

Seismic performance assessment of non-ductile columns

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ABSTRACT: The collapse of the Canterbury Television (CTV) building in the 22 February 2011 Lyttelton earthquake has highlighted the potential vulnerability of some reinforced concrete ‘gravity columns’ in structures designed to NZS 3101:1982. Over 300 multi-storey buildings throughout New Zealand designed between 1982 and 1995 have been identified as having ‘potentially non-ductile columns’ in a preliminary review undertaken by the Ministry of Business, Innovation and Employment (MBIE) in 2012.

This paper provides some background to the issue before presenting the methodology that the authors have adopted for assessing the seismic vulnerability of these ‘potentially non-ductile columns’. While the context is gravity columns that have been designed for the non-seismic provisions of NZS 3101:1982, the methodology is relevant for checking other reinforced concrete columns of any era. The assessment involves determination of lateral displacement capacity, through the detailed review of various potential failure mechanisms that may lead to the loss of gravity-load carrying capacity of the columns. A range of simplified and more sophisticated analyses to predict inelastic lateral displacement demand are briefly examined. A worked example is provided to illustrate the assessment methodology.

1 INTRODUCTION

The Canterbury Earthquakes Royal Commission (CERC) report into the collapse of the CTV building in the 22 February 2011 Lyttelton earthquake found that the lack of ductile detailing in the gravity columns was likely to have been a contributing factor to the collapse of the building. The CTV building was designed in 1986 and had a six-storey reinforced concrete ‘shear wall protected gravity load system’ structure. The report highlighted the potential possibility of concrete columns being inadequately detailed to accommodate the displacement demand of the building when particular clauses in the Concrete Structures Standard NZS 3101:1982 were interpreted by designers as classifying these columns as secondary elements. As a result, CERC made a recommendation that the Department of Building and Housing (DBH, now a part of MBIE) assess other buildings in New Zealand to identify any that might possess similar potential structural weakness.

The DBH subsequently began an investigation. It commissioned engineers to review the building records for all reinforced concrete buildings, three storeys and taller, designed between 1982 and 1995. This was carried out with assistance from the relevant territorial authorities (TA). After the initial review, the DBH engineers narrowed the list to over 300 buildings that could potentially contain reinforced concrete gravity columns designed to NZS 3101:1982 that may not meet the full seismic ductility requirements of this standard and the current standard NZS 3101:2006. For the purposes of the DBH investigation, columns that met these criteria were identified as ‘potentially non-ductile columns’.

A list of these buildings was then passed back to the relevant TAs. Consequently, many of the TAs issued letters to the owners of these buildings informing them that their building “may have non-ductile columns” and recommending that they “have a suitable qualified and experienced engineer assess the implications of any non-ductile columns on the earthquake performance” of their building.

In the months that followed, many structural engineers around the country began to receive requests to

carry out assessments of buildings with potentially non-ductile columns. This paper has been produced to present the procedure adopted by the authors in our assessments of this issue. In addition to the general assessment procedure, more specific technical tools for determining the lateral displacement capacity, demand and corresponding performance of reinforced concrete columns is outlined with reference to recommended guidelines and papers where appropriate. A worked example is provided in the appendix to illustrate the assessment methodology.

2 SEISMIC VULNERABILITY OF POTENTIALLY NON-DUCTILE COLUMNS

Gravity columns are common in structural systems that contain shear walls, seismic frames, or a combination of both, as the lateral load resisting system. The gravity columns are generally required to support often significant areas of floor, while not being relied upon to contribute to the strength of the lateral system. In order to perform this function, they must remain capable of carrying axial load while undergoing the lateral displacements of the structural system to at least the ultimate limit state, i.e. have sufficient capacity under displacement-induced actions to maintain displacement compatibility. If these displacements are particularly large, or larger than intended for the gravity columns, there is the potential for them to be a critical structural weakness (CSW) with potentially catastrophic consequences.

2.1 General characteristics of columns displaying poor performance in earthquakes

The most critical aspect of the detailing of reinforced concrete columns for flexural ductility capacity is the amount of transverse reinforcement provided, and in particular the spacing between adjacent reinforcement sets (Park and Paulay, 1975). The transverse reinforcement provides confinement to the core concrete and prevents the longitudinal bars from buckling. In general, the lower the transverse reinforcement ratio, the less ductile the column will be under lateral displacements such as those experienced during an earthquake.

The performance of non-ductile reinforced concrete columns with low quantities of transverse reinforcement has been covered extensively in literature (see particularly Boys et al, 2008, Elwood and Moehle, 2005 and Kam et al, 2011).

Figure 1 shows examples of column failures in earthquakes due to inadequate provision of transverse reinforcement for shear, confinement and anti-buckling. Refer to the 2012 CERC and Hyland-Smith reports for further discussion on the non-ductile column behaviour in the CTV building.



Figure 1. Column failures during the 2011 Christchurch earthquake due to inadequate provision of transverse reinforcement: a) Interior column of the CTV building, b) 'Short column' shear failure of perimeter column of a 1970's 7-storey building.

Figure 2 presents examples of gravity column axial-shear failure simulated in laboratory testing.

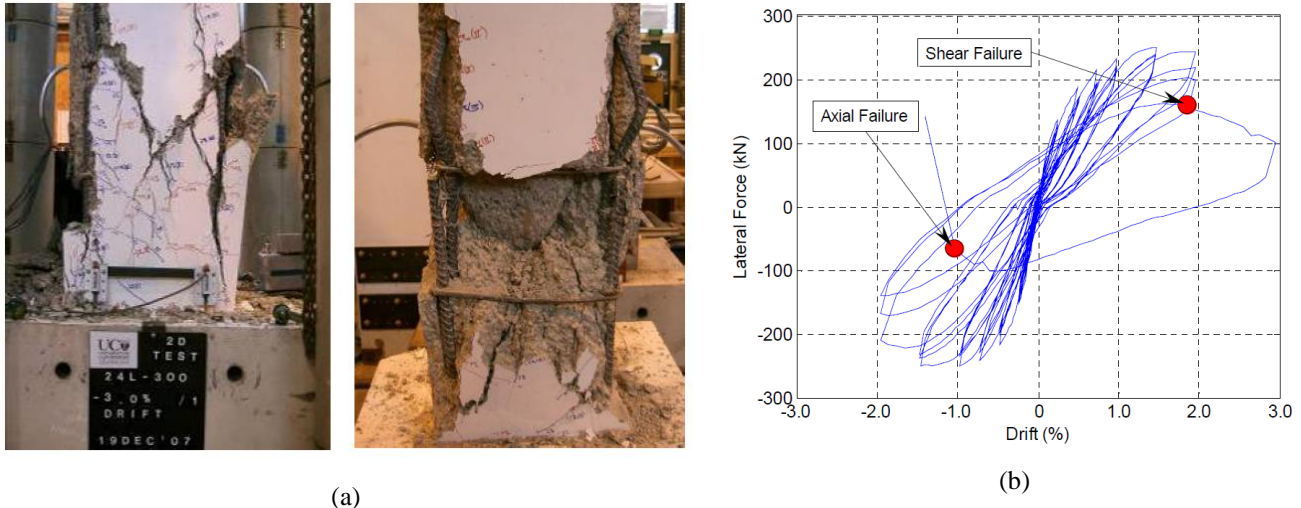


Figure 2. Performance of poorly confined gravity column

In addition to low quantities of transverse reinforcement, several other characteristics of a column can contribute to its vulnerability in an earthquake. The following is a list of indices that may suggest the columns are susceptible to non-ductile behaviour. Suggested limits are given, based on available literature and our experience, and have not been extensively tested. The nomenclature used is that given in NZS 3101:

