20 COLD FORMED STEEL SECTIONS- II

1.0 INTRODUCTION

In the last chapter, the special features and attractions of cold formed steel sections for many industrial applications were presented and discussed. In view of the use of very thin steel sheet sections, (generally in the 1 mm - 3 mm range), particular attention has to be paid to buckling of these elements. Stiffened and unstiffened elements were compared and the concept of effective width to deal with the rapid design of compression elements together with suitable design simplifications, outlined. Finally, the methods adopted for the design of laterally restrained beams and unrestrained beams were discussed. The techniques of eliminating lateral buckling in practice, by providing lateral braces or by attachment to floors etc were described so that the compression flanges would not buckle laterally.

In this chapter the design of columns for axial compression, compression combined with bending as well as for torsional-flexural buckling will be discussed. The diversity of cold formed steel shapes and the multiplicity of purposes to which they are put to, makes it a difficult task to provide general solution procedures covering all potential uses. Some design aspects are nevertheless included to provide a general appreciation of this versatile product. It is not unusual to design some cold formed steel sections on the basis of prototype tests or by employing empirical rules. These are also discussed in a summary form herein.

2.0 AXIALLY COMPRESSED COLUMNS

As pointed out in the last chapter, local buckling under compressive loading is an extremely important feature of thin walled sections. It has been shown that a compressed plate element with an edge free to deflect does not perform as satisfactorily when compared with a similar element supported along the two opposite edges. Methods of evaluating the effective widths for both edge support conditions were presented and discussed.

In analysing column behaviour, the first step is to determine the effective area (A_{eff}) of the cross section by summing up the total values of effective areas for all the individual elements.

The ultimate load (or squash load) of a short strut is obtained from

$$P_{cs} = A_{eff} \,.\, f_{yd} \quad = \quad Q.\, A.\, f_{yd} \qquad (1)$$

where P_{cs} = ultimate load of a short strut

A_{eff} = sum of the effective areas of all the individual plate elements

Q = the ratio of the effective area to the total area of cross section at yield stress

In a long column with doubly - symmetric cross section, the failure load (P_c) is dependent on Euler buckling resistance (P_{EY}) and the imperfections present. The method of analysis presented here follows the Perry-Robertson approach presented in the chapter on "Introduction to Column Buckling". Following that approach, the failure load is evaluated from

$$P_c = \frac{1}{2}\left\{ \left[P_{cs} + (1+\eta)P_{EY}\right] - \sqrt{\left[P_{cs} + (1+\eta)P_{EY}\right]^2 - 4P_{cs}\cdot P_{EY}} \right\} \tag{2}$$

$$\text{where } \eta = 0.002\left(\frac{\lambda_e}{r_y} - 20\right), \quad \text{for } \frac{\lambda_e}{r_y} > 20 \tag{2a}$$

$$\eta = 0, \quad \text{for } \frac{\lambda_e}{r_y} \le 20 \tag{2b}$$

P_{EY} = the minimum buckling load of column = $\dfrac{\pi^2 EI_{min}}{\lambda_e^{\,2}}$

and r_y = radius of gyration corresponding to P_{EY}.

