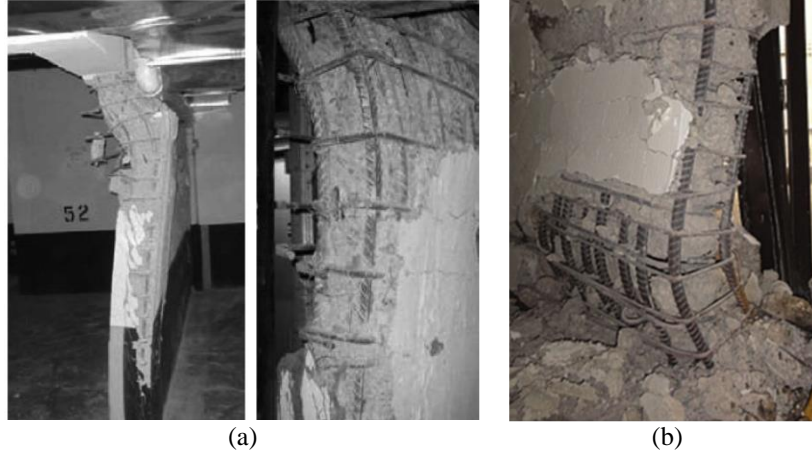
<div align="center">(a)   (b)</div>

Figure 1. Out-of-plane instability of rectangular walls: (a) 2010 Chile earthquake (Wallace 2012); (b) 2011 Christchurch earthquake (Elwood 2013)

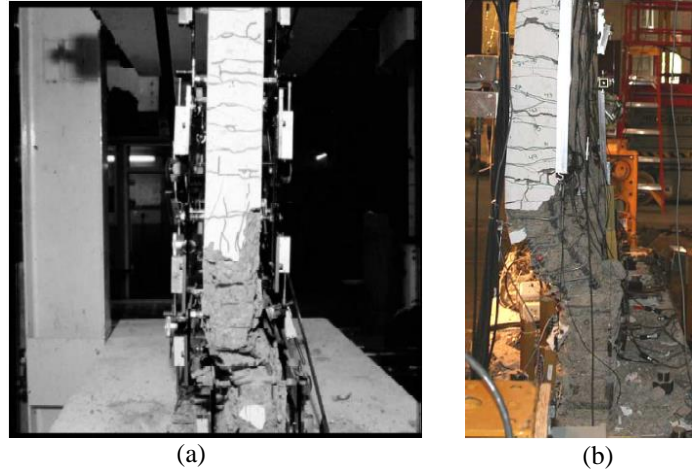<div align="center">(a)   (b)</div>

Figure 2. Wall Out-of-plane instability in laboratory tests: (a) Goodsir (1985); (b) Johnson (2010)

In this study, the analytical methods proposed for prediction of out-of-plane instability in rectangular structural walls are compared with experimental measurements of a wall specimen that exhibited this mode of failure as the only and main failure pattern.

2 ANALYTICAL PREDICTIONS OF OUT-OF-PLANE DEFORMATION

Paulay and Priestley (1993) scrutinized the mechanism of out-of-plane instability by idealization of the part of the wall height that has undergone out-of-plane deformation with a circular shape. By expressing the lateral displacement δ in terms of the wall thickness b, i.e., $\delta = \xi b$, and using expressions developed for estimation of the radius of curvature, the eccentricity ratio ξ was calculated as:

$$\xi = \frac{\varepsilon_{sm}}{8\beta}\left(\frac{l_o}{b}\right)^2 \tag{1}$$

where

ε_{sm} = the maximum tensile strain (the relatively small elastic recovery was neglected and the residual strain was assumed to be of the order of ε_{sm})

l_o = the height along which out-of-plane instability develops, assumed to be equal to the theoretical length of the plastic hinge

βb = the distance from the layer of elastic reinforcement to the point of initial crack closure