- Low or inadequate quantities of transverse reinforcement – spacing, $s > d/2$
- High axial load demand – $P/A_g f'_c > 0.3$
- Low core-to-gross concrete area – $A_c/A_g < 0.7$
- High inelastic inter-storey drift demand – $> 1.5\%$ drift
- Detailing – inadequate lap-splice length, lap-splice located in potential plastic hinge zone, poor detailing of transverse reinforcement anchorage e.g. 90 degree bends, welded detailing, lack of support to longitudinal bars
- Location of column – in location prone to inelastic torsional amplification of displacements e.g. corner column or column on opposite face to eccentric shear core

2.2 Gravity column design to NZS 3101:1982

As highlighted by the CERC report on the CTV building, the interpretation of Clause 3.5.14 of NZS 3101:1982 may have led some designers to classify gravity columns as ‘secondary elements’, regardless of whether or not this was aligned with best-practice in the industry at the time. NZS 3101:1982 provided three options for the level of ductile detailing that was to be used in a secondary element. These were; non-seismic provisions, seismic provisions for limited ductility and seismic provisions.

Clause 3.5.14 specifies which of the provisions should be selected, based on the level of design displacement at which the column reaches its elastic limit. If the column could be shown to remain elastic “when the design loads are derived from the imposed deformations $v\Delta$, specified in NZS 4203”, the non-seismic provisions could be used. However, the clauses are open to interpretation and in practice it appears they were applied in an inconsistent manner.

Figure 4 shows an example of the detailing present in a column designed to the non-seismic provisions of NZS 3101:1982. It can be seen that the spacing of the spiral is greater than $d/2$ and the core-to-gross concrete area is less than 0.6.

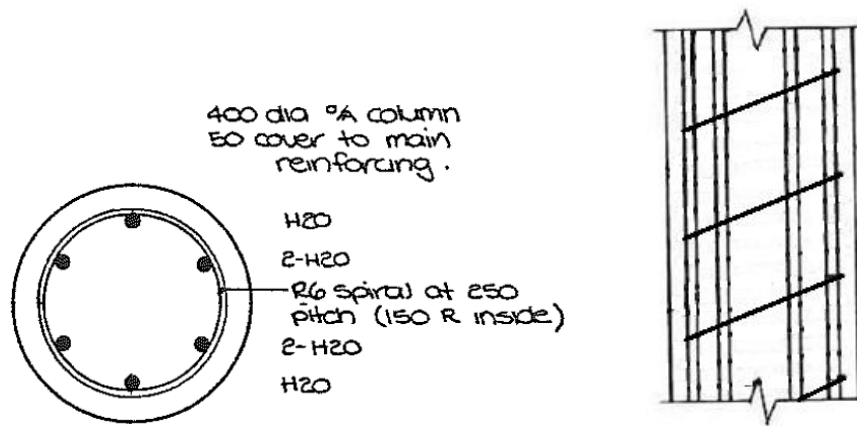


Figure 3. Example of a column designed as a secondary element to the non-seismic provisions of NZS 3101:1982: a) Section showing details (Hyland and Smith, 2012) b) Elevation showing wide spacing

Table 1 provides a comparison of the minimum transverse reinforcement spacing between the previous standard (NZSS 1900 Chapter 9.3:1964) and the three levels of ductile detailing available in NZS 3101:1982 and the versions thereafter (NZS 3101:1995 and NZS 3101:2006).

Table 1. Comparison of transverse reinforcement spacing requirements in concrete structures standards

Design Standard	Non-seismic spacing limit	Limited-ductile spacing limit	Ductile spacing limit
NZSS 1900 Chapter 9.3:1964	For spirally-wound columns, min. of 75mm or $d_c/6$		
NZS 3101:1982	Min. of $h, b_c,$ $16d_b, 48d_{bt}$	Min. of $h, b_c,$ $10d_b, 48d_{bt}$	Min. of $h/5, b_c/5,$ $6d_b, 200\text{mm}$
NZS 3101:1995 and NZS 3101:2006	Min. of $h/3, b_c/3,$ $10d_b$	Min. of $h/4, b_c/4,$ $10d_b$	Min. of $h/4, b_c/4,$ $6d_b$

With reference to Table 1, the primary focus is on columns designed to the non-seismic and limited-ductile provisions of the 1982 code. Even the 1964 code and the non-seismic provisions in the 1995/2006 standards require a fairly close spacing of transverse reinforcement sets, which means columns that have been designed using the non-seismic or limited ductile provisions of the 1982 code are the primary concern.

It is also worth noting, for the 1982 code, the far more stringent requirements for the seismic provisions compared to the non-seismic and limited-ductile provisions and hence how much more ductility a column will possess if the seismic provisions had been chosen instead.

While the focus of the DBH investigation was on buildings designed to the 1982 code, (i.e. between 1982 and 1995) the issue is still relevant for secondary columns from other eras, e.g. pre-1982 and post-1995. Any secondary or non-seismic column that is required to undergo significant ductility demand is a potential CSW if the demand exceeds the capacity of the column.

3 GENERAL ASSESSMENT METHOD

Following the DBH investigation, structural engineers around the country received requests from building owners to provide advice on how to proceed with the assessment of buildings that had been identified as containing potentially non-ductile columns. Initially at the request of the Wellington City Council (WCC) in an advisory role, but also to streamline our approach, Beca has developed a flowchart outlining the recommended approach for assessing these buildings. The flowchart is shown in **Figure 4**.