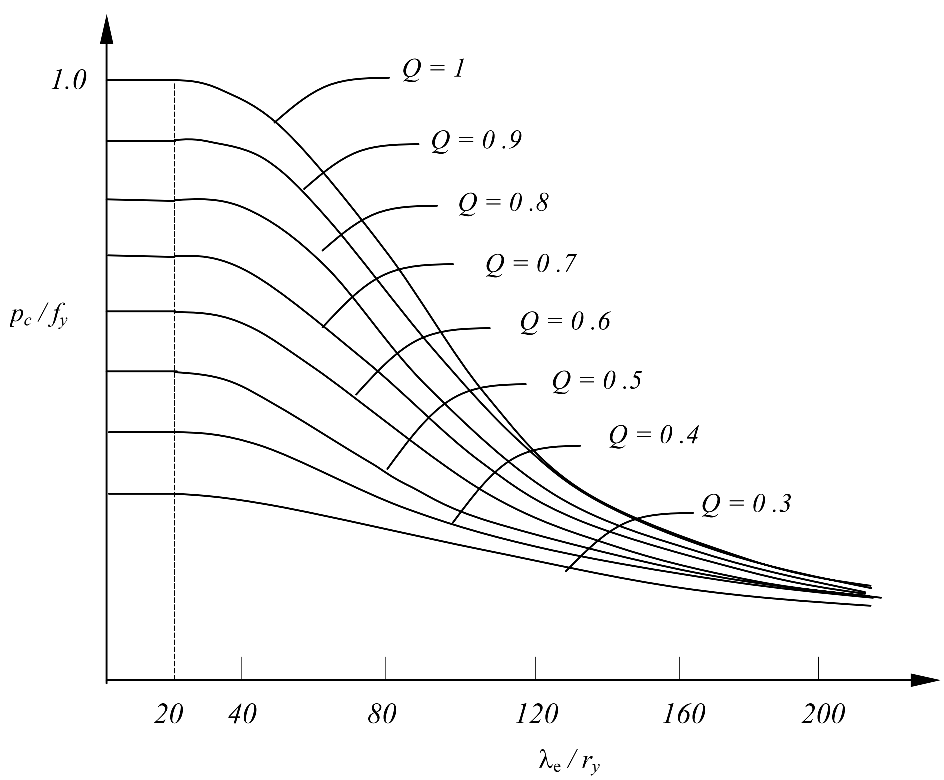
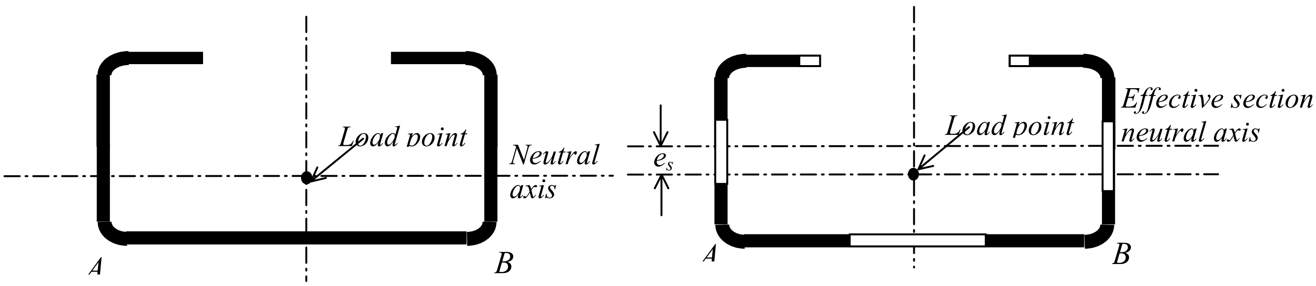


Fig. 1 Column Strength (non- dimensional) for different Q factors

Fig. 1 shows the mean stress at failure (p_c = P_c / cross sectional area) obtained for columns with variation of λ_e /r_y for a number of "Q" factors. (The y-axis is non dimensionalised using the yield stress, f_y and "Q" factor is the ratio of effective cross sectional area to full cross sectional area). Plots such as Fig. 1 can be employed directly for doubly symmetric sections.



(a) Channel section loaded through its centroid

(b) The move of the neutral axis (due to plate buckling) causes an eccentricity e_s and a consequent moment P. e_s . This would cause an additional compression on flange AB

Fig. 2 Effective shift in the loading axis in an axially compressed column

2.1 Effective shift of loading axis

If a section is not doubly symmetric (see Fig. 2) and has a large reduction of effective widths of elements, then the effective section may be changed position of centroid. This would induce bending on an ***initially concentrically loaded section***, as shown in Fig. 2. To allow for this behaviour, the movement of effective neutral axis (e_s) from the geometric neutral axis of the cross section must be first determined by comparing the gross and effective section properties. The ultimate load is evaluated by allowing for the interaction of bending and compression using the following equation:

$$P_{ult} = \frac{P_c \cdot M_c}{M_c + P_c . e_s} \tag{3}$$

where P_c is obtained from equation (2) and M_c is the bending resistance of the section for moments acting in the direction corresponding to the movement of neutral axis; e_s is the distance between the effective centroid and actual centroid of the cross section.

2.2 Torsional - flexural buckling

Singly symmetric columns may fail either (a) by Euler buckling about an axis perpendicular to the line of symmetry (as detailed in 2.1 above) or (b) by a combination of bending about the axis of symmetry and a twist as shown in Fig. 3. This latter type of behaviour is known as Torsional-flexural behaviour. Purely torsional and purely flexural failure does not occur in a general case.

