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# Influence of Boundary Condition on Cold-Formed Column-Channel Bases Subjected to a Moment and Axial Load

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***Abstract***

*The aim of this study is to explore the initiation of stress concentrations around the base connections of columns made of cold-formed channels, and its influence on the combined ultimate axial and flexural capacity. Recent numerical studies on columns under axial load showed that welding the column's flanges and/or web to the base plate causes significant stresses at the edges of the flange and web elements, which leads to premature failure of the column. In this paper, a finite element model of several cold formed lipped channel cross sections, connected rigidly to the base through the web only, flanges only and web and flanges only, is developed to create different boundary conditions at the bottom of the column. The models are subjected to axial load, and combined axial and moment through an eccentrically applied load. The study is extended to include the effect of the end distance of the welds on the capacity of the column.*

***Keywords:*** Cold-formed, Column, Base Connection.

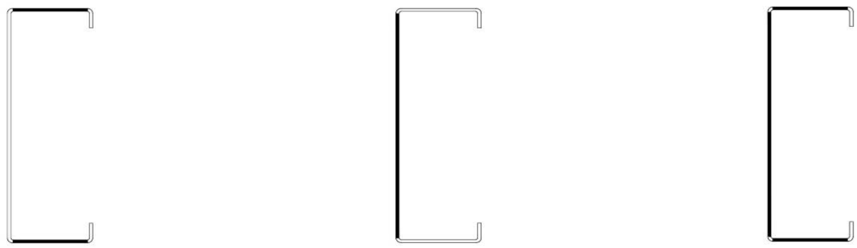
## 1. INTRODUCTION

The use of cold-formed members as the main frames has made it possible to achieve optimum design and construction of industrial buildings, which are composed entirely of cold-formed sections. The advantages of thin walled sections, such as light weight, easy constructability and durability are the reasons for growth in this field. Portal frames as the premier part of industrial sheds have been investigated by Zaharia and Dubina (2006) and Lim et al. (2016). Dundu and Kemp (2006a,b) has studied a full-scale moment resistant frame of cold-formed lipped channels with connected webs at the joint. The channels at eaves were positioned back-to-back and bolted on the web. Although this study was concerned with the ductility of the connection, it also demonstrated acceptable performance of the back-to-back connection in terms of the control of the torsion and simplicity of the joint. Later Dundu (2012) conducted a set of experimental tests on base connections of single lipped channels. The columns were connected to the base either through flanges, web or both using cold-formed and hot-rolled angle cleats, and it was observed that base connectors fabricated from hot-rolled angle cleats could support columns subjected to axial load and moments, because of their rigidity.

Investigations by Kwon, Chung and Kim (2006) and Zhang, Rasmussen and Zhang (2015) focused on cold-formed portal frames with pitched roofs, and reported the effect of the eaves and apex connections as well as instability due to using slender members. Öztürk and Pul (2015) has performed experimental and numerical studies on the back-to-back connected rafters and showed the effect of stiffener plates on local and flexural buckling of the member. The current study investigates the behaviour of a cold-formed lipped channel bases, as part of a portal frame, which is connected back-to-back at the top (Dundu and Kemp (2006a,b)).

# 2. STRUCTURAL CONFIGURATION OF THE BASE

A total of 108 numerical model of cold-formed lipped channel columns were performed. In order to investigate the effect of the base connection assembly on the column capacity, three boundary conditions of the column's base were considered (Figure 1). The first category consists of columns that were welded to the base through flanges only (Figure 1(a)), while the other elements of the cross section such as web, lips and corners were free. The length of the weld was set equal to the flat part of the flange and a weld size of 5 mm. The second category consists of lipped channels that were welded to the base through the flat part of the web with the same size of the weld (Figure 1(b)). The last category comprises of cold-formed columns with both flanges and web welded to the base (Figure 1(c)). Preliminary unpublished work revealed that when a cold-formed lipped channel cross-section is partially connected to the base and the connected area is located at the very end of the column, the capacity of the column under axial load will be affected by the excessive stress concentration and deformation of the area near the connection. Based on the results on this work, an end distance of 30mm was used for all models in this paper.