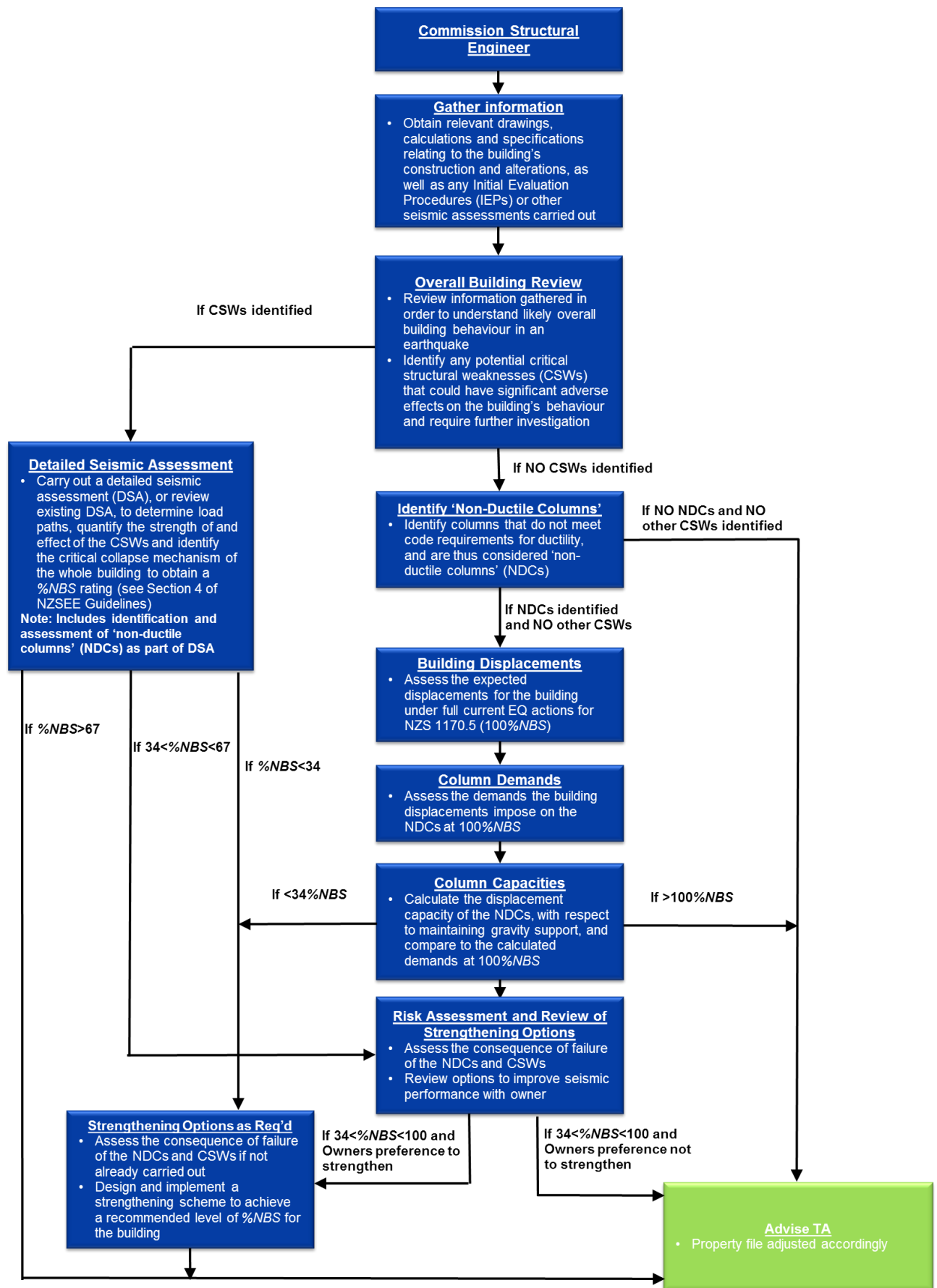


Figure 4. Beca-developed potential non-ductile columns assessment flowchart.

With reference to the flowchart, the approach has the following key steps:

- Gather information – it is important to gather as much information about the building as possible including council records, construction issue drawings (for the initial build and any modifications), strengthening information and any seismic assessments carried out on the building. This process may bring to light new information, or save time on the assessment about to be carried out. Intrusive inspections and/or non-destructive reinforcing bar scanning may need to be carried out in the absence of a suitable set of structural drawings.
- Overall building review – a high-level review of the information should be carried out by an experienced structural engineer to understand the likely building behaviour in an earthquake. This includes identification of any potential CSWs, e.g. plan or vertical irregularities, soft-storeys etc. If any potential CSWs are identified, a full detailed seismic assessment (DSA) of the whole building should be carried out, including assessing the gravity columns. Only in a regular building with a clear lateral load mechanism and no other apparent CSWs should the gravity columns be considered in isolation.

Assessment of potentially non-ductile columns – either as part of a DSA of the overall structure, or in isolation, the process involves calculation and comparison of the lateral displacement capacity with the lateral displacement demand.

4 DETERMINATION OF COLUMN LATERAL DISPLACEMENT CAPACITY

In this section a method is presented that can be employed to determine the lateral displacement at which columns lose their axial load carrying capacity. The method is predominantly based on that outlined in Chapter 7 of the NZSEE assessment guidelines (NZSEE, 2006) for the assessment of existing RC columns, supplemented by research findings.

Whilst intended to be used predominantly in the assessment of suspected ‘non-ductile’ columns designed to the non-seismic provisions of NZS 3101:1982, the method is applicable to columns of any era with any level of transverse reinforcement detailing. A worked example is provided in the appendix to illustrate the assessment methodology.

4.1 Step #1: Obtain necessary column details

The displacement capacity assessment of potentially non-ductile columns of a building should address a range of columns covering different magnitudes of gravity load and different transverse reinforcement detailing. Several key pieces of information are required to undertake the assessment. These are generally all obtainable from a complete set of structural drawings of the building. The information required includes material properties, column dimensions, storey heights, reinforcement details and plans sufficient to carry out an axial-load take-down on the column under consideration. If there are no drawings available, intrusive (e.g. breaking out concrete cover) or non-destructive (reinforcement scanning) inspections may need to be undertaken.

4.2 Step #2: Establish likely governing mechanism

Determining the lateral force-displacement behaviour of a non-ductile column under increasing displacement demand and for different inelastic mechanisms provides a good indication of the expected performance (Refer to Figure 5). Some inelastic mechanisms that can lead to loss of axial-load carrying capacity and non-ductile behaviour are shear failure, brittle flexural mechanisms and premature longitudinal bar buckling or fracture. When assessing these mechanisms, the gravity load acting on the column being assessed needs to be taken into consideration.

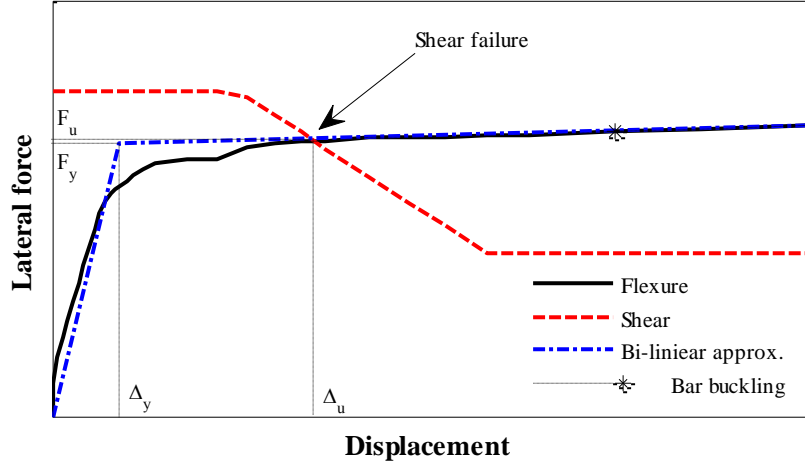


Figure 5. An example of lateral force-displacement behaviour of a curvature-dependent shear-governed column

The probable shear strength of a column can be estimated using Equation 7(6) of the NZSEE assessment guidelines. If the column is identified to be shear-governed based on comparison of its un-degraded probable shear strength and probable flexural strength, then comparison of the different failure mechanisms as shown in **Figure 5** is not necessary and Equation (1) can be employed to determine the displacement at which loss of axial load carrying capacity occurs.

When the un-degraded shear strength of a column is greater than its probable flexural strength as shown in **Figure 5**, then all of the inelastic failure mechanisms need to be taken into consideration. However, the column might still be shear-governed, as the probable shear strength is anticipated to degrade with increased ductility demand due to the reduction in the contribution of the concrete mechanisms, with increasing inelastic cyclic loading (Refer to **Figure 5**). In addition, when a non-ductile column is flexure-governed, longitudinal bar buckling could prevent the column from attaining its ultimate flexural capacity because of the potentially inadequate lateral support provided to the longitudinal bars once the cover concrete spalls.

4.3 Step #3: Lateral displacement capacity if shear-governed

Shear failure of RC columns does not necessarily imply loss of axial load carrying capacity. After shear failure, axial load can still be supported by the longitudinal reinforcing bars and force transfer through shear friction. The displacement of a column at shear failure when fully governed by shear can be calculated from (Elwood and Moehle, 2005):

$$\Delta_s = L_c \left(0.03 + 4\rho_s - 0.024 \frac{\nu}{\sqrt{f'_c}} - 0.025 \frac{P}{A_g f'_c} \right) \geq 0.01L_c \quad (1)$$

An increased level of displacement demand is required to bring about loss of axial load carrying capacity. Based on experimental data, Elwood and Moehle (2005) proposed the ultimate lateral displacement capacity of a fully shear-governed column to be calculated as:

$$\Delta_u = 0.04L_c \left(\frac{1 + \tan^2 65^\circ}{\tan 65^\circ + P \left(\frac{s}{A_{st} f_{yt} d_c \tan 65^\circ} \right)} \right) \quad (2)$$

Limited experimental tests undertaken at the University of Canterbury, New Zealand have shown that

Equation (2) satisfactorily captures the displacement at which shear-dominated RC columns lose their axial load carrying capacity (Boys et al., 2008). We note that the post-peak displacement behaviour of columns in a shear mechanism is still a developing area of research with different researchers proposing various models (e.g. Elwood and Moehle, 2005 and Yoshimura, 2008).

When a column is shear-governed because of an elevated level of ductility demand (Refer to **Figure 5**), the ultimate displacement capacity can be estimated as the point at which the probable shear strength is equal to the probable flexural strength.

4.4 Step #4: Lateral displacement capacity if flexure-governed

The seismic performance of a flexure-governed column depends principally on the quantity of confinement and anti-buckling reinforcement provided to the column. The ultimate displacement capacity of this type of column is typically limited by the lesser of the displacement at which the concrete attains its ultimate strain in compression zones and the displacement at which onset of longitudinal bar buckling is anticipated to occur.

There are several sectional analysis programs that can undertake aspects of the analysis, such as Response-2000 (Bentz and Collins, 2000) and CUMBIA (Montejo and Kowalsky, 2007). However, it is recommended to undertake simplified hand calculations to verify outputs from these programs.

4.4.1 Determine yield displacement

The yield curvature of a column can be estimated from an idealised bi-linear moment-curvature plot of the column, based on effective secant stiffness to the actual moment-curvature curve at 70-75% of the yield moment. Alternatively, the yield curvature can be estimated using the following equations provided in the NZSEE guidelines:

$$\phi_y = \frac{2.35\varepsilon_y}{D}, \text{ for circular columns} \quad (3)$$

$$\phi_y = \frac{2.12\varepsilon_y}{h}, \text{ for rectangular columns} \quad (4)$$

The yield displacement of a column responding in double bending can be calculated from the yield curvature as (adapted from Priestley et al, 1996 for double curvature):

$$\Delta_y = \frac{\phi_y}{6} L_{eff}^2 \quad (5)$$

$$L_{eff} = L_c + 2L_{sp}, \quad L_{sp} = 0.022 f_y d_b \quad (6)$$

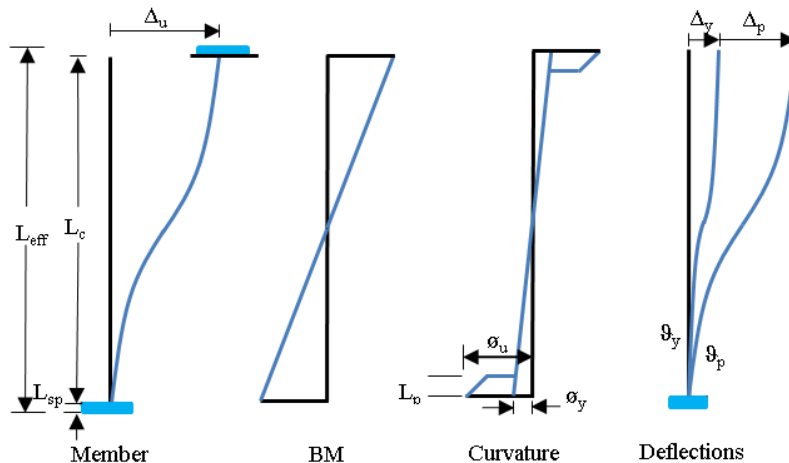


Figure 6. Inelastic deformation of a typical column

4.4.2 Determine ultimate displacement capacity governed by concrete crushing

If the transverse reinforcement detailing renders the column concrete core ‘unconfined’ as per the criteria outlined in the NZSEE assessment guidelines, the ultimate concrete strain (ϵ_{cu}) can be conservatively limited to 0.004. When the core is considered to be ‘confined’, an elevated ultimate concrete strain can be attained (Mander et. al, 1988):

$$\epsilon_{cu} = 0.004 + \left(\frac{1.4 \rho_s f_{yt} \epsilon_{su}}{f_{cc}} \right), \quad \text{where } f_{cc} = 1.5 f'_c \quad (7)$$

This confined ultimate strain value is to be used for the concrete in the confined core only, with the cover concrete assumed to spall at strain magnitudes greater than 0.004. Therefore, the contribution of the cover concrete to the strength of the column could be ignored beyond a strain of 0.004.

The ultimate curvature can be calculated using ultimate strain values as:

$$\phi_u = \frac{\epsilon_{cu}}{c} \quad (8)$$

Alternatively, the ultimate curvature at crushing of concrete can be determined using sectional analysis programs such as Response-2000 (Bentz and Collins, 2000) and CUMBIA (Montejo and Kowalsky, 2007).

The ultimate displacement capacity of a column responding in double-bending can be determined from (Priestley et al., 1996):

$$\Delta_u = \frac{M_u}{M_y} \Delta_y + L_p \left(\phi_u - \phi_y \frac{M_u}{M_y} \right) (L_{eff} - L_p) \quad (9)$$

$$L_p = 0.08 \left(\frac{L_c}{2} \right) + L_{sp} \geq 2L_{sp}, \quad L_{sp} = 0.022 f_y d_b \quad (10)$$

4.5 Step #5: Determine ultimate displacement capacity governed by onset of longitudinal bar buckling

The ultimate capacity of a flexure-governed non-ductile column could be limited by longitudinal bar buckling before initiation of concrete crushing. Equations proposed by Moyer & Kowalsky (2003) and Berry & Eberhard (2005) can be employed to estimate the lateral displacement at which buckling of the longitudinal bars of this type of column is initiated. Based on assessment of a limited number of columns the equation proposed by Moyer & Kowalsky (2003) was observed to provide a more conservative displacement capacity estimate than that proposed by Berry & Eberhard (2005). In some instances, the equation proposed by Moyer & Kowalsky (2003) was found to predict onset of longitudinal bar buckling even before the column starts to yield.

Based on the equation proposed by Berry & Eberhard (2005), the lateral displacement of a column at onset of longitudinal bar buckling can be estimated as:

$$\Delta_u = 3.25 L_c \left(1 + k_{e_bb} \rho_{eff} \frac{d_b}{D} \right) \left(1 - \frac{P}{A_g f'_c} \right) \left(1 + \frac{L_c/2}{10D} \right) \quad (11)$$

$k_{e_bb} = 0$ for columns with $s/d_b \geq 6$

$k_{e_bb} = 40$ and 150 for rectangular columns and spiral-reinforced columns respectively.

4.6 Step #6: Determine the governing ultimate displacement capacity

If the column is fully shear-governed, the ultimate displacement capacity is that determined using Equation (2). If the column is flexure-governed, the ultimate displacement capacity can be estimated as the lesser of the displacements calculated using Equation (9) and Equation (11). The assessment of the column is completed by comparing this displacement capacity with a displacement demand imposed by a design level earthquake. The following section discusses simplified and sophisticated methods for the prediction of the lateral displacement demand.

It is important to note that other mechanisms are possible and an assessment of their capacity made if likely to govern. These include but are not limited to beam-column joint and foundation anchorage and lap-splice capacities.

5 SIMPLIFIED AND SOPHISTICATED ANALYSES FOR THE PREDICTION OF COLUMN INELASTIC LATERAL DISPLACEMENT DEMAND AND DETERMINATION OF %NBS

To assess the performance of a gravity column, the expected inelastic lateral displacement within the storey being assessed must be determined for comparison with the displacement capacity of the column (determined as outlined in Section 4). The following section outlines several ways to determine the displacement demand, varying from simplified hand methods to sophisticated methods utilising non-linear computer analyses. The choice of method may depend on the amount of information available, the complexity of the structure, the objective of the assessment or other restraints such as time.

As with a DSA, the simpler techniques are often more conservative and should be utilised first to obtain a representative estimate that may be sufficient to meet the objectives of the assessment. More sophisticated analyses should only be used to refine the assessment, or if a high degree of reliability is required.

A selection of methods for determining the inelastic lateral displacement demand is presented in increasing order of complexity, along with approaches to calculate the gravity column performance expressed in ‘percentage of New Building Standard’, or %NBS.

5.1 2.5% Drift Limit

A simple check that can be used for screening purposes involves comparing the lateral displacement capacity (expressed as a drift ratio %) of the gravity column within a storey against the ultimate limit state inter-storey deflection limit of 2.5% as prescribed in Clause 7.5.1 of NZS 1170.5:2004. If a building contains a number of columns to be checked, those with a displacement capacity less than 2.5% drift should be prioritised in the assessment.

However, this method alone does not determine the hierarchy of capacity of different elements within a building i.e. the gravity columns may still govern even if their capacity is greater than 2.5% drift. It assumes the building’s maximum inter-storey drift is equal to or less than 2.5%, which may not be the case if there is inadequate strength or stiffness in the building.

5.2 Simplified Hand Method Using Displacement-Based Assessment

Displacement-based assessment (DBA) allows for a relatively quick determination of the lateral displacement demand of the structural system using direct hand calculation methods, without the need for computer-based analysis. The performance of the gravity columns (in %NBS) can be calculated by the ratio between the lateral displacement capacity (Section 4) and the expected lateral displacement demand within each storey.

This topic is well covered in literature (Priestley 1996, Priestley et al 2007, NZSEE 2006) for reinforced concrete buildings. Due to space constraints, the method will not be outlined in detail in this paper. Instead, reference is made to *Displacement-based Seismic Assessment: Practical*

Considerations (Kam et al, 2013), a paper that outlines an approach suited to assessing existing buildings with a direct hand method.

The advantages of the method are that it considers the likely inelastic deformation mechanism that is contributing to the demand on the gravity column, without the need to carry out non-linear analyses. It achieves this through consideration of the governing inelastic mechanism, conversion to a single degree of freedom system and determination of displacement demands using the pseudo-displacement spectra (**Figure 7**).