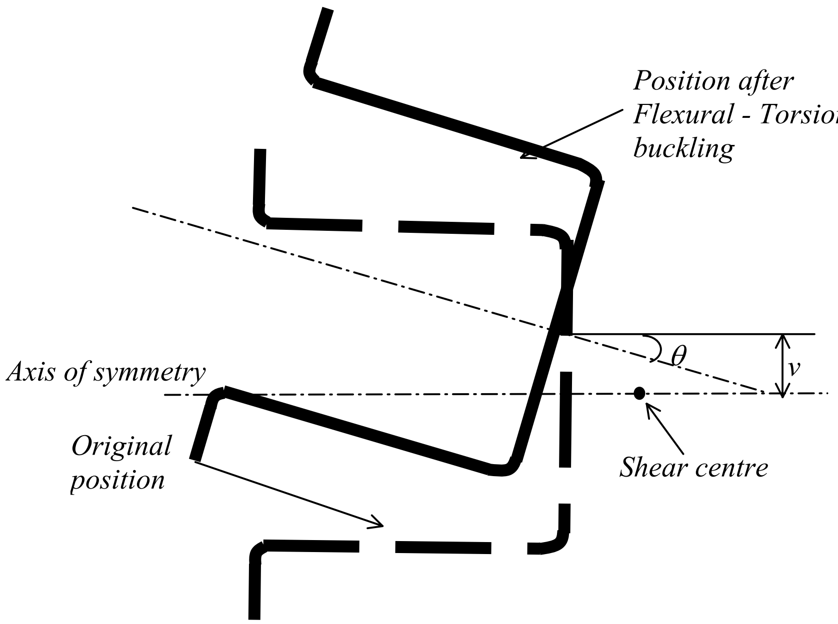


Fig. 3 Column displacements during Flexural - Torsional buckling

Theoretical methods for the analysis of this problem was described in the chapters on Beam Columns. Analysis of torsional-flexural behaviour of cold formed sections is tedious and time consuming for practical design. Codes deal with this problem by simplified design methods or by empirical methods based on experimental data.

As an illustration, the following design procedure, suggested in BS5950, Part 5 is detailed below as being suitable for sections with at least one axis of symmetry (say x - axis) and subjected to flexural torsional buckling.

Effective length multiplication factors (known as α factors) are tabulated for a number of section geometries. These α factors are employed to obtain increased effective lengths, which together with the design analysis prescribed in 2.1 above can be used to obtain torsional buckling resistance of a column.

$$For\ P_{EY} \le P_{TF}, \quad \alpha = 1$$

$$For\ P_{EY} > P_{TF}, \ \alpha = \sqrt{\frac{P_{EY}}{P_{TF}}} \qquad (4)$$

α values can be computed as follows:

where P_{EY} is the elastic flexural buckling load (in Newtons) for a column about the y-axis, i.e. $\dfrac{\pi^2 EI_y}{\lambda_e^{\;2}}$

λ_e = effective length (in mm) corresponding to the minimum radius of gyration
P_{TF} = torsional flexural buckling load (in Newtons) of a column given by

$$P_{TF} = \frac{1}{2\beta}\left[\left(P_{EX} + P_T\right) - \left\{\left(P_{EX} + P_T\right)^2 - 4\beta P_{EX} P_T\right\}^{1/2}\right] \quad (5)$$

where P_{EX} = Elastic flexural buckling load of the column (in Newtons) about the x- axis given by $\dfrac{\pi^2 EI_x}{\lambda_e^{\;2}}$

P_T = Torsional buckling load of a column (In Newtons) given by

$$P_T = \frac{1}{r_0^2}\left(GJ + \frac{2\pi^2 \cdot E\,\Gamma}{\lambda_e^{\;2}}\right) \quad (6)$$

β is a constant given by $\beta = 1 - \left(\dfrac{x_0}{r_0}\right)^2$ (7)

In these equations,

r_o = polar radius of gyration about the shear centre (in mm) given by

$$r_0 = \left(r_x^2 + r_y^2 + x_0^2\right)^{1/2} \quad (8)$$

where

r_x, r_y are the radii of gyration (in mm) about the x and y- axis
G is the shear modulus (N/mm^2)
x_0 is the distance from shear centre to the centroid measured along the x axis (mm)

J St Venants' Torsion constant (mm^4) which may be taken as $\sum \dfrac{bt^3}{3}$, summed up for all elements, where b = flat width of the element and t = thickness (both of them measure in mm)

I_x the moment of inertia about the x axis (mm^4)
Γ Warping constant for all section.

2.3 Torsion Behaviour

Cold formed sections are mainly formed with "open" sections and do not have high resistance to torsion. Hence the application of load which would cause torsion should be avoided where possible. Generally speaking, by adjusting the method of load application, it is possible to restrain twisting so that torsion does not occur to any significant extent.

In general, when examining torsional behaviour of thin walled sections, the total torsion may be regarded as being made up of two effects:

- St. Venant's Torsion or Pure Torsion
- Warping torsion.

St.Venant's torsion produces shear stresses, which vary linearly through the material thickness. Warping torsion produces in-plane bending of the elements of a cross section, thus inducing direct (i.e. normal) stresses and the angle of twist increases linearly.

Since cold formed sections are thin walled, they have very little resistance to St. Venant's Torsion and will twist substantially. The extent of warping torsion in a thin walled beam is very much dependent on the warping restraint afforded by the supports as well as the loading conditions and the type of section.

If the beam ends are restrained from warping, then short beams exhibit high resistance to warping torsion and the total torque acting on such a beam will be almost completely devoted to overcoming warping resistance, the St Venant's Torsion being negligible. Conversely, the resistance to warping torsion becomes low for long beams and warping stresses and degrees of twist become very large.

A detailed theoretical treatment of beams subject to bending and torsion is given in another chapter. As stated previously, particular care and attention should be paid to the detailing of the connections and the method of load application so that the design for torsion does not pose a serious problem.

3.0 COMBINED BENDING AND COMPRESSION

Compression members which are also subject to bending will have to be designed to take into account the effects of interaction. The following checks are suggested for members which have at least one axis of symmetry: (i) the local capacity at points of greatest bending moment and axial load and (ii) an overall buckling check.

3.1 Local Capacity Check

The local capacity check is ascertained by satisfying the following at the points of greatest bending moment and axial load:

$$\frac{F_c}{P_{cs}} + \frac{M_x}{M_{cx}} + \frac{M_y}{M_{cy}} \leq 1 \qquad (9)$$

F_c = applied axial load
P_{cs} = short strut capacity defined by $A_{eff}.P_{yd}$ (eqn. 1)
M_x, M_y = applied bending moments about x and y axis
M_{cx} = Moment resistance of the beam about x axis in the absence of F_c and M_y
M_{cy} = Moment resistance of the beam about y axis in the absence of F_c and M_x.

3.2 Overall buckling check

For members not subject to lateral buckling, the following relationship should be satisfied:

$$\frac{F_c}{P_c} + \frac{M_x}{C_{bx} \cdot M_{cx}\left(1-\frac{F_c}{P_{EX}}\right)} + \frac{M_y}{C_{by} \cdot M_{cy}\left(1-\frac{F_c}{P_{EY}}\right)} \leq 1 \tag{10}$$

For beams subject to lateral buckling, the following relationship should be satisfied:

$$\frac{F_c}{P_c} + \frac{M_x}{M_b} + \frac{M_y}{C_{by} \cdot M_{cy}\left(1-\frac{F_c}{P_{EY}}\right)} \leq 1 \tag{11}$$

where

P_c = axial buckling resistance in the absence of moments (see eq. 2)
P_{EX}, P_{EY} = flexural buckling load in compression for bending about the x- axis and for bending about the y-axis respectively.
C_{bx}, C_{by} = C_b factors (defined in the previous chapter) with regard to moment variation about x and y axis respectively.
M_b = lateral buckling resistance moment about the x axis defined in the previous chapter.

4.0 TENSION MEMBERS

If a member is connected in such a way as to eliminate any moments due to connection eccentricity, the member may be designed as a simple tension member. Where a member is connected eccentrically to its axis, then the resulting moment has to be allowed for.

The tensile capacity of a member (P_t) may be evaluated from

$$P_t = A_e \ . \ P_y \tag{12}$$

where

A_e is the effective area of the section making due allowance for the type of member (angle, plain channel, Tee section etc) and the type of connection (eg. connected through one leg only or through the flange or web of a T- section).

p_y is design strength (N/mm^2)

Guidance on calculation of A_e is provided in Codes of Practice (eg. BS 5950, Part 5). The area of the tension member should invariably be calculated as its gross area less deductions for holes or openings. (The area to be deducted from the gross sectional area of a member should be the maximum sum of the sectional areas of the holes in any cross section at right angles to the direction of applied stress).

Reference is also made to the chapter on "Tension Members" where provision for enhancement of strength due to strain hardening has been incorporated for hot rolled steel sections. The Indian code IS: 801-1975 is in the process of revision and it is probable that a similar enhancement will be allowed for cold rolled steel sections also.