In order to establish a stability criterion for the section undergoing out-of-plane deformations, a section equilibrium approach was used (Figure 3).
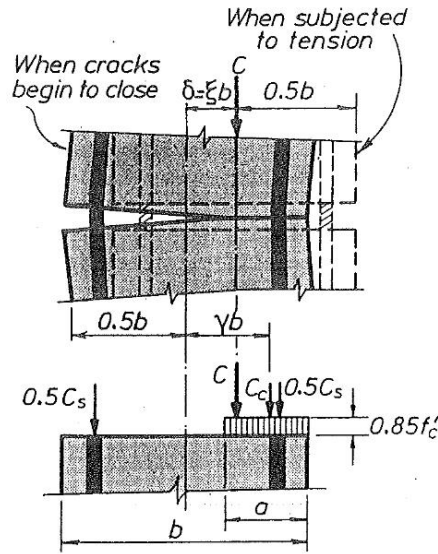

Figure 3. Relation of internal forces to eccentricity (Paulay and Priestley 1993)

According to Paulay and Priestley (1993), out-of-plane instability of the section will occur if the lateral displacement exceeds half of the wall thickness. The equilibrium of the section shows that the compression force (C) applied with an eccentricity of $\delta = \xi b$ is sustained by the compressive actions of the reinforcement as well as some concrete compression force C_c within the crack closure area. It is assumed that all bars develop yield stress f_y at partial crack closure although the two layers of reinforcement would undergo different values of compressive strains. The equilibrium equations,

$$C = C_c + C_s \tag{2}$$

and

$$C_c = (\xi/\gamma)C \tag{3}$$

can be rearranged as

$$C = C_s/\left(1 - \frac{\xi}{\gamma}\right) \tag{4}$$

If C_s is replaced by its maximum value for a unit length of the wall $C_s = \rho b f_y$:

$$C_c = \rho b f_y/(\gamma/\xi - 1) \tag{5}$$

Increasing reinforcement content in the end region of the wall section would increase both the total and the concrete compression force resulting in the lever arm γb being smaller and occurrence of instability at a lower eccentricity ξb.

Considering the assumption of the equivalent rectangular compression stress block:

$$\gamma = \frac{1}{2}(1 - C_c/(0.85f'_c b) \tag{6}$$

And substituting from Equation 5 into Equation 6:

$$\gamma = \frac{1}{2}\left[(\xi + 0.5) - \sqrt{(\xi + 0.5)^2 - 2\xi(1 + 1.176m)}\right] \tag{7}$$

where $m = \rho f_y/f'_c$

As the term inside the square root sign needs to be non-negative, the stability criterion of the wall section was derived as

$$\xi \leq 0.5(1 + 2.35m - \sqrt{5.53m^2 + 4.70m}) \tag{8}$$

The upper bound limit for ξ was considered as 0.5, i.e., if the out-of-plane displacement exceeds half of the wall thickness, the instability of the section would be inevitable.

Minimum wall thickness can be calculated as

$$b_c = l_o \sqrt{\frac{\varepsilon_{sm}}{8\beta\xi}} \tag{9}$$

Chai and Elayer (1999) used the same stability criterion as Equation 8 and considered three components for ε_{sm} as:

$$\varepsilon_{sm} = \varepsilon_e + \varepsilon_r + \varepsilon_a^* \tag{10}$$

$$\varepsilon_{sm} = \eta_1\varepsilon_y + \eta_2\varepsilon_y + \varepsilon_a^* \tag{11}$$

1) ε_e = an elastic strain recovery for the unloading from a tensile excursion;

2) ε_r = a reloading strain associated with compression yielding of the reinforcement (and depends on the cyclic characteristic of the reinforcing steel since a reduced stiffness in the steel is expected due to the Bauchinger'strain effect)

3) ε_a^* = an axial strain at first closure of cracks

Based on experimental observations on the response of axially loaded reinforced concrete columns that represented boundary regions of rectangular walls, η_1 was defined to be close to 1.5 and η_2 in the range of 3 to 5.

