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Effect of Column Deformation for Steel Frames with Semi-rigid Connection

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As semi-rigid behaviour will give a more economical design, it should be applied in the structural steel design. The properties of semi-rigid connection may affect the column design and is the main focus in this study. Complex calculation with semi-rigid consideration in design may cause unfavourable time consuming. Therefore, a simplified column design is needed to eliminate complicated procedures while accommodating the semi-rigid condition. In this analysis, the validated finite element procedure has been used for the analysis of a semi-rigid column in a non-sway frame. Parametric study was performed within variables of types of connection, loading patterns, locations of investigated column and column base fixity. The influences of column slenderness and beam flexibilities were also included and the implications towards the column design were then identified.

Keywords: column; semi-rigid; non-sway frame; finite element; steel frames

I. INTRODUCTION

Realising the potential implication of semi-rigid connections, the codes like Eurocode and American Institute of Steel Construction (AISC) code have introduced provisions to consider this behaviour in the design of structural steel frames. In view of the advantages possessed by the semi-rigid connections, there has been a substantial volume of research studying such behaviour (Díaz *et al.*, 2011; Zhao *et al.*, 2019; Bao *et al.,* 2019; Lee *et al.,* 2015; Lee *et al.,* 2017, Ng *et al.,* 2022) to reduce the complexity in design. It has been claimed that weight savings of more than 11% can be achieved in the case of columns if the inherent stiffness at the connection is taken into account (Bjorhovde, 1984).

While these codes recognise the use of the semi-rigid nature of the connections, but the design guidelines and methods are not specifically detailed. Thus, it fell short of meeting the needs of practising engineers who traditionally wish to use a straightforward method in their design offices.

From previous studies (Davison *et al.,* 1987; Kirby *et al.,* 1992; Gibbons *et al.,* 1993), indicated that columns in simple construction might be designed as axially loaded struts without the inclusion of any allowance for the transfer of moments. This simplified method is known as the α-pin approach. The α-pin factor is a measure of the net benefit of semi-rigid joints on the compressive resistance of a given column. A value of α-pin greater than unity implies that the benefits of the restraining effect due to the semi-rigid connection, which enhanced the column ultimate load outweighs the disadvantageous moments transferred into the column. Hence, the column may be safely designed by just considering axial loads only, as opposed to axial loads and moments.

Simplified approach should be applied for routine design tasks in achieving design efficiency while maintaining its structural integrity. The α-pin factor approach has been only confined to the study of monotonically increasing load to failure. This type of loading is not entirely realistic for many

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applications because frames in buildings are subjected not only to loading and unloading behaviour but also to pattern loading. Hence, the main objective of this study is to investigate various loadings (included dynamic loads) and patterns on columns behaviour. Parametric study will be carried out in order to know the behaviour of column with semi-rigid connections. Previous analytical derivations (Mohammad *et al.*, 2018; 2020) have been recorded and used in this study.

II. PARAMETRIC STUDY

The most convenient approach is to start with the analysis of a basic problem, which serves as a reference for the parametric study. In this case, the basic problem is the column with perfectly pinned-ended connections. The variations of the main parameters are summarised in Table 1. A pinned-end column has been traditionally used as the reference with which real columns are compared and designed. In this case, a column section with a yield stress, length and elastic modulus that corresponds to the one used in the frame is selected. A portal frame configuration with a pinned connection is used to simulate the pinned-ended column behaviour. Initial out-of-straightness is assumed in the form of a half-sine wave with a central deflection of L/1000. The axial load is applied incrementally over a full range of column heights.

Table 1. Main variables for parametric study

Description	**Stage of analysis**		
	Stage 1	**Stage 2**	**Stage 3**
	Monotonic	**Load and unload**	**Extreme**
Connection	Extended end plate, flush end plate, flange cleat		
Column (Fig. 1)	C8, C7, C5, C4, C1	C8, C7, C5, C4, C1	C8, C7
Applied load	Monotonic	Monotonic and slow cyclic	extreme
Frame	2 bays 3 storeys		
Beam length	6.35 m		
Beam section	533x210 UB 122		
Column height	3.4 m, 7.5 m		
Column section	203 × 203 UC 86		
Base fixity	Rigid Pin	Rigid Pin	Pin

A 3-storey, 2-bay typical non-sway frame taken from part of a real building with dimensions and section sizes shown in Figure 1, is chosen to represent the different column categories that are common in steelwork buildings. With reference to this figure, the five columns examined are the internal (C8) and edge column (C7) at ground floor, the internal column (C5) and the edge column (C4) at an intermediate floor level and the edge column (C1) of the top floor. The first stage studies the behaviour of column in semi-rigid frames due to variable monotonic loading. In the second stage, the effect of variable loading and unloading patterns also known as the ‘slow cyclic’ loading pattern are dealt with. Some extreme load conditions are considered in the third stage. The fourth stage of the study examines the column behaviour due to changes in connection stiffness, column slenderness and beam flexibilities in a more systematic manner.

