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TAJUK PROJEK : **Analytical Studies of the Behaviour of Semi-rigid Non-sway Frames with Tubular Columns, Vot 71645.**

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ANALYTICAL STUDIES OF THE BEHAVIOUR OF SEMI-RIGID NON-SWAY FRAMES WITH TUBULAR COLUMNS

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ABSTRACT

The analytical studies conducted in this research have contributed to a better understanding of the behaviour of inelastic columns, particularly the development of moment shedding and redistribution of moments at the ultimate load levels. It is seen that, as a result of the redistribution of moment, the beams and columns can be treated as isolated members. The beams can be designed as simply supported with a certain degree of end restraint moment to incorporate the effects of semi-rigid connections and the beam-columns as axially loaded. The understanding of the true behaviour of inelastic columns has enabled the development of the simplified design method of semi-rigid non-sway frames. A straightforward method of designing low rise multi-storey buildings for simple, semi-rigid and rigid non-sway frames is known as the α_{pin} approach. The α_{pin} design method has been developed based on the response of loss of stiffness and the phenomenon of moment shedding occurring in the beam-columns.

ABSTRAK

Kajian analitik yang dijalankan dalam penyelidikan ini telah menghasilkan satu kefahaman yang lebih mendalam mengenai kelakuan tiang pada peringkat bukan elastik. Kelakuan yang paling penting adalah fenomena kehilangan momen dan pemindahan semula momen pada peringkat keadaan had muktamad. Dalam kajian ini, dapat diperhatikan, akibat daripada agihan semula momen, rasuk dan tiang boleh dianalisis sebagai anggota berasingan. Iaitu, rasuk boleh dianalisis dan direklabnetuk sebagai bertupang mudah dengan hujung-hujungnya mempunyai momen terikat yang disebabkan oleh keadaan sambungan separa tegar. Manakala, tiang pula direkabentuk sebagai tiang yang menanggung beban paksi sahaja. Kefahaman mengenai kelakuan sebenar tiang bukan elastik, telah membolehkan satu kaedah rekabnetuk Bangunan berbilang tingkat yang dibina secara pembinaan mudah, separa tegar atau tegar ini dikenali sebagai kaedah apin. Kaedah rekabnetuk ini dihasilkan berdasarkan kelakuan kehilangan momen yang berlaku pada tiang.

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Chapter 1

Introduction

1.1 Introduction

The structural system of steel frames in multi-storey buildings consists of main components beams and columns and connections. The latter play an important role in joining the beams and columns and it is well known that the connections show a variation of behaviour in terms of stiffness and strength. This in turn affects the frame behaviour and the way in which the frames are designed.

In traditional methods of design, the connections are normally assumed as either perfectly pinned or perfectly rigid. The assumption of pinned connections implies no rotational continuity within the frame; in other words, no moment is transmitted from the beam to the column. The connections which are normally assumed as pinned are flexible connections such partial depth end plate, web cleat and flange cleat types. The assumption of perfectly pinned connections as normally adopted in non-sway frames may lead to:

- over-estimation of beam moments, resulting in larger, heavier beams
- over-estimation of service deflections in beams
- under-estimation of column end moments.

On the other hand, the assumption of perfectly rigid connections implies full moment continuity. Connections which are normally assumed as rigid are stiffened extended end plate and full welded connection types. The assumption of perfectly rigid connections may lead to:

- over-estimation of column end moment, resulting in larger, heavier columns
- over-estimation of connection moments, resulting in more complex connections which normally require the use of stiffeners at the column web.

The two assumptions described above are merely based on idealistic conditions. In reality, the results of experimental studies show that the connections which are normally assumed as pinned actually possess some rotational stiffness, while connections assumed as rigid often display some flexibility. With this regard, all connections are more correctly to be described as semi-rigid which covers the full spectrum between the two extreme pin and rigid conditions.

The amount of moment transmitted depends on the stiffness of connections and on the stiffness of the connected beams and columns. The benefits of considering the real semi-rigid characteristics are:

- A more rational design method based on realistic action of the connections.
- A more economical design. Research on beam economies shows that the use of double web angles, flush end plates and extended end plates resulted in beam weight saving of the order of 6%, 22% and 29% respectively. Other results show that an overall cost saving for a non-sway planar frame of 5.5% can be achieved if semi-rigid design is adopted instead of simple design. In the case of sway frames, about 20% of saving of the total cost of the frame can be achieved.