Based on the relationship of the transverse curvature at mid-height of the column with the mid-height out-of-plane displacement and axial strain corresponding to the first crack closure, the following kinematic relation was derived:

$$\varepsilon_a^* = \left(\frac{1}{2c}\right)\left(\frac{b}{l_o}\right)^2 \xi_m \tag{12}$$

Where c depends on the transverse curvature distribution of the column and ξ_m is the out-of-plane displacement at mid-height of the column, normalized by the wall thickness.

The following assumptions were made:

- The out-of-plane displacement for the crushing limit state was assumed to be fairly close to the out-of-plane displacement at first crack closure.

- The limit state for calculation of the out-of-plane displacement was concrete crushing. i.e. ξ is the out-of-plane displacement corresponding to the concrete crushing and the out-of-plane displacement should be limited to ξ.

- $\eta_1 = 1.0$, and $\eta_2 = 2.0$

- The curvature distribution was considered sinusoidal, i.e., coefficient $c = 1/\pi^2$

Based on these assumptions, the maximum tensile strain that may be imposed on the column was written as

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$$\varepsilon_{sm} = \frac{\pi^2}{2}\left(\frac{b}{l_o}\right)^2 \xi + 3\varepsilon_y \qquad (13)$$

3 EXPERIMENTAL OBSERVATIONS

The experimental observations of a wall specimen tested as part of an experimental campaign investigating the key parameters governing out-of-plane instability of rectangular walls has been used in this study to evaluate the analytical models proposed for prediction of this mode of failure. Details of the test program are presented in Dashti et al. (2017). As the out-of-plane instability was the only failure pattern of this specimen, the assumptions used in the literature for the mathematical determination of the critical strain values, such as the height of the wall involved in development of the out-of-plane instability could be investigated using the observations of this experimental program. The specimen was half-scale (2.0 m high, 1.6 m long and 125 mm thick), representing the first story of a four story prototype wall designed according to NZS3101:2006.

Out-of-plane displacement initiated during the 1.5% drift cycle and increase in the following drift cycles. Figure 4 displays the out-of-plane displacement profile of one of the specimen boundary regions corresponding to different drift cycles and Figure 5 displays the average strain measurements along this boundary region. As discussed in Section 2, the maximum level of strain developed in a specific cycle is recognised as the main parameter triggering formation of out-of-plane deformation in rectangular walls. The stability criterion proposed by Paulay and Priestley (1993), given by Equation 8, is calculated for the specimen and indicated in Figure 4. During the 2.5% drift cycle, the maximum out-of-plane displacement exceeded this stability criterion. In a specific cycle, out-of-plane displacement initiates when the specimen is unloaded from the peak displacement and is being reloaded in the opposite direction. The out-of-plane displacement generally reaches its peak value when unloading from the peak drift and going back through the original undeformed position (0.0% drift). This out-of-plane displacement recovers substantially while approaching the peak displacement in the opposite direction. However, this recovery was not very considerable during the 2.5% drift level (about 40% and 66% in the west and east boundary regions, respectively)and a significant amount of residual out-of-plane displacement was formed in the boundary region. With this amount of residual out-of-plane displacement, if the loading had continued in the opposite direction, the specimen might have become unstable due to excessive compressive actions accompanied by P-delta effects generated by this residual eccentricity. Therefore, the proposed stability criterion could be a conservative limit state for prevention of out-of-plane displacement from reaching a critical value. During the 3.0% drift cycle, the out-of-plane displacement reached a value equivalent to 55% of the wall thickness ($\xi = 0.55$) during the unloading at around 0.0% drift level, which not only did not recover, but also increased steadily and turned into out-of-plane instability. Figure 4 shows the maximum out-of-plane displacement measurement during the 3.0% drift cycle.