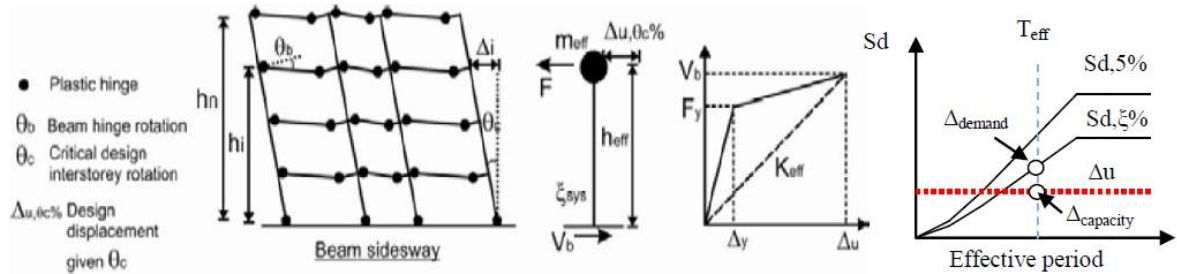


Figure 7. Fundamentals of displacement-based assessment (Kam et al 2013)

However, successful application of this method requires a clear understanding of the building's potential mechanisms and several key assumptions of the building's inelastic behaviour. Therefore, it is recommended that this method only be used for buildings with well-defined load paths and probable inelastic deformation mechanism.

5.3 Linear-Elastic Computer Analysis

Linear-elastic computer analysis of either 2D or 3D models of the building can be used in either a force-based or displacement-based assessment approach. Assuming the building was designed to capacity-design principles and has well-defined inelastic behaviour, equivalent static and modal response spectrum analysis can be used to approximate the lateral displacement and internal action demand on the columns (Figure 8).

The performance (%NBS) of a gravity column can be calculated for a particular storey under consideration by taking the ratio between the lateral displacement capacity (Section 4) and the demand as calculated above.

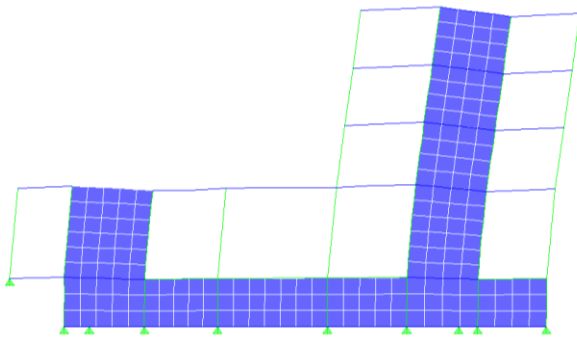


Figure 8. Displaced shape of a shear-wall protected gravity load system under equivalent static loads modelled in ETABS.

The displacements obtained from the analysis represent those of the elastic system and are modified as per Section 7 of NZS 1170.5:2004 to account for ductility and non-linear behaviour. It should be noted that this approach is only recommended for regular buildings with no identified potential CSWs (such as soft-storeys). Otherwise, elastic-analysis displacement prediction can be grossly inaccurate for non-ductile behaviour.

Alternatively, the displacement demands can be estimated using the displacement-based assessment method described in the preceding section. The linear model can then be used to determine the internal

actions for a given displacement demand and deformed shape profile, i.e. checking the column capacities under an imposed displacement profile. Care is needed to implement this pseudo-non-linear pushover approach as the engineer would need to use the appropriate deformed shape profile as determined by the expected global inelastic behaviour.

5.4 Non-linear Computer Analysis

The next level of sophistication is to carry out an explicit non-linear computer analysis of the whole building; either a non-linear pushover or a non-linear time history analysis (Refer to Figure 9). The detailed explanation of these methods is outside the scope of this paper, but it is important to note that these methods are available, and are likely to be more reliable if carried out by an engineer experienced in the non-linear modelling of structures. These analyses are likely to be carried out as part of a full DSA of the building. The various input parameters would require careful evaluation to ensure reliable modelling of the inelastic behaviour including the capacity of critical structural members, e.g. shear walls, columns and beams of a seismic frame and gravity columns.

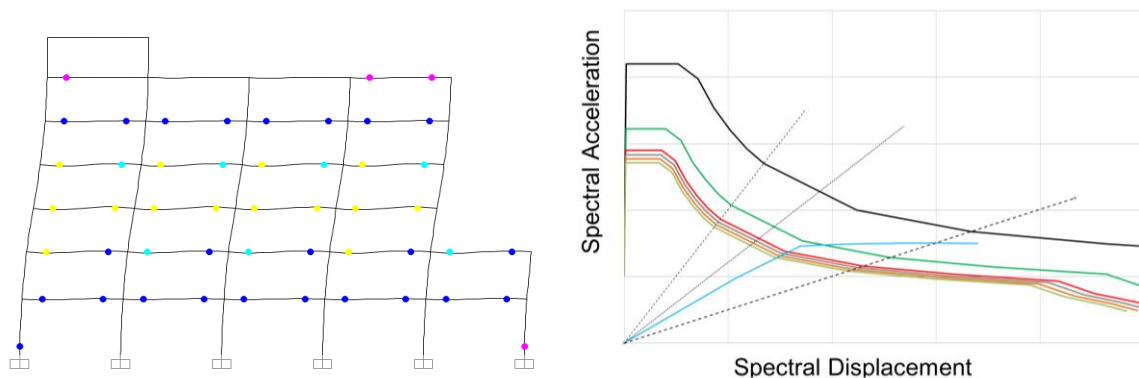


Figure 9. Fundamentals of non-linear pushover assessment: a) 2D model in SAP2000 showing flexural hinges; b) Capacity curve plotted on acceleration-displacement response spectrum (ADRS) to determine performance.

6 CONCLUSIONS AND FUTURE RESEARCH

Columns that are susceptible to non-ductile behaviour and potential loss of gravity load capacity that may lead to partial or complete collapse of the building under seismic shaking present a critical structural weakness (CSW).

While a framework has been proposed to assess a building identified as having potentially non-ductile reinforced concrete columns, we have highlighted the need for a holistic review of the whole building in order to gain an appreciation of the likely inelastic mechanism, and any other potential CSWs.

From our experience, non-ductile columns present a greater risk to the building when other global factors amplify the demand on them, e.g. severe irregularities, soft-storey mechanisms and premature failure of the primary lateral load system.

Several topics were identified as areas for future research and clarification within the existing guidance document (NZSEE 2006):

- Axial-shear failure drift limit state – is the data used by Elwood and Moehle (2005) or Yoshimura (2008) applicable to New Zealand?
- Premature buckling of longitudinal reinforcing in columns with very light transverse reinforcement ($s > d/2$) is an area which requires further testing. The data collected by Berry and Eberhard (2005) is limited to well-designed columns.