**(a) Welded flanges** **(b) Welded web** **(c) Welded flanges and web**

**Figure 1 Boundary conditions at the column base**

To study the effect of free elements of the cross-section on the base-connected part of the column, three different sizes of flanges of 50, 75 and 100mm and lips 15, 20, 25mm were considered. However, in order to apply a constant moment and have a criterion to compare the sections, the depth of the web, radius of corners and section thickness were set to the constant dimensions of 250, 10mm and 3 mm, respectively. The length of all columns was selected to be 1000 mm so that no overall buckling happens in the slenderest column. As proposed in the back-to-back connection of the channels, the load was applied at the web plane Dundu and Kemp (2006a,b). The load was applied at four points, namely; at the centre of the web (point P0), to simulate an axial load, at one-third of the web depth (point P1), at the edge of the section (point P2) and at a distance equivalent to the web depth from point P2 (point P3), as illustrated in Figure 2.

# 3. NUMERICAL INVESTIGATION

The commercial finite element software ABAQUS was used to simulate the bases, using non-linear analysis, under Dynamic/Explicit step. A linear 4 node shell with reduced integration points (S4R) was used in the analysis. This element has been used in similar works and has shown reliable accuracy and efficiency (Ting and Lau (2016) and Li and Young (2017)). To obtain more accurate results, mesh size on flat parts of the section were set to 5x5 mm, and refined to 3x5 mm on the curved areas. In order to decrease the analysis time without losing significant accuracy, a mass scale of 100 was applied to the model. Reaction forces were retrieved from a set of Reference Points (RP), which were defined at the center of each welded area. In order to simulate the effect of a back-to-back connection well, a square area of the web, with sides equal to the web depth was chosen, at the top of the column, as shown in Figure 2. All elements inside the square are “Tied” to the RP located at the center of the square. This point is restrained against rotating about the X and Z axis, and prevented from translating along the Y axis. Another Reference Point, which is subjected to 5 mm downward displacement along the Z axis, is defined so as to apply a displacement control loading to the column. This Reference Point is restricted from moving in both the X and Y axis, and is connected to the square's Reference Point with a rigid beam using MPC Constraint option of the software. These two RPs simulate boundary conditions of the both ends of the rigid beam.

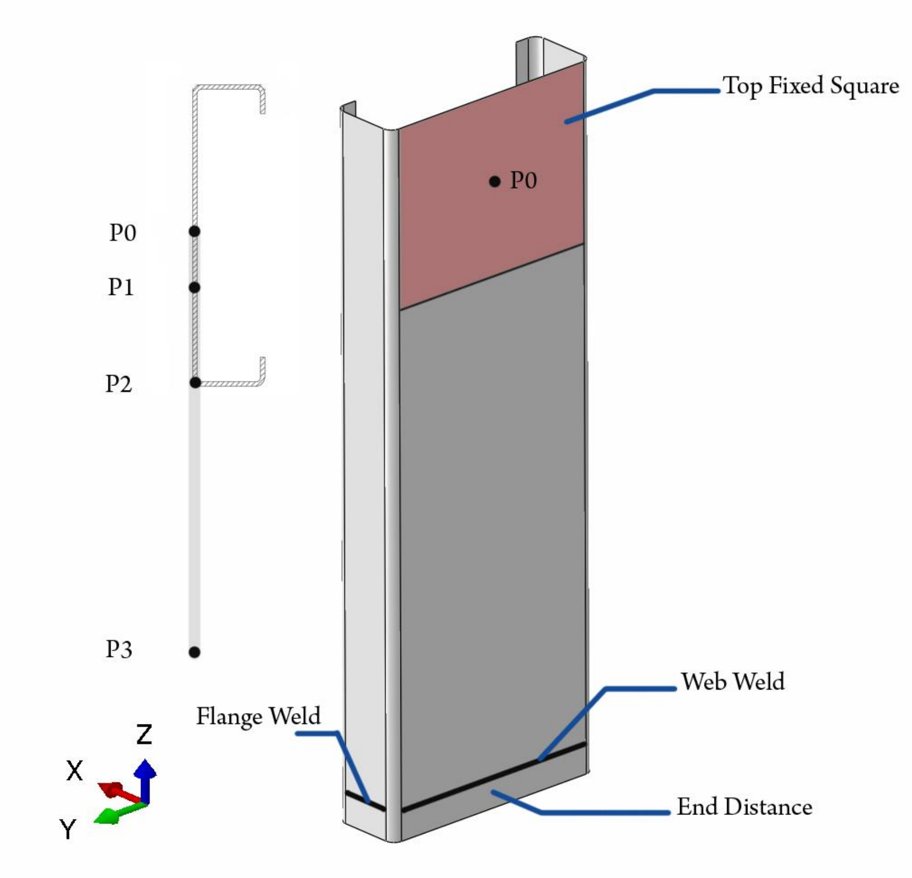



**Figure 2 Typical set-up and loading positions**

## 4. MATERIAL MODELLING

The material properties of the numerical model were obtained by tensile coupon tests. The coupons were extracted from the flat and corner areas of the section before performing the test. Flat coupons in the longitudinal direction of the centre of the web represent properties of the virgin material and corner coupons of the section show the effect of cold working on material properties. The coupons and tensile tests were prepared and tested in accordance with ASTM-E8/E8M-09. The measured average material properties of the web and corner areas are shown in Figure 3.