Numerous investigations have been undertaken to study the influence of semi-rigid connections on the performance of individual members and global frames for the past 60 years. However a comprehensive semi-rigid design method is not yet readily available in any leading design code. Although, Clause 2.1.2.4(b) BS 5950 has provided a simplified semi-rigid design method, the provisions are somewhat arbitrary and do not realise the full potential offered by semi-rigid design. On the other hand, the methods proposed by various researchers are too complicated for hand calculations and hence are not suitable for routine daily design. Most of the rigorous design methods require the information of M-ϕ curves of the connections which can only be obtained from experimental tests or alternatively by using complex analytical models although work on databanks of both experimental tests and analytical models are being developed. The design methods based on the idealistic assumption as either pin or rigid continue to dominate the way the structures are being designed because of their simplicity.

Chapter 2

Review of Previous Work

2.1 Introduction

Semi-rigid connections and their potential benefits to multi-storey steel frame structures have been the focus of many investigations for the past 60 years. Numerous investigations into the global behaviour of steel frames as influenced by the presence of semi-rigid connections have been reported by many researchers. The studies were mainly focused on steel frames utilising open section columns and open section beams. Lately, however, the use of tubular columns with open section beams coupled with an innovative bolting system has become an alternative for the construction of multi-storey buildings. This chapter presents a brief overview of the main areas that have contributed to the understanding the behaviour of semi-rigid connections, columns and frames. The related main subject areas are:

- semi-rigid connections
- the application of tubular columns in multi-storey buildings
- connections between open beam sections and SHS columns
- analysis of frames with semi-rigid connections
- axially loaded columns with pin ends and axially loaded columns with semi-rigid ends
- beam-columns with end restraints
- phenomenon of moment shedding in beam-columns.

2.2 The Application of Tubular Columns in Multi-storey Buildings

The use of tubular columns in multi-storey buildings is gaining more popularity with architects, engineers and the public. Structures employing tubular members can be more visible and display an elegant aesthetic appearance, and provide more internal space with little intrusion. There are various types of tubular sections suitable for applications as structural members in multi-storey buildings such as circular hollow section (CHS), square hollow sections (SHS) and rectangular hollow section (RHS).

From a structural point of view, SHS columns have been regarded as efficient in carrying axial compressive loads and have been effectively used as columns in low to medium rise buildings. Other benefits of SHS columns as opposed to open sections columns are :

- column sizes can be set to be uniform with increasing storey heights by varying the section thicknesses
- equal flexural stiffnesses in both axes
- high strength to weight ratio.

The design of SHS columns follows the same procedures as designing open section columns as specified by BS 5950. In terms of local buckling, the problem can be avoided by satisfying the limitations of web and flange slenderness ratios specified by Table 7 of BS 5950. The limitations will restrict the use of very thin walled columns which may cause local buckling failure prior to failure by general yielding. McGuire suggested that where tubular columns are used in ordinary bridges and buildings, they are usually of the type for which regular column design procedures apply and the special problems of shell buckling need not be considered.

The main obstacle to the use of SHS columns is the cost and complexity of the connection systems. As compared to open sections, tubular columns are more difficult to manufacture and hence a little more expensive weight for weight. Additionally, traditional connection systems require welding to the column wall which contributes to further difficulty in the fabrication work.

However, realising the many benefits of SHS columns, many research investigations are currently underway to improve:

(ii). the column strength by concrete filling. The results show that the concrete filling can improve the properties of the section, gaining a higher compressive strength, associated with good ductility and large energy absorption.

2.3 Connections Between Open Section Beams and SHS Columns

White and Fang conducted one of the earliest experimental studies on connections between open sections and SHS columns in the mid 1960s. The connection types suggested were mainly welded. At that time, welding was the only practical method because of the closed cross section of the tube. Overall, early developments on open beam to SHS columns were mainly concentrated on welded connection types. The complex fabrication of a welded connection has made its use in practice limited and restricted, more probably due to difficulty in site welding or joints have to be made twice, first in the factory and second on site.

The problem with SHS sections is lack of access to the inside of the tube for the nut, if traditional bolting system is to be employed. A major development to overcome this problem is the use of blind bolting system. A hole may be cut into the hollow section to give access for the blind bolts. This system then allows the installation of bolts from one side only in which nuts are not required as normally used in the traditional bolting.

Tanaka et al. conducted investigations on a new method combining the SHS column with blind bolts. The column wall at the connection zone has larger thickness as compared to other part of the column. The objective is to prevent excessive out-of-plane deformation and to increase the bolt contact length. The thicker wall is obtained by first heating the required wall area to a pre-defined temperