Design Example – Column in Simple Construction

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Date: 15th February 2012
Ref: MEM8001

Key Words
Column, Simple Construction, NZS 3404, Design example,

Introduction

It is common and cost effective to separate the building gravity only steel structure from seismic/wind lateral bracing frames. The gravity structure can then be designed as simple construction in accordance with NZS 3404 Steel Structures Standard. In simple construction the bending members may be assumed to have their ends connected for shear only and to be free to rotate. Examples of such connections are shown in figure 1. In beams and column frames for which simple construction is assumed, there will none the less be bending moments acting on the columns which are caused by eccentricity of the beam reactions.

The minimum eccentricities which must be used for column design purposes are stated in NZS 3404 4.3.4.2. Beam reactions are taken as a minimum distance of 100mm from the face of the column towards the span or at the centre of bearing, whichever gives the greater eccentricity, except that for a column cap, the load shall be taken as acting at the face of the column, or edge of packing if used, towards the span, see figure 1.

Figure 1: a) Web side plate (WP) connected to column flange b) Flexible end plate connected to column web c) Beam supported on column cap plate

A worked example has been prepared to illustrate the design of columns in simple construction in accordance with NZS 3404 Steel Structures Standard (SNZ, 2007).

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Design Parameters

The column is continuous and forms part of a structure of simple construction. The column is nominally pinned at the base. Beams are connected to the column flange by flexible end plates. The exercise is to check that a 200UC46 column section is adequate for the ground floor column.

Ultimate Limit State Loadings

Reaction from beam 1  \( V_1^* = 37kN \)

Reaction from beam 2  \( V_2^* = 147kN \)

Reaction from beam 3  \( V_3^* = 28kN \)

Compression in column between levels 1 and 2
\( N_{1,2}^* = 377kN \)

Design Actions

The total compression force in the column between ground and level 1
\( N_{\text{Ground-1}}^* = N_{1,2}^* + V_1^* + V_2^* + V_3^* = 377 + 37 + 147 + 28 = 589kN \)

For columns in simple construction, NZS 3404 4.3.4.2 states that the beam reactions are assumed to act at a distance of 100 mm from the face of the column. The bending moments calculated are used directly with no need to consider second order effects.

For a 200UC46 column the bending moments at level 1 are:
\[
M_{1,x}^* = V_2^* \left( \frac{d_2}{2} + 100 \right) = 147 \times \left( \frac{203}{2} + 100 \right) \times 10^{-3} = 29.62kNm
\]

In the y-axis the bending moments in the column are proportional to the difference in beam reactions of the two incoming beams (beam1 and beam3)
\[
M_{1,y}^* = V_1^* - V_3^* \left( \frac{b_{wc}}{2} + 100 \right) = 37 - 28 \times \left( \frac{7.3}{2} + 100 \right) \times 10^{-3} = 0.93kNm
\]

In accordance with NZS 3404 4.3.4.3 these bending moments are distributed between the column lengths above and below level 1 in proportion to their bending stiffness. This clause also states; this moment has no effect on
the floor or frames above or below this level ie uses a triangular moment distribution. Therefore the design bending moments acting on the column length between ground and level 1 are:

Major axis (x-axis) \( M_x^* = 29.62 \times \frac{3}{8} = 11.11\text{kNm} \)

Minor axis (y-axis) \( M_y^* = 0.93 \times \frac{2}{8} = 0.35\text{kNm} \)

Note that the bending stiffness of the column between ground and level 1 is reduced due to the presence of the nominally pinned base connection. It is conservative to ignore this in determining the design bending moments.

![Figure 3: Column Loads](image)

**Column Member Compression Capacity**

Section 6 of NZS 3404 covers members subject to axial compression. A member subject to compression must satisfy both section and member capacity checks.

The buckling effective length may be taken as:

\( L_e = \text{Length ground to level 1} = 5000\text{mm} \)

This assumes an effective length factor \((k_e)\) of 1.0 which is conservative.