The material definition in ABAQUS for Dynamic/Explicit analysis incorporates density of the material, modulus of elasticity and poison ratio for elastic and also a multi-line graph to represents the non-elastic behaviour of the material. Table 2 contains all data that was used to define two material properties on the software. Material-1 is obtained from the results of the web coupons and assigned to the flat parts of the cross section, viz; web and flanges, and Material-2 represents the material behaviour of the corners.

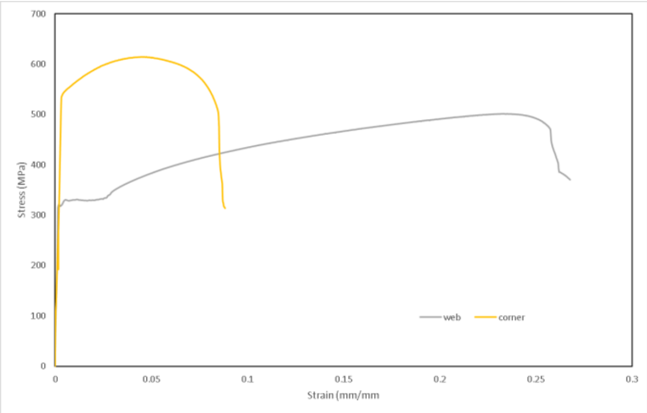



**Figure 3 Typical stress-strain graphs**

**Table 1 - Material properties**

| | Yield Stress (MPa) | Plastic Strain |
|---|---|---|
| **Material - 1** | 330 | 0 |
| | 364 | 0.012 |
| | 426 | 0.062 |
| | 488 | 0.162 |
| | 500 | 0.212 |
| **Material - 2** | 540 | 0 |
| | 590 | 0.017 |
| | 612 | 0.032 |
| | 614 | 0.047 |
| | 620 | 0.087 |
| **Modulus of Elasticity** | 200000 MPa | |
| **Poisson's Ratio** | 0.3 | |
| **Density** | 8.05e-5 N/mm3 | |

# 5. RESULTS

The ultimate load capacity of the columns is given in Table 3, and the mode of failure and deformation characteristics of each category is explained below. In this table, a welded column is defined, for example, by C50-15, where C represents the column, 50 represents the width of the flange and 15 represents the width of the lip. Also, loading condition of the column is followed by base connection category is defined under Assembly column.

**Table 3 - Ultimate capacity of the columns (kNs)**

| Assembly | C50-15 | C75-15 | C100-15 | C50-20 | C75-20 | C100-20 | C50-25 | C75-25 | C100-25 |
|---|---|---|---|---|---|---|---|---|---|
| **F-P0** | 190.42 | 225.94 | 248.33 | 199.97 | 239.97 | 269.03 | 209.28 | 252.55 | 282.89 |
| **F-P1** | 188.44 | 221.94 | 244.49 | 197.44 | 235.41 | 258.60 | 205.78 | 243.83 | 263.32 |
| **F-P2** | 159.10 | 180.6 | 193.75 | 161.21 | 181.89 | 190.57 | 165.13 | 183.55 | 192.9 |
| **F-P3** | 82.40 | 94.51 | 99.39 | 85.90 | 96.68 | 99.85 | 87.74 | 97.13 | 100.49 |
| **W-P0** | 182.94 | 209.64 | 221.34 | 183.52 | 211.80 | 221.93 | 184.67 | 211.75 | 222.60 |
| **W-P1** | 181.86 | 207.06 | 220.13 | 182.70 | 209.94 | 220.89 | 183.42 | 210.99 | 221.69 |
| **W-P2** | 167.46 | 175.92 | 184.61 | 169.89 | 179.81 | 188.32 | 171.87 | 182.08 | 190.67 |
| **W-P3** | 83.77 | 93.3 | 96.72 | 86.85 | 95.54 | 97.56 | 88.97 | 96.51 | 97.97 |
| **WF-P0** | 253.37 | 271.88 | 302.39 | 267.55 | 295.52 | 313.0 | 281.34 | 308.42 | 320.47 |
| **WF-P1** | 233.70 | 262.06 | 291.45 | 239.98 | 272.09 | 300.92 | 252.9 | 284.56 | 309.64 |
| **WF-P2** | 198.03 | 209.24 | 215.95 | 205.23 | 217.41 | 222.6 | 204.38 | 221.70 | 220.88 |
| **WF-P3** | 108.38 | 118.08 | 118.67 | 102.83 | 117.84 | 109.28 | 105.57 | 120.73 | 109.63 |