NOTATION

A_c	area of concrete core of section measured to outside of peripheral stirrup, mm^2
A_g	gross area of section, mm^2
A_{st}	area of transverse reinforcement parallel to the applied shear and having spacing s , mm^2
b_c	width of column section, mm
c	distance from extreme compression fibre to neutral axis, mm
d	distance from extreme compression fibre to centroid of tension reinforcement, mm
D	diameter of section, mm
d_b	diameter of longitudinal reinforcing bar, mm
d_{bt}	diameter of transverse reinforcing bar, mm
d_c	depth of concrete core measured parallel to the applied shear, mm
f_c	probable compressive strength of concrete, MPa
f_{cc}	probable confined compressive strength of concrete, MPa
f_y	probable yield strength of longitudinal reinforcing bar, MPa
f_{yt}	probable yield strength of transverse reinforcing bar, MPa
h	overall depth of column section, mm
L_c	clear column length, mm
L_{eff}	effective column length, mm
L_p	plastic hinge length, mm
L_{sp}	strain penetration length, mm
N^*	column axial load at ultimate limit state positive for compression, N
P	column axial load at ultimate limit state positive for compression, N
s	centre-to-centre spacing of stirrup-ties along the member, mm
Δ_p	plastic lateral displacement of column centreline at top of storey relative to bottom, mm
Δ_u	ultimate lateral displacement of column centreline at top of storey relative to bottom, mm
Δ_y	yield lateral displacement of column centreline at top of storey relative to bottom, mm
ε_{cu}	ultimate compressive strain of concrete
ε_{su}	ultimate tensile strain of longitudinal reinforcement
ε_y	yield strain of longitudinal reinforcement
v	shear stress corresponding to the plastic shear capacity of a section, MPa
Φ_y	yield curvature of column section, rad/mm
Φ_u	ultimate curvature of column section, rad/mm
ρ_s	volumetric ratio of transverse reinforcement

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APPENDIX A

WORKED EXAMPLE – LATERAL DISPLACEMENT CAPACITY OF A COLUMN

This worked example will detail a procedure for determining the lateral displacement capacity of a reinforced concrete column with light transverse reinforcement. The steps are set out to correspond with those of Section 4 of the paper.

Step #1: Obtain necessary column details

- a) Column section, elevation and reinforcement details: See **Figure 10**. Note: column reinforcement consistent full height of storey.

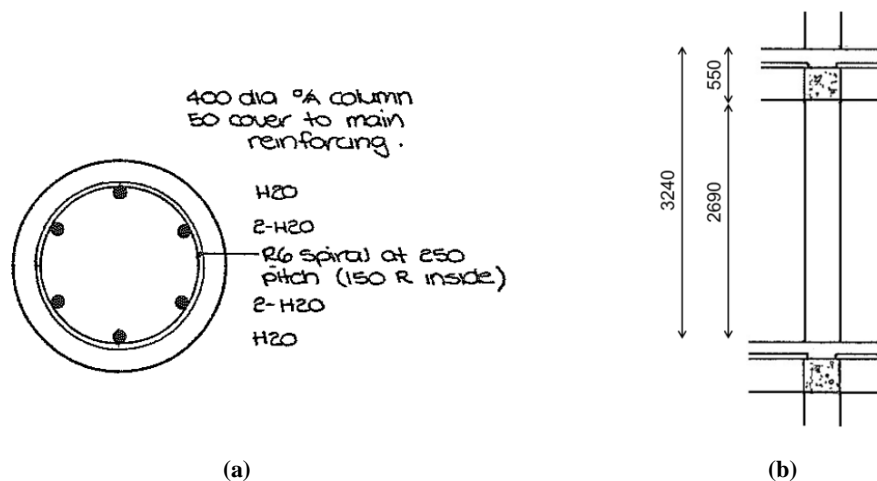


Figure 10. a) Column cross-section; b) Column elevation

- c) Material properties:

Probable concrete compressive strength, $f'_c = 27.5\text{MPa}$

Longitudinal reinforcement probable yield strength, $f_y = 448\text{MPa}$

Longitudinal reinforcement probable ultimate tensile strength, $f_u = 603\text{MPa}$

Transverse reinforcement probable yield strength, $f_{yh} = 330\text{MPa}$

d) Axial load take-down

In a multi-storey building, an axial load take-down should be completed for each column being assessed. The displacement capacity is then calculated separately for each storey, as the capacity depends on axial load and column details (dimensions, reinforcement etc.) which may vary over different storeys.

For simplicity, only one axial load value will be considered in this example:

$$N^* = 500\text{kN}$$

e) Calculate important column variables

At this stage it is useful to calculate some variables that will be used regularly in the assessment.

Diameter, $D = 400\text{mm}$

Gross concrete area, $A_g = 125,664\text{mm}^2$

Core concrete diameter, $d_c = 312\text{mm}$

Core concrete area, $A_c = 76,454\text{mm}^2$

Volumetric transverse reinforcement ratio, $\rho_s = 4A_{tr}/d_c s = (4 \times 28.3\text{mm}^2)/(312\text{mm} \times 250\text{mm}) = .0015$

Effective reinforcement ratio, $\rho_{eff} = \rho_s f_y/f'_c = 0.0015 \times 330 / 27.5 = 0.018$

Effective depth, $d = \text{approx. } 0.8D = 0.8 \times 400\text{mm} = 320\text{mm}$

$s = 250\text{mm} > d/2 = 160\text{mm}$, so column appears to be poorly confined.

Step #2: Determine whether column shear or flexure-governed

The column may be fully flexure-governed, fully shear-governed or curvature-dependent shear-governed. A series of checks have to be made to determine which of the three categories applies.

a) Determine probable flexural strength of the potential plastic region in the storey being assessed

Strength reduction factor (ϕ) for flexure to be taken as 1.0 (NZSEE, 2006)

From routine section analysis for an axial load of 500kN: $M_p = 156\text{kNm}$

For over-strength moment, take $1.25/1.08 = 1.16$ times M_p .

$$M_o = 1.16M_p = 1.16 \times 156 = 181\text{kNm}$$

b) Determine shear strength demand assuming flexural hinges at top and bottom of column

$$V^* = 2M/L_c$$

Using probable flexural strength $V_p^* = (2 \times 156\text{kNm})/2.69\text{m} = 116\text{kN}$

Using over-strength flexural strength $V_o^* = (2 \times 181\text{kNm})/2.69\text{m} = 135\text{kN}$

c) Determine probable shear strength

If the transverse reinforcement varies within the storey (e.g. larger spacing in mid-height region, closer spacing in end regions) the shear strength should be taken as the smaller of those strengths calculated

for the end region and middle region.

Since the transverse reinforcement for the example column does not vary throughout the storey, the critical position for calculating the shear strength will be in the end regions (potential plastic hinge regions) as they are susceptible to degradation of shear strength when subjected to inelastic cyclic demand.

The approach outlined in Section 7 of the NZSEE assessment guidelines can be used. Equation 7(6) of NZSEE states: $V_p = 0.72(V_c + V_s + V_n)$

Concrete contribution; use Equation 7(7): $V_c = k(\sqrt{f_c})0.8A_g$

From Figure 7.7b of the NZSEE assessment guidelines the factor k can vary between 0.29 for low levels of curvature ductility demands and 0.1 for high levels of curvature ductility demands.