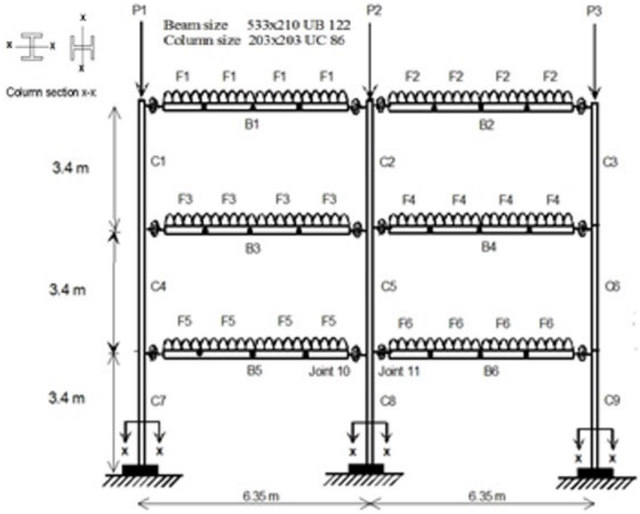

Figure 1. Frame geometry and designation of members

III. RESULT AND DISCUSSION

A. Effect of Column Deformation Due to Variable Monotonic Loading

Consider the internal column C8 of length 3.4 m at ground level, subjected to variable monotonic and slow cyclic loading as well as to the extreme loading cases. The members of these frames were connected using the extended end cleat and connections at the column bases were taken as pinned supports. Consider one of the load-deflection histories of a column, 3.4 m in length subjected to monotonic load sequence. In this case, beams 1 to 5 were loaded simultaneously up to the total design load, 660 kN, while beam 6 was loaded up to 180 kN (dead load). Under practical design conditions, this will induce a typical maximum moment experienced by the beam before transferring to the column. On loading, joint 10 rotates more than joint 11. This is obvious as greater load was applied on beam 5.

Figure 2 shows the relationship between the column load and the column-end moment. It is also noted that, the rate of increase of column deformation reduces when the column end-moment reduces. This effect is actually dictated by the reduction of connection stiffness. Once the beams were loaded to the designated failure load, the increment in column load actually represents increasing load on higher storeys of the frame. As the columns starts to experience this loading, joint 10 starts to unload and follows a path parallel to the initial connection loading path while joint 11 keeps increasing moment but at a very slow rate. The effect of these connection responses in turn reduces the column deformation very slightly. Upon further loading until failure, a sudden drop in column deformation is noticed. This is due to the yielding that occurs at the top and mid-height of column 8. By closely examining the cross-sectional properties, *EI* (flexural stiffness) at the two locations, it was revealed that the top and bottom fibre had progressively yielded to about 5% at the top end of the column and 63% at the column mid-height.

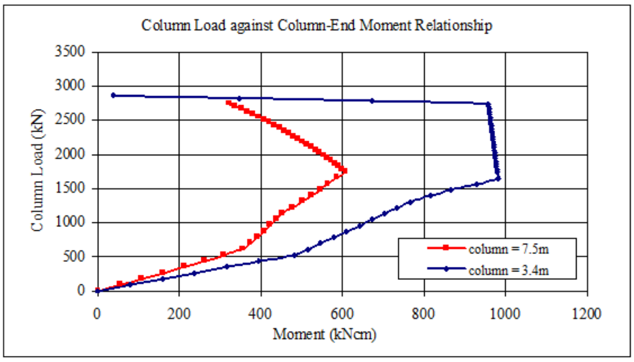

Figure 2. Load-column end moment history for column C8 due to varying monotonic loading

B. Effect of Column Deformation Due to Variable Slow Cyclic Loading

Initially, both beams 5 and 6 are simultaneously equally loaded until stage 1. Hence, there is no moment transferred to the column because joints 10 and 11 are experiencing the same connection behaviour up to stage 1. As the moments cancel each other out, no lateral deformation was also observed at the mid-height of column C8. Once the loading on beam 6 increases and loading on beam 5 decreases, the moment at joint 11 increases. However, at joint 10, the moment decreases because the connection is experiencing unloading behaviour, which is parallel to the initial connection response. Consequently, the column deflects up to stage 2. Then the reverse loading process takes place. Moment at joint 10 increases, while moment at joint 11 decreases.