Paulay and Priestley (1993) and Chai and Elayer (1999) proposed Equation 1 and Equation 13, respectively, as relationships between the maximum tensile strain ε_{sm} over l_o and the normalized out-of-plane displacement ξ. The plastic hinge length, l_p (given by Equation 14), was postulated to be a reasonable approximation of the potential length of the wall over which out-of-plane buckling may occur, l_o.

$$l_p = 0.2l_w + 0.044h_w \qquad (14)$$

Where

l_w = horizontal length of the wall section

h_w = full height of the cantilever wall

The plastic hinge length calculated for the tested specimen using Equation 14 is 584mm resulting in $\varepsilon_{sm} = 0.042$, and $\varepsilon_{sm} = 0.041$ from Equation 1 and Equation 13, respectively. The strain measurements along the boundary region (Figure 5) indicate that the strain corresponding to the maximum out-of-plane displacement at 2.5% drift cycle, which is slightly higher than the stability criterion, is around 0.017. The two approaches give fairly close and considerably large values for the

maximum tensile strain corresponding to the stability criterion calculated using Equation 8.
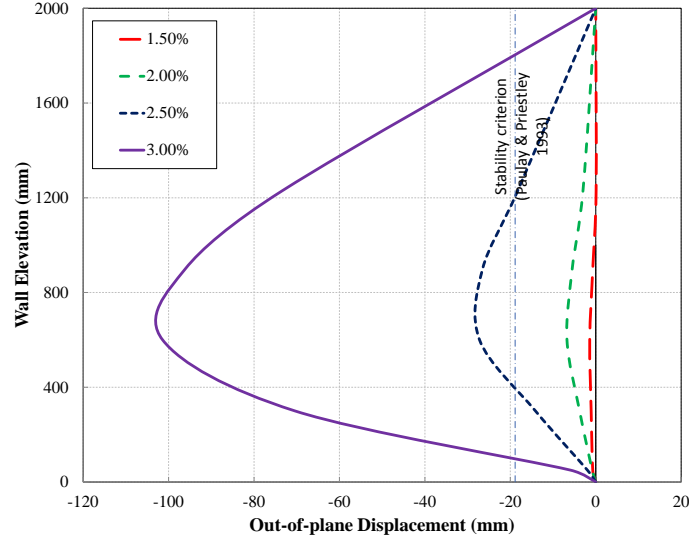

Figure 4. Out-of-plane displacement along the height of the west boundary during different drift cycles
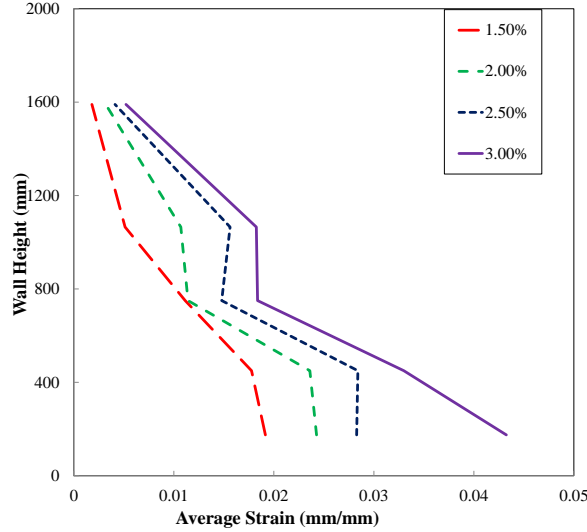

Figure 5. Average strain along the height of the west boundary at different drift levels

The height of the wall involved in the formation of out-of-plane buckling was measured at different drift levels using the crack pattern along the wall height and across the wall thickness. This height was measured to be 1430 mm at 2.5% drift level. Using the value of $l_o = 1430$, the maximum tensile strain is calculated as $\varepsilon_{sm} = 0.007$, and $\varepsilon_{sm} = 0.013$ from Equation 1 and Equation 13, respectively. To compare these estimations with the test measurements, the value of the out-of-plane displacement at 2.5% drift level ($27.5\ mm$, Figure 4) is used as the stability criterion and the corresponding tensile strains are calculated as $\varepsilon_{sm} = 0.01$, and $\varepsilon_{sm} = 0.015$ from Equation 1 and Equation 13, respectively. The values predicted by Equation 1 and Equation 13 are conservative approximations of the maximum tensile strain corresponding to the out-of-plane displacement at 2.5% drift level (0.017, Figure 5), which is fairly close to and slightly after the stability criterion developed by Paulay and Priestley (1993). The value of the buckling height at 2.5% drift level, $l_o = 1430$, is about 70% of the wall height.