F – Fixed flanges; W – Fixed web; WF – Fixed web and flanges

## 5.1. Welded Flanges

The results of the finite element models with the flanges only connected to the base indicate that regardless of loading type, there is a small relationship between columns capacity and the size of the lips. Columns with connected flanges, and under pure axial load experience significant distortional buckling above the base connections, leading to the failure of the column. As shown in Figure 4,

bigger lip provides more stiffening and limit distortional buckling. However, the lip-to-flange ratio defines the behaviour of the column.
In the columns with small flanges, the effect of the lip's size on columns capacity is significant when the moment increases, while on the columns with larger flanges, this effect is insignificant. For example, in the cases C50-15 and C50-25, with assembly conditions of F-P2 and F-P3, an increase in the columns capacity of 6kN is realised, while the difference on the cases with larger flanges such as C100-15 and C100-25, with same assemblies, is not considerable. This behaviour can also be seen in the web cases too.

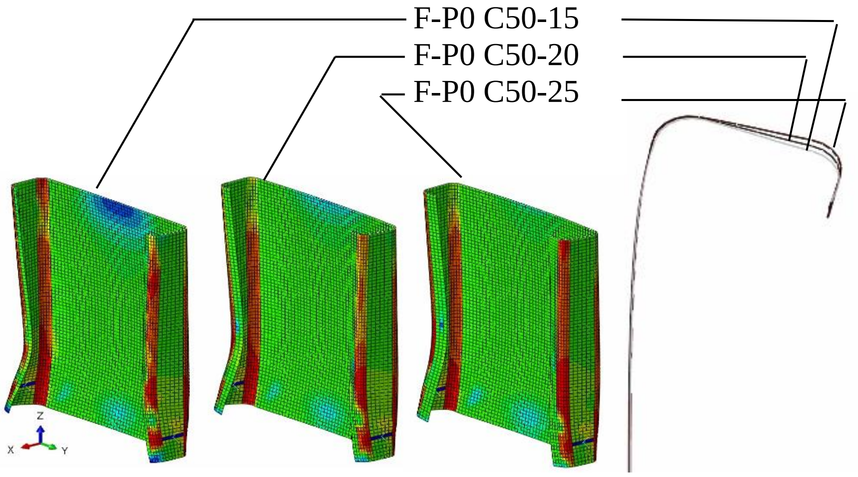



**Figure 4 Effect of the lips on the bottom part of the column**

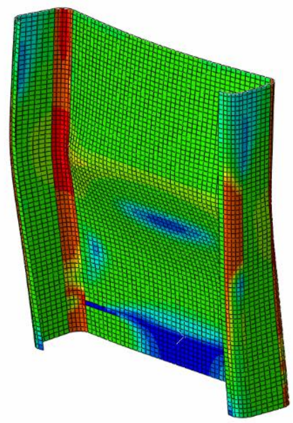


**Figure 5 Local buckling of the web in the bottom part of the column**

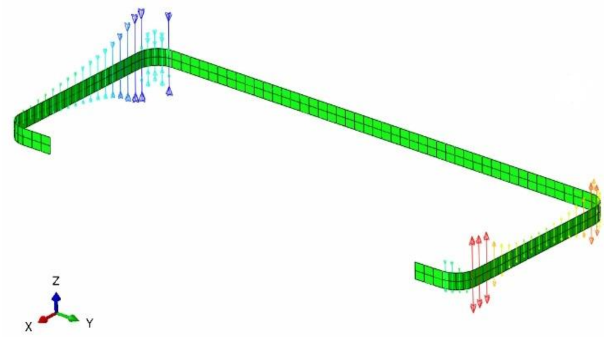
## 5.2. Welded Web

In cases where the web is welded to the base, the length of the lip has less influence on the column's ultimate capacity. When the load's eccentricity is less than half of the column's depth, this case shows lower capacity than the cases with welded flanges, especially when it is under pure axial load. As shown in Figure 5, despite fixing a long area which is more than the flanges' length, local buckling still fails bottom of the column. When the load is applied at point P2, that is, at the edge of the column, local buckling moves to the top of the column, just below the fixed square area. In this condition, columns with welded web show significant improvements in load capacity, compare with the other two cases.

