Assessment of non-ductile concrete columns

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ABSTRACT: Reinforced concrete column stubs detailed in accordance with the non-seismic provisions of the Concrete Structures Standards NZS 3101:1982 were found during uniaxial compression testing to fail suddenly due to elastic compression buckling of the longitudinal reinforcing steel. Review of the test results has led to provisional recommendations for assessing the performance of suspected non-ductile concrete columns for this particular vulnerability relative to current seismic loadings and design standards.

1 INTRODUCTION

This paper firstly reviews the reported elastic compression buckling initiated failure of CTV Building column stubs during testing at the University of Canterbury. The buckling limited reinforcing steel axial capacity is then assessed using the Steel Structures Standard NZS 3404:1997. A method for estimating the bar buckling limited drift capacity of a column detailed conforming with the NZS 3101:1982 Concrete Structures Standard non-seismic provisions is then proposed. The method draws on recommended limits for use of non-ductile slender or buckling limited columns in steel structures.

2 COLUMN STUB COMPRESSION TEST RESULTS

2.1 Bar Buckling Initiated Failure

Uniaxial compression testing was undertaken at the University of Canterbury on a number of column stubs extracted from the debris of the collapsed CTV Building (Mander 2012). These were 400 mm diameter with 6 H20 (Grade 380) longitudinal reinforcing steel bars with 50 mm cover, confined by R6@250 mm centre spiral reinforcing.

It was reported that failure occurred rapidly with the onset of elastic buckling of the longitudinal reinforcing steel and spalling of the cover concrete. The column stubs were therefore not able to reach their ideal strength in compression required by NZS 3101:1982.

This also indicates that the confinement requirements of cl. 6.4.7.1(b) of NZS 3101:1982 may not always be sufficient to ensure that the ideal strength of columns designed using those provisions were attained if Grade 380 or stronger grade reinforcing steel was used. Similarly this could occur if Grade 275 steel reinforcing was used with actual yield stress that exceeded a certain minimum value.

2.2 Bar Effective Length

An effective length factor ke = 0.9 for the H20 reinforcing bars restrained at 250 mm centres by the R6 spiral, was back calculated using NZS 3404:1997 for the H20 bars consistent with the reported buckling strain from the tests of $\varepsilon_{sc} = 0.0018$.

The corresponding compressive stress at that buckling strain in the reinforcing steel was calculated to be 360 MPa. The calculation of the effective length factor ke used the average tested reinforcing steel yield stress of fy = 448 MPa as reported in Table 1 of the CTV Site Examination and Materials Testing Report (Hyland 2012).

2.3 **Ductile vs Non-Ductile Concrete Column Behaviour**

The reported experimental gross section stress-strain results for column stub "C5 Lower" and "C13 Lower" have been plotted onto a comparable ductile column concrete core stress-strain plot in Figure 1 (Mander 2012). The concrete core stresses of column stub "C5 Lower" and "C13 Lower" would have been a little lower than the gross section stresses due to the reinforcing area being included, but the plot serves to contrast the difference in post-peak stress behaviour and ductility of the CTV Building column stubs, and those like them, compared to a similar ductile concrete column.

The reported experimental results for the three column stubs "C5 Upper", "C5 Lower" and "C13 Lower" all showed non-ductile behaviour which is characterised in Figure 1 by the steeply falling stress to strain curve after the peak load was attained at a gross section compression strain of 0.0018.