The beam loading remains but the column is progressively loaded up to 1000 kN. The moment at joint 11 is loading while at joint 10, unloading simultaneously at the same rate. Thus, the moment at the column-end remains. However, as expected, there is a slight increase in column lateral deflection due the axial loading. When the load applied to the column reaches 905 kN, a sudden drop in column end-moment and deflection was observed due to yielding that occurs at the top of column 8. By closely examining the cross-sectional properties at the two locations, it was revealed that the *EI* (flexural stiffness) and *EA* (axial stiffness) values at top and bottom fibre had reduced.

Once the column load reaches 1000 kN, it was then unloaded back to zero, for cyclic load behaviour investigation. A similar but opposite connection behaviour was noted at joints 10 and 11. One important point to note at this unloading stage is the permanent deformation due to yielding that occurs at the top of column C8 and does not follow back previous deformation. This again demonstrates the ability to simulate the inelastic behaviour of the material properties. The columns were then loaded again until failure. A deflection and column end-moment behaviour similar to that at previous load stage was observed. This also applies to the connection behaviour at joints 10 and 11. However, a further decrease in column deformation was noted when column C8 fails at a value of 2857 kN.

C. Effect of Variable Loading on the Column Failure Loads

Another important point to note is the effect of variable gravity loading on the column failure load. Perhaps the most significant findings to note is the percentage difference in column ultimate load for columns C1, C4, C5, C7 and C8. The variable monotonic and slow cyclic loading applied to column C8 with 3.4 m in height that is load stages 1 and 2 respectively are referred. These particular loading conditions represent the practical design load in a typical building. On the other hand, the column slenderness represents the most typical column slenderness used in multi-storey buildings. The first point to note that the failure loads are almost the same if the column is initially subjected to the same beam load before the column is loaded to failure, irrespective of the loading sequence. However, for the two load stages considered, the maximum difference in the failure load is only 1.3% for any connection types and support fixity considered. The differences are expected as the connection behaviour, which in turn affecting the moment transferred to the column, differs when different loading sequences are applied. The different magnitude of beam loading applied to the frame is also a main contributing factor.

As the column slenderness ratio is 36.7 that is considered to be at the lower range, the column failure load due the varying loading applied to the column on a typical building under non-sway case is rather insignificant. Even if applying a much greater beam loading to represent the extreme loading condition (Stage 3), the failure load differs by only 3%. The difference is even smaller when flush-end plates and flange cleats were used to connect the frame members. So far, only column category, C8 was considered. From the same tables, it can be seen that a similar conclusion was arrived for other column categories (C1, C4, C5 and C7). The findings are actually in agreement with the findings by Rifai (1987). He concluded that one of the influences of semi-rigid connection to the column strength is to extend the range of slenderness ratios for which the column may sustain the squash load. In other words, the effects of these restraints are to make the plateaus that are governed by the squash load longer. Now, consider the effect of varying load on a slenderer column represented by column of 7.5 m in length. The maximum difference in the column failure load over the whole loading sequences increases to 4.0% if the column length increases from 3.4 m to 7.5 m. The presence of different magnitudes of beam loading is actually the main contributing factor. The application of a greater beam load causes a greater lateral deflection in the column which increases the effect of the end restraint in reducing the column end moment. Again, the difference is even less when flush-end plates or flange cleats were used to connect the frame members. For columns other than the columns at ground floor level, it can be seen that, regardless of the support fixity at the column base, the failure load of a 3.4 m column is the same when subjected to any loading case. This is also true even if the column length is 7.5 m. This indicates that the effect of rigidity at column bases on the column failure load seems to be insignificance for those columns other than the ground floor column. For columns at the ground level, the above findings also apply for a stocky column that is the 3.4 m column length. This indicates that the failure loads are governed by the stocky effect of the column, outweighs the effect of column fixity. However, a significant difference was observed for the 7.5 m column, which is slenderer, showing the effect of column fixity in reducing the lateral deflection on a slenderer column.

D. Effect of Column Failure Loads in Semi-rigid Frames Due to Different Column Slenderness and Beam Flexibility

As before, the members of the frame are connected using the extended-end plates (EEP), flush-end plates (FEP) or flange-cleats (FC). Six columns of different heights were considered and following the analysis, column strength curves were constructed. Even the most flexible connection considered, which is the flange cleat, gives much higher column strengths than that corresponding to a pin-ended column. Relative to the pinned-ended connection, it can be seen also that there is virtually no effect towards the column strength due to the different type of non-idealised connections considered if the column slenderness ratio is less than 85, where columns strength with lower slenderness may sustain values close to squash load due to the effect of the end-restraint.