When a member is subjected to both combined bending and axial tension, the capacity of the member should be ascertained from the following:

$$\frac{F_t}{P_t} + \frac{M_x}{M_{cx}} + \frac{M_y}{M_{cy}} \leq 1 \tag{13}$$

and

$$\frac{M_x}{M_{cx}} \leq 1 \tag{14}$$

and

$$\frac{M_y}{M_{cy}} \leq 1 \tag{15}$$

where F_t = applied load

P_t = tensile capacity (see eqn. 12)

M_x, M_y, M_{cx} and M_{cy} are as defined previously.

5.0 DESIGN ON THE BASIS OF TESTING

While it is possible to design many cold formed steel members on the basis of analysis, the very large variety of shapes that can be formed and the complex interactions that occur make it frequently uneconomical to design members and systems completely on theoretical basis. The behaviour of a component or system can often be ascertained economically by a test and suitable modifications incorporated, where necessary.

Particular care should be taken while testing components, that the tests model the actual loading conditions as closely as possible. For example, while these tests may be used successfully to assess the material work hardening much caution will be needed when examining the effects of local buckling. There is a possibility of these tests giving misleading information or even no information regarding neutral axis movement. The specimen lengths may be too short to pick up certain types of buckling behaviour.

Testing is probably the only realistic method of assessing the strength and characteristics of connections. Evaluating connection behaviour is important as connections play a crucial role in the strength and stiffness of a structure.

In testing complete structures or assemblies, it is vital to ensure that the test set up reflects the in-service conditions as accurately as possible. The method of load application, the type of supports, the restraints from adjacent structures and the flexibility of connections are all factors to be considered carefully and modeled accurately.

Testing by an independent agency (such as Universities) is widely used by manufacturers of mass produced components to ensure consistency of quality. The manufacturers also provide load/span tables for their products, which can be employed by structural designers and architects who do not have detailed knowledge of design procedures. An advantage to the manufacturers in designing on the basis of proof testing is that the load/span tables obtained are generally more advantageous than those obtained by analytical methods; they also reassure the customers about the validity of their load/span tables.

6.0 EMPIRICAL METHODS

Some commonly used members such as Z purlins are sometimes designed by time-tested empirical rules; such rules are employed when theoretical analysis may be impractical or not justified and when prototype test data are not available. (Members designed by proven theoretical methods or by prototype testing need not comply with the empirical rules). As an illustration the empirical rules permitted by BS 5950, Part 5 is explained below.

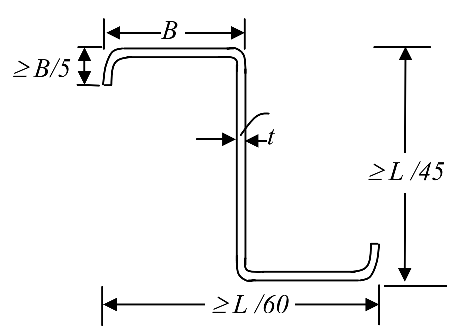


Fig. 4 Z Purlins

6.1 Z Purlins

A Z purlin used for supporting the roofing sheet is sketched in Fig. 4. In designing Z purlins with lips using the simplified empirical rules the following recommendations are to be complied with:

- Unfactored loads should be used for designing purlins
- Imposed loads should be taken to be at least $0.6\ kN/mm^2$
- Claddings and fixings should be checked for adequacy to provide lateral restraint to the purlin and should be capable of carrying the component of load in the plane of the roof slope.
- The purlin should be considered to carry the load normal to roof slope (and a nominal axial load due to wind or restraint forces)
- These rules apply to purlins up to 8 m span in roof slopes up to $22\frac{1}{2}^o$
- Antisag bars should be provided to ensure that laterally unsupported length of the purlin does not exceed 3.8 m. These should be anchored to rigid apex support or their forces should be transferred diagonally to main frames.
- Purlin cleats should provide adequate torsional restraint.

6.2 Design rules

The following design rules apply with reference to Fig. 4

- The overall depth should be greater than $100\ t$ and not less than $L/45$.
- Overall width of compression flange / thickness ratio should be less than 35.
- Lip width should be greater than $B/5$
- Section Modulus $\geq \frac{WL}{1400}\ cm^3$ for simply supported purlins

and $\geq \frac{WL}{1800}\ cm^3$ for continuous or semi rigidly jointed purlins.

In the above,

L = span of the purlin (in mm)
W = Normal component of unfactored (distributed dead load+imposed load) in kN
B = Width of the compression flange in mm
T = thickness of the purlin in mm.