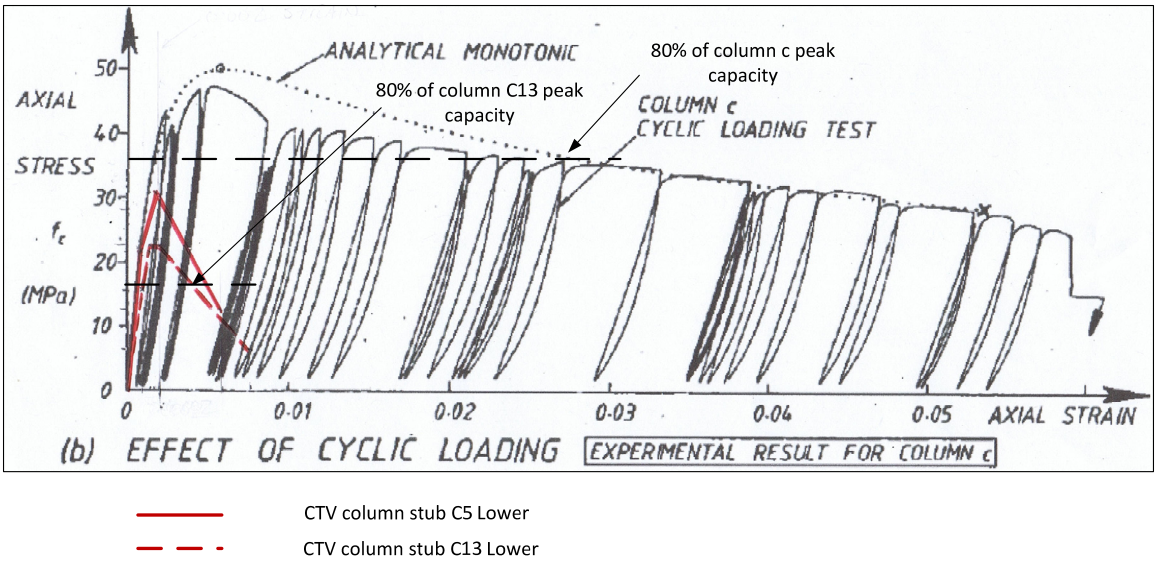

Figure 1. Comparison of ductile and non-ductile column axial compressive stress and strain behaviour

The ductile "column c" tested some years earlier by the same researcher, shown in Figure 1, can be seen to have been able to sustain a significant proportion of its maximum load at much greater strains than the CTV Building column stubs. The ability of a structural element to sustain load with stabilised deformation is a recognised requirement in assessing safe structural performance (Bares and Fitzsimons 1975). A typical engineering criterion is to set a maximum allowable level of post-peak load deformation at 80% of the peak load capacity. The plot of "column c" shows some stable post-peak load capacity due to the slow drop off of its axial stress–strain plot. The CTV Building column stubs however did not show stable post-peak load capacity as their axial stress-strain plots drop away steeply.

The extent of the steeply falling line may also be an overly generous indication of performance, related to the nature of displacement controlled laboratory testing, which reduced the applied load as the test column crushed to maintain the displacement rate. In a real building situation the peak load applied would likely force a rapid and uncontrolled collapse of the column as the strain energy in the column was released into kinetic energy as collapse initiated.

2.4 **Confined vs Unconfined Concrete Column Behaviour**

The comparative behaviour of the columns shown in Figure 1 is similar to that of unconfined and confined concrete behaviour as shown in Figure 2. It therefore appears appropriate, for assessment of ductile performance and drift capacity purposes, to assume unconfined concrete column behaviour occurs once elastic buckling of the longitudinal reinforcing steel initiates.

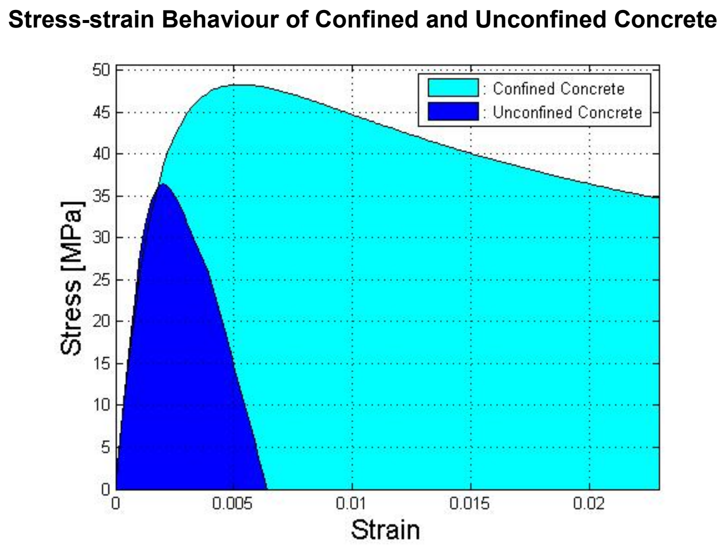

Figure 2. Theoretical comparative stress-strain relationship for unconfined and confined concrete (Montejo and Kowalsky 2007)