Another observation from Figures 3 to 5 is the order of the column failure load. For a stiffer connection, the rate of decrease in column strength tends to be lesser than the one with flexible connections. This is true for the case when the column 12 m in height is connected to a beam of 4 m in length. This particular column height was chosen in order to show the effect of slenderness on the column behaviour. All three types of connections seem to cause little effect on the column failure load for stocky column ($L/r_y < 85$).

Figures 3 and 4 respectively show the column axial load against column deformation and column end moment relationship. It is observed that at the early stage of loading, which is considered to be at elastic region, the stiffer connection would appear to transfer more moment to the column in resulting a larger deflection at column mid-height. However, as the column starts loading, the combination of the unloading effect of the connection, the axial load and the large displacement effect causes the column stiffness to reduce at a slower rate than for the less stiff connection.

As a result, before the column fails, it is observed that this diminishing column stiffness causes the deflection rate to decrease and it reaches the plateau region at a later stage as compared to a less stiff connection. This is not the case if the same column is connected to a longer beam span. The rate of decrease in column strength for columns connected to the extended-end plate is greater than for the flange cleat connection. This also shows the amount of moment transferred to the column is influenced by the connection behaviour together with the effect of the axial load and large displacement (due to column slenderness and beam flexibility).

As the beam span increases, more beam rotation is expected at the beam to column connection, when beam load is applied. This also shows the influence of the column slenderness to the column rotation, which in turn affects the actual rotation at the connection. After applying beam loads and upon further loading the column, the rotation at the connection decreases, which consequently, reduces the moment at the column end. This is illustrated in Figure 5. The reduction follows the path parallel to the initial connection stiffness, and the connection is termed as 'unloading' or 'opening'.

This moment shedding is also noticed by Davison (1987), when he conducted a sub-assemblage experimental study using semi-rigid connections. In this case, the decreasing rate of the column stiffness is greater for the stiffer connection. This in turn causes an increase to the rate of column deformation. Thus, this is in contrast to the earlier findings when the column is connected to beams of 2 m or 4 m in length. The use of a stiffer connection may increase or decrease the column strength, depending on the frame geometry and the particular loading type adopted. The governing factor for this behaviour is actually the column stiffness (KT). This stiffness is dependent on the combination of the small displacement effect stiffness matrix (KE), large displacement matrix (KL) and the current stress level, which account for the effect of axial force (KG).

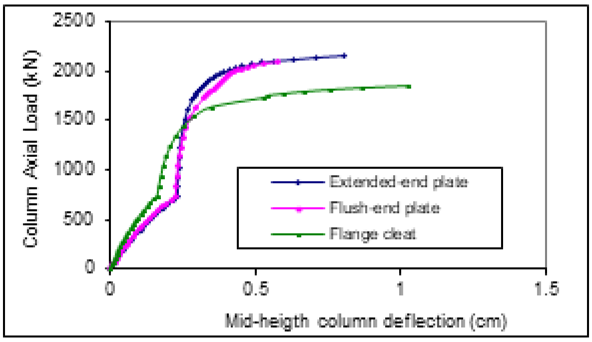

Figure 3. Load-deflection relationship for Column (C7) connected to 4.0 m beam

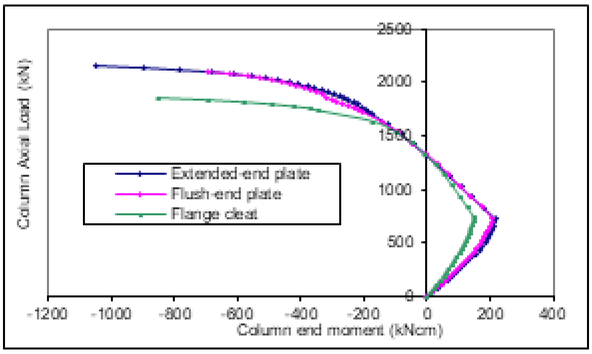

Figure 4. Load against column end-moment for Column (C7) with column slenderness equals to 105 connected to 4.0 m beam