In Steel Advisor MEM6101 a method is presented for taking advantage of column continuity to reduce the buckling length.

Using Australian Steel Institute *Design Capacity Tables for Structural Steel Volume 1: Open Sections* (AISC, 1999)

\[ \phi N_y = \phi k_f A_f f_y = 1590\text{kN} \text{ (section capacity)} \]

\[ \phi N_{cx} = \alpha_{cx} \phi N_y = 1255\text{kN} \text{ (major axis member capacity)} \]

\[ \phi N_{cy} = \alpha_{cy} \phi N_y = 786\text{kN} \text{ (minor axis member capacity)} \]

\[ N_{\text{Ground-1}} \leq \phi N_y, \phi N_{cx}, \phi N_{cy} \text{ min} \quad \text{OK!} \]

**Column Combined Biaxial Bending and Compression**

The column is subject to combined biaxial bending and compression. Section 8 of NZS 3404 covers combined actions. A column member subject to biaxial bending must be checked for combined actions irrespective of the level of axial force. There is a helpful flow diagram, figure C8.1.1 in NZS 3404 Part 2:1997 which helps explain how the combined bending checks of section 8 are applied in various situations.
The capacity of the column member is dependent on whether the column member is fully lateral restrained.

*Is the column fully laterally restrained (FLR)?*

Beams of open sections bent in their stiffer principal plane are susceptible to a type of buckling known as flexural torsional buckling involving a combination of lateral deflection and twist.

A segment or member bent about its major axis, that has a member moment capacity equal to the section moment capacity, i.e. $\phi M_{bx} = \phi M_{sx}$ (ie $\alpha_m \geq 1.0$), is deemed as having full lateral restraint, and the ability of the member to resist the applied loads is solely dependent on the section moment capacity. This mode of buckling does not apply to open sections bent about their minor axis ie $\phi M_{by} = \phi M_{sy}$

The column is restrained laterally in both the x direction and the y direction at each floor level, but is unrestrained between the floors.

The buckling length may be taken as:

$L_e$ = Length ground to level 1 = 5000mm

As previously discussed, a triangular distribution of moment is to be assumed, therefore the appropriate moment modification factor is $\alpha_m = 1.75$ (Table 5.6.1 NZS 3404)

The following 200UC 46 design capacities have been taken from the Australian Steel Institute Capacity Tables (AISC, 1999).

**Section**

- $\phi N_x = 1590 \text{ kN}$
- $\phi M_{sx} = 133 \text{ kN}$
- $\phi M_{sy} = 60.3 \text{ kNm}$

**Member**

- $\phi N_{cx} = 1255 \text{ kN}$
- $\phi N_{cy} = 786 \text{ kN}$
- $\phi M_{bx} = 89.4 \text{ kNm}$ (based on $\alpha_m = 1.0$)

Check if column is fully laterally restrained

$\alpha_m \phi M_{bx} = 1.75 \times 89.4 = 156 > \phi M_{sx}$ therefore the column is fully laterally restrained ie $\alpha_m \phi M_{bx} = \phi M_{sx}$

**Column Biaxial Bending and Compressive Axial Load Section Check**

Cl 8.3.4 NZS 3404 sets out the *section* check for a column subject to biaxial bending and compression axial load. The general design provisions of 8.3.4.1 NZS 3404 will be used.

$$\frac{N_x^{\prime}}{\phi N_x} + \frac{M_{sx}^{\prime}}{\phi M_{sx}} + \frac{M_{sy}^{\prime}}{\phi M_{sy}} \leq 1.0$$

$$\frac{589}{1590} + \frac{11.11}{133} + \frac{0.35}{60.3} = 0.46 \leq 1.0$$

OK!