3 ASSESSMENT OF COLUMN DRIFT CAPACITY LIMITED BY BAR BUCKLING

The proposed method for assessing the inter-storey drift capacity at which bar buckling initiates is therefore as follows:

1. Calculate the nominal buckling limited capacity N_c of the longitudinal reinforcing bars in accordance with axial compression strut provisions of NZS 3404:1997. Assume lateral restraint to the bar at the spacing of the confining hoops and ties with an effective length factor ke=0.9.
2. If N_c is not limited by buckling at the lower characteristic yield stress of the steel f_y, and at $1.15f_y$ or the expected grade mean yield, then nominally ductile μ=1.25 behaviour can be assumed to be achievable.
3. If N_c is limited by bar buckling calculate the bar axial strain at which N_c is attained.
4. Calculate the inter-storey drift at which the bar buckling strain occurs using a lower bound 28 day concrete strength f'c -5 MPa and f'c +15 MPa assuming confined concrete assumptions.
5. Once the bar buckling strain is achieved calculate the remaining drift capacity if any, assuming unconfined concrete assumptions and limiting concrete core strain of ε_{sc}=0.0016 with no post peak residual strength allowance.
6. Compare the limiting drift capacity with the equivalent ultimate limit state drift demand for the structure calculated in accordance with Section 7 of NZS 1170.5: 2004 assuming a Structural Performance Factor Sp =0.9.

4 ADDITIONAL CONSIDERATIONS

Once the drift capacity is assessed consideration needs to be given to the appropriateness of the column member performance within the structure as a whole. The Steel Structures Standard NZS 3404:1997 uses a system of frame and member categorisation that requires primary or column members to have a level of ductile capability compatible with the secondary members or beam members connected to them. The secondary members are expected to sustain the majority of the damage during an earthquake and protect the primary members.

However slender columns, which are considered non-ductile due to them being buckling limited in terms of their performance, are only acceptable for supporting regular single storey structures (cl. 12.2.6(c) NZS 3404:1997) to minimise the consequences of failure.

The development of elastic bar buckling and unconfined concrete behaviour appears to be in some ways equivalent in terms of non-ductile behaviour to that of slender steel column behaviour in terms of structural frame consequences. It seems reasonable therefore that additional reserves of drift capacity should be required if a column with non-ductile performance characteristics is to be acceptable in an existing structure greater than one storey high.

A reasonable approach appears to be to require such concrete columns to have a minimum drift capacity at least 1.5 times that required by the ultimate limit state drift requirements of the structure. In addition it is recommended that a critical height limit be applied of 4 storeys or 5 if the roof and top storey is lightweight and less than 150 kg/m^2 , as is required for nominally ductile Category 3 frames with μ=1.25 in NZS 3404:1997.

5 CONCLUSIONS

Axial compression tests on CTV Building column stubs, detailed in accordance with the non-seismic requirements of the Concrete Structures Standard NZS 3101:1982, showed that they were susceptible to failure initiating with elastic compression buckling of longitudinal reinforcing steel.

Comparisons with ductile concrete column tests indicate that once bar buckling initiates, the behaviour of the concrete column degrades to that of unconfined concrete with little or no residual post-peak strength.

A method has therefore been proposed for assessing bar buckling limited behaviour of existing concrete columns in structures relative to current standards, drawing on approaches used in the Steel Structures Standard NZS 3404:1997 for the use of slender columns.

No other issues contributing to non-ductile behaviour of concrete columns have been considered in this proposed method, but these should also be considered in any overall assessment. Such issues may include lap splice break-out, non-recoverable tensile yield elongation, beam-column joint detailing and short-column effects caused by interference from non-structural elements.

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