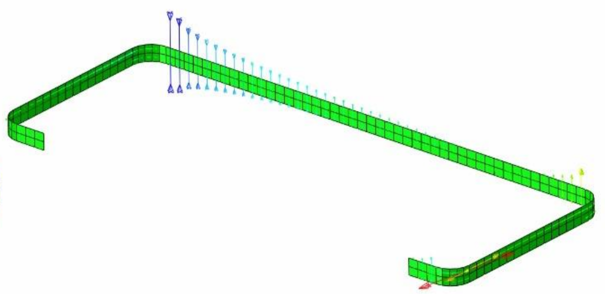
## 5.3. Welded Flanges and Web

Connecting more cross-section elements at the base, created more stiffening of the column's cross-section and also provided more contact area at the base to support the applied load. Columns with welded web and flanges show more load capacity than the other cases. The load capacity of the column is clearly larger than the other cases, and this was more significant when the load was applied within the cross-section of the column, that is at points P0 and P1. Failure of the column, under axial load at point P0, is due to local buckling of the web near mid-height. Applying load at point P1, causes a local buckle at the web and flange closer to the loading point. The position of failure is at the upper half of the column. By shifting the loading point from P1 to the edge of the column's cross-section (P2), the location of failure of the web moved closer to the top of the column. In this situation, there is no significant difference between the maximum load capacity of this column and the one with welded web only. Loading beyond the column's depth, on point P3, yield the results close to the welded web case.

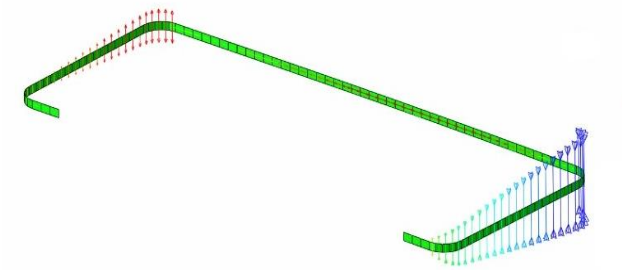
A thorough scrutiny of the WF-P3 base connection, revealed the results shown in Figure 6. Figure 6 (a) and (b) shows in-plane stresses in the flanges and web separately, where outward and inward arrows represent the tension and compression stresses, respectively. This strip of the cross section is extracted from the bottom, at the top of the welds. As shown in Figure 6 (a) and (b), the flange closer to the loading point and a small portion of the web are in tension while the outer flange and the rest of the web are in compression. Another strip from the mid-height of the column (Figure 6 (c) and (d)) shows that the stresses are as expected, that is, the inner flange and a portion of web are in compression and the rest are in tension. Further studies to find an explanation of these forces is under way.



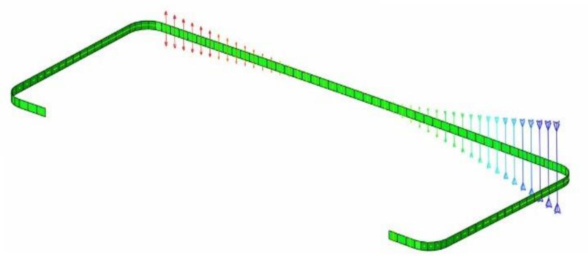

(a) In-plane bottom stresses in the flanges



(b) In-plane bottom stresses in the web



(c) In-plane mid-height stresses in the flanges



(d) In-plane mid-height stresses in the web

**Figure 6 - Stress at the bottom and mid-height of the column**

# 6. CONCLUSION

In this paper, the capacity of the cold-formed lipped channel sections under both axial and the moment has been investigated, using numerical analysis. The connection at the top of the column was set as an ideal representative of a back-to-back connection to simulate the effect of loading through the web while the other elements of the cross section such as flanges, corners and lips at the top of the column are free. In such a situation, an applied moment on the web can cause distortional deformation at the top of the column, which can propagate down the column. Three different base connections, namely; columns with welded flanges only, columns with welded webs only and columns with both welded flanges and webs were examined. Columns with larger welded flanges showed better performance than similar columns with the welded web when loading was inside the cross section. On the other hand, models with welded web and the load is applied at the edge of the section have shown to be more efficient, in terms of ultimate capacity and connection length. Despite the great expectations of column bases in the last category, where both flanges and web were fixed, numerical models have shown that load is not able to pass through base connections, making the connection inefficient.

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