The column biaxial bending and compressive axial load section check is satisfied. If the general design provisions of 8.3.4.1 NZS 3404 is not satisfied an alternative provision may be used, for completeness this is set out below. The alternative provisions of 8.3.4.2 NZS 3404 may be used if all the requirements of 8.1.5 are satisfied.
### NZS 3404 8.1.5

**Requirements** | **Satisfied?** | **Description**
--- | --- | ---
8.1.5(a) | Yes | The column is a doubly symmetric I section
8.1.5(b) | Yes | The flange plate slenderness \( ((b_f - t_w)/2t_w)\sqrt{(f_y/250)} = 9.7 \) is less than the slenderness limit in table 8 (10). The web slenderness is also less than the slenderness limit in table 8.
8.1.5(c) | Yes | The form factor \((k_f)\) is unity
8.1.5(d) | Yes | The member is not subject to transverse loading

**All requirements satisfied?** | Yes |

As all requirements of 8.1.5 are satisfied the alternative provisions of 8.3.4.2 may be used.

\[
\left( \frac{M_x}{\phi M_{sx}} \right)^{1.4} + \left( \frac{M_y}{\phi M_{sy}} \right)^{1.4} \leq 1.0
\]

where:

\[
\phi M_{sx} = 1.18\phi M_{sx} \left( 1 - \frac{N'}{\phi N_{sx}} \right) \leq M_{sx}
\]

\[
\phi M_{sx} = 1.18 \times 133 \times \left( 1 - \frac{589}{1590} \right) = 98.8\,\text{kNm}
\]

\[
\phi M_{sy} = 1.19\phi M_{sy} \left( 1 - \frac{N'}{\phi N_{sy}} \right)^2 \leq \phi M_{sy}
\]

\[
\phi M_{sy} = 1.19 \times 60.3 \times \left( 1 - \left( \frac{589}{1590} \right)^2 \right) = 61.9\,\text{kNm} > \phi M_{sy} \therefore \phi M_{sy} = 60.3\,\text{kNm}
\]

\[
\gamma = 1.4 + \left( \frac{N'}{\phi N_{sy}} \right) \leq 2.0
\]

\[
\gamma = 1.4 + \left( \frac{589}{1590} \right) = 1.77
\]

\[
\therefore \left( \frac{M_x}{\phi M_{sx}} \right)^{1.4} + \left( \frac{M_y}{\phi M_{sy}} \right)^{1.4} = \left( \frac{11.11}{98.8} \right)^{1.77} + \left( \frac{0.35}{60.3} \right)^{1.77} = 0.021 \leq 1.0 \quad \text{OK!}
\]

**Column Biaxial Bending and Compressive Axial Load Member Check**

CI 8.4.5.1 NZS 3404 sets out the member check for a column subject to biaxial bending and compression axial load.

\[
\left( \frac{M_x}{\phi M_{sx}} \right)^{1.4} + \left( \frac{M_y}{\phi M_{sy}} \right)^{1.4} \leq 1.0
\]

where:

- \( M_{cx} = M_x \) for a member with full lateral restraint
- \( \phi M_{sx} = \phi M_{sx} \left( 1 - \frac{N'}{\phi N_{cx}} \right) \)
- \( \phi M_{sx} = 133 \times \left( 1 - \frac{589}{1255} \right) = 70.6\,\text{kNm} \)
- \( \phi M_{sy} = \phi M_{sy} \left( 1 - \frac{N'}{\phi N_{cy}} \right) \)
- \( \phi M_{sy} = 60.3 \times \left( 1 - \frac{589}{786} \right) = 15.1\,\text{kNm} \)

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\[
\left( \frac{M_x'}{\phi M_{tx}} \right)^{1.4} + \left( \frac{M_y'}{\phi M_{ty}} \right)^{1.4} = \left( \frac{11.11}{70.6} \right)^{1.4} + \left( \frac{0.35}{15.1} \right)^{1.4} = 0.08 \leq 1.0 \quad \text{OK!}
\]

A 200UC 46 would be adequate for the ground floor column.

References

AISC (now ASI), Design Capacity Tables for Structural Steel Volume 1: Open Sections Third Edition, Australian Institute of Steel Construction (now Australian Steel Institute), Sydney, 1999