To determine if the column is fully flexure-governed, calculate the concrete contribution to shear strength assuming $k = 0.1$:

$$V_c = 0.1 \times (\sqrt{27.5\text{MPa}}) \times 0.8 \times 125664\text{mm}^2 = 52,719\text{N} = 53\text{kN}$$

To determine if the column is fully shear-governed, calculate the concrete contribution to shear strength assuming $k = 0.29$:

$$V_c = 0.29 \times (\sqrt{27.5\text{MPa}}) \times 0.8 \times 125664\text{mm}^2 = 152,885\text{N} = 153\text{kN}$$

Steel contribution, use equation 7(9):

$$\begin{aligned} V_s &= (\pi/2) A_{sp} f_{yt} d'' (1/s) (1/\tan 30^\circ) \\ &= (\pi/2) \times 28.3\text{mm}^2 \times 330\text{MPa} \times 312\text{mm} \times (1/250\text{mm}) \times (1/\tan 30^\circ) = 31,710\text{N} = 32\text{kN} \end{aligned}$$

Axial compressive load contribution, use equation 7(10): $V_n = N \tan \alpha$

Where α is the angle between the longitudinal axis of the column and a straight line between the centroids of the concrete compressive blocks at the top and bottom of the column.

$$\text{so } \alpha = \tan^{-1}((400 - \beta c)/L_c)$$

$$c = 144\text{mm (from Response)}$$

$$\text{so } \alpha = \tan^{-1}((400\text{mm} - 0.85 \times 144\text{mm})/2690\text{mm}) = 6^\circ$$

$$\text{and } V_n = 500\text{kN} \times \tan 6^\circ = 53\text{kN}$$

Therefore, the total shear strength of plastic hinge zone using $k = 0.1$:

$$V_n = 0.72 \times (53\text{kN} + 32\text{kN} + 53\text{kN}) = 99\text{kN}$$

And, the total un-degraded shear strength of the plastic hinge zone assuming $k = 0.29$:

$$V_n = 0.72 \times (153\text{kN} + 32\text{kN} + 53\text{kN}) = 171\text{kN}$$

The un-degraded shear strength is greater than the flexural over-strength. However, the degraded shear strength is less than the probable flexural strength so if the column is capable of achieving high levels of curvature ductility it may be shear-governed.

Step #3: Lateral displacement capacity if shear-governed

Provided the un-degraded shear strength is greater than the probable flexural strength, the column is not expected to fail in pure shear. However, at elevated levels of ductility demand the shear strength is less than the flexural over-strength. Therefore, the governing mechanism is dependent on the magnitude of the displacement demand.

Step #4: Lateral displacement capacity if flexure-governed

a) Yield displacement

$$\Phi_y = 2.35 \cdot \epsilon_y / D = 2.35 \cdot (448 / 200000) / 400 = 13 \cdot 10^{-6} \text{ rad/mm}$$

$$L_{eff} = L_c + 2L_{sp} = 2690 \text{ mm} + (2 \times 0.022 \times 448 \times 20) \text{ mm} = 3084 \text{ mm}$$

$$\Delta_y = \Phi_y L_{eff}^2 / 6 = 13 \cdot 10^{-6} \text{ rad/mm} \times (3084)^2 / 6 = 21 \text{ mm}$$

b) Ultimate displacement

The core is considered to be unconfined as $s > d/2$, $\epsilon_{cu} = 0.004$

$$\Phi_u = \epsilon_{cu} / c = 0.004 / 144 \text{ mm} = 28 \cdot 10^{-6} \text{ rad/mm}$$

$$L_p = 0.08(L_c/2) + L_{sp} \geq 2L_{sp} = 0.08 \times 1345 \text{ mm} + 0.022 \times 448 \times 20 \text{ mm} \geq 2 \times 0.022 \times 448 \times 20 \text{ mm} = 394 \text{ mm}$$

$$\begin{aligned} \Delta_u &= M_u / M_y \times \Delta_y + L_p (\Phi_u - \Phi_y \times M_u / M_y) (L_{eff} - L_p) \\ &= 156 \text{ kNm} / 136 \text{ kNm} \times 21 \text{ mm} + 394 \text{ mm} \times (28 \cdot 10^{-6} \text{ rad/mm} - 13 \cdot 10^{-6} \text{ rad/mm} \times 156 \text{ kNm} / 136 \text{ kNm}) \times (3084 \text{ mm} - 394 \text{ mm}) = 24 \text{ mm} + 14 \text{ mm} = 38 \text{ mm} \end{aligned}$$

Step #5: Determine ultimate displacement capacity governed by onset of longitudinal bar buckling

$$\begin{aligned} \Delta_u &= 0.0325 L_c (1 + k_{e_bb} \rho_{eff} d_b / D) (1 - P / (A_g f'_c)) (1 + L_c / (20D)) \\ &= 0.0325 \times 2690 \times (1 + 0 \times 0.0174 \times 20 / 400) \times (1 - 500000 / (125,664 \times 27.5)) \times (1 + 2690 / (20 \times 400)) \\ &= 0.0325 \times 2690 \times 1 \times 0.855 \times 1.336 = 100 \text{ mm} \end{aligned}$$

Step #6: Determine the governing ultimate displacement capacity

The expected response of the column when subjected to earthquake induced lateral forces is as shown in **Figure 11**. The ultimate displacement capacity of the column is **38mm**, which is the smaller of the capacity at concrete crushing failure and at the onset of longitudinal bar buckling.

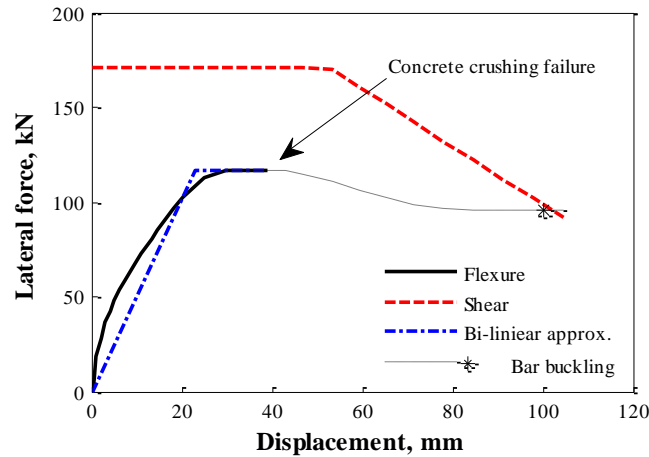


Figure 11. The expected performance of the column