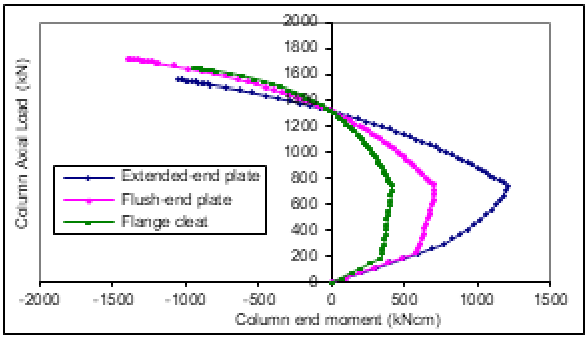

Figure 5. Load against column end-moment for column (C7) with slenderness ratio equals to 105 connected to 9.0 m beam

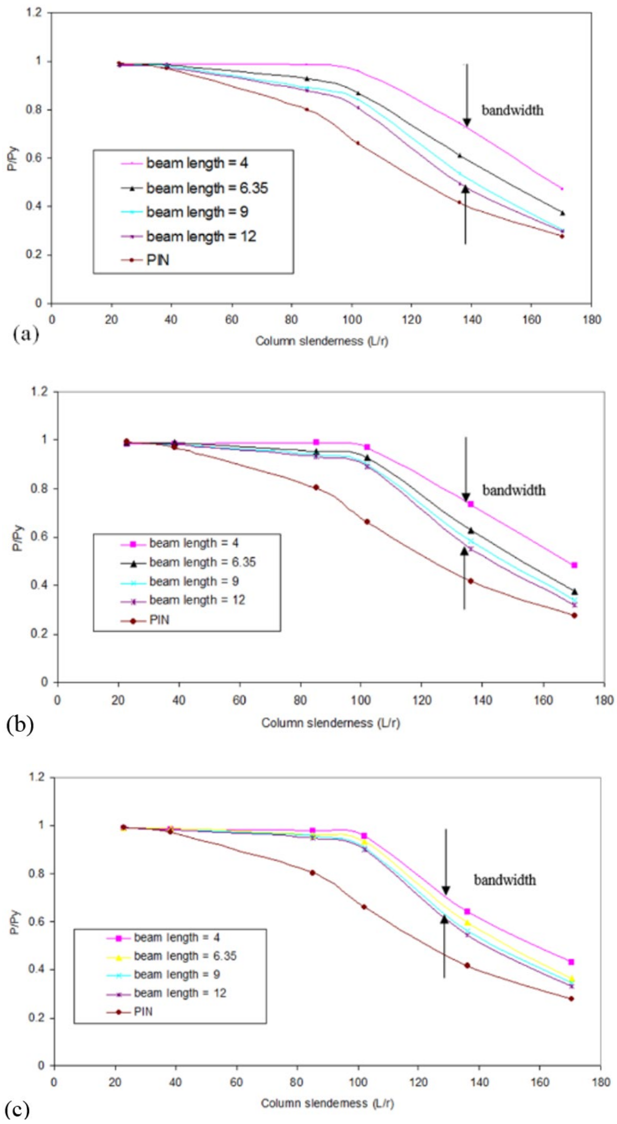

Figure 6. Column strength curves for (a) extended-end plate, (b) flush-end plate and (c) flange plate with different beam spans

Consideration of the strength curve for the same column connected to 4 different beam spans (4 m, 6.35 m, 9 m and 12 m) for the three types of connection, as shown in Figure 6, the column strength curve for the pin-ended column is also included for comparison. Similar behaviour is observed for all these curves. However, for the flexible connection, the beam flexibility has less impact on the column strength as compared to a stiffer connection. This is shown by the reduction in the bandwidth of the column curves for flexible connections corresponding to the different beams span. In other words, the effect of the beam stiffness towards the column strength decreases, with increasing beam span when using flexible connections.

IV. CONCLUSION

In this study, parametric study has been conducted to understand the behaviour of column for semi-rigid joined frames. This has led to the following conclusions,

- The variation of loading sequences is shown to affect the moment-rotation behaviour, which in turn will affect how much moment is transferred to the columns. However, the effect on the column failure load due to these varying loading sequences does not appear to be significant. As the differences in the column failure loads are small, for practical design purposes this parameter can be safely ignored.
- The range of the difference in column failure load of a column in semi-rigid frames subjected to different magnitude of beam load increases with increasing column slenderness. However, this difference seems to reduce when the column is connected to a more flexible connection.
- The use of any practical semi-rigid connections in normal building frames, tends to reduce the column susceptibility to strength loss when compared to an ideal pin connections.
- The beam flexibility has lesser impact on the column strength in columns with flexible connection as compared to stiffer connections.

V. ACKNOWLEDGEMENT

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