The inconsistency between assumption of $l_o = l_p$ and the experimental measurements of l_o has been observed by Rosso et al. (2015) and Johnson (2010), as well. The value of l_o was identified as 75% of the wall unsupported height for two singly reinforced wall specimens tested by Rosso et al. (2015).

4   NEW ZEALAND CODE REQUIREMENTS

The out-of-plane buckling of slender walls is addressed in Section 11.4.2 (Dimensional Limitations) of the New Zealand standard (NZS3101 2006).

For walls with axial force levels greater than $0.05f_c'A_g$ and for ductile or limited ductile plastic region the thickness in the boundary region of the wall section, extending over the lesser of the plastic hinge length or the full height of the first storey, shall not be less than:

$$b_m = \frac{\alpha_r k_m \beta (A_r + 2) L_w}{1700\sqrt{\xi}} \tag{15}$$

where

α_r=1.0 for doubly reinforced walls and 1.25 for singly reinforced walls; and

$\beta = 5$ for limited ductile plastic regions

$\beta = 7$ for ductile plastic regions

A_r = aspect ratio of wall (h_w/L_w)

$k_m = 1.0$, unless it can be shown that for long walls:

$$k_m = \frac{L_n}{(0.25 + 0.055A_r)L_w} < 1.0 \tag{16}$$

and

$$\xi = 0.3 - \frac{\rho_l f_y}{2.5f_c'} > 0.1 \tag{17}$$

where

ρ_l= vertical reinforcement ratio of the boundary region

The buckling length is assumed to be equal to the theoretical length of the plastic hinge, considered as $l_p = (0.25 + 0.055A_r)L_w$. As previously discussed, this length, depending on the effective height of the cantilever wall and the wall length, is not equal to the experimentally observed buckling length, and will give considerably smaller values if the wall length is rather small.

According to the findings of previous studies conducted by the authors (Dashti et al. 2015), the effect of axial load on the formation and development of out-of-plane deformation is not straightforward. Although this parameter has not been experimentally investigated, the findings of the parametric studies using FEM simulation show that the axial load can prevent development of high residual strains in the reinforcement and contribute to crack closure to happen before any out-of-plane deformation can initiate. However, if this crack closure does not happen on time, the axial load can trigger faster development of out-of-plane deformation by increasing the P-delta effect. Therefore, further research needs to be done addressing the parameter of axial load ratio in provisions of Section 11.4.2.

5   CONCLUSIONS

Out-of-plane instability was the main and only failure pattern that was observed in a wall experiment conducted by the authors, and the response of the specimen was not influenced by other failure patterns such as bar buckling. Therefore, the test observations were compared with the assumptions and analytical models that formed the basis for the current New Zealand code requirements (NZS3101:2006) that address the instability failure of rectangular walls.

The test measurements were in line with the upper bound out-of-plane displacement limit proposed by the analytical models and the stability criterion proved to be a limit above which the out-of-plane deformations cannot recover completely and considerably large residual out-of-plane deformations

develop in the boundary zones.

The assumptions regarding the height of the wall effectively involved in formation of out-of-plane deformations were not in good agreement with the test observations and resulted in non-conservative over estimation of the reinforcement strain corresponding to instability failure of the specimen.

The NZS3101:2006 requirements related to instability failure of structural walls need to be revised in terms of the axial load threshold specified for application of the thickness limitations.

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