- The net allowable wind uplift in a direction normal to roof when purlins are restrained is taken as *50%* of the (dead + imposed) load.

7.0 CONCLUDING REMARKS

In the two preceeding chapters on cold rolled steel, a detailed discussion of design of elements made from it has been provided, the major differences between the hot rolled steel products and cold rolled steel products outlined and the principal advantages of using the latter in construction summarized. Design methods, including methods based on prototype testing and empirical design procedures have been discussed in detail.

Thin steel products are extensively used in building industry in the western world and this range from purlins and lintels to roof sheeting and decking. Light steel frame construction is often employed in house building and is based on industrialized manufacture of standardized components, which ensure a high quality of materials of construction. The most striking benefit of all forms of light steel framing is their speed of construction, ease of handling and savings in site supervision and elimination of wastage in site, all of which contribute to overall economy.

In the Indian context, industrialized methods of production and delivery of cold rolled steel products to site have the potential to build substantially more houses than is otherwise possible, with the same cash flow, thus freeing capital and financial resources for other projects. Other advantages include elimination of shrinkage and movement cracks, greater environmental acceptability and less weather dependency. Properly constructed light steel frames are adaptable to future requirements and will provide high acoustic performance and a high degree of thermal insulation. Provided the sheets are pre-galvanised, the members provide adequate corrosion protection when used close to the boundaries of the building envelope. The design life of these products exceeds 60 years.

8.0 REFERENCES

1. Rhodes, J. and Lawson, R. M., Design of Structures using Cold Formed Steel Sections, The Steel Construction Institute, Ascot, 1992.
2. Chung, K. F Worked examples to BS 5950, Part 5, 1987, The Steel Construction Institute, Ascot, 1993.
3. Wei-Wen Yu , Cold Formed Steel Structures by, Mc Graw Hill Book Company, 1973.

Structural Steel Design Project	Job No.	Sheet ***1*** of ***2***	Rev.
	Job title: ***Column Design***		
	Worked Example. ***1***		
		Made by ***RSP***	Date ***April 2000***
CALCULATION SHEET		Checked by ***RN***	Date ***April 2000***

COLUMN DESIGN

Design a column of length 2.7 m for an axial load of 550 kN.

Axial load $P = 550\ kN$

Length of the column, $L = 2.7\ m$

Effective length, $\lambda_e = 0.85L = 0.85 \times 2.7 = 2.3\ m$

Try $200 \times 80 \times 25 \times 4.0$ *mm Lipped Channel section*

Material Properties:

$$E = 205\ kN/mm^2$$
$$f_y = 240\ N/mm^2$$
$$p_y = 240 / 1.15 = 208.7\ N/mm^2$$

Section Properties:

$$A = 2 \times 1576 = 3152\ mm^2$$
$$I_{xx} = 2 \times 903 \times 10^4\ mm^4$$
$$I_{yy} = 2\,[124 \times 10^4 + 1576 \times 24.8^2\,] = 442 \times 10^4\ mm^4$$

$$r_{min} = \sqrt{\frac{442 \times 10^4}{2 \times 1576}} = 37.4\ mm$$

Cl. 6.2.2 BS 5950: Part 5

Load factor Q $= 0.95$ *(from worked example 1)*

The short strut resistance, $P_{cs} = 0.95 \times 2 \times 1576 \times 240 / 1.15 = 625\ kN$

$\therefore\ P = 550\ kN < 625\ kN \qquad \therefore$ *O. K*

Axial buckling resistance

Check for maximum allowable slenderness

$$\frac{\lambda_e}{r_y} = \frac{2.3 \times 10^3}{37.4} = 61.5 < 180 \qquad \therefore O.K$$

Structural Steel Design Project	Job No.	Sheet ***2*** of ***2***	Rev.
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	Worked Example. ***1***		
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In a double section, torsional flexural buckling is not critical and thus $\alpha = 1$

Modified slenderness ratio,

$$\bar{\lambda} = \frac{\alpha \dfrac{\lambda_e}{r_y}}{\lambda_y}$$

Ref.2 Table 6

$$\lambda_y = \pi\sqrt{\frac{E}{p_y}} = \pi\sqrt{\frac{2.05 \times 10^5}{208.7}} = 98.5$$

$$\therefore \bar{\lambda} = \frac{1 \times 61.5}{98.5} = 0.62$$

$$\frac{P_c}{P_{cs}} = 0.91$$

$$P_c = 0.91 \times 625 = 569\ kN > P \qquad \therefore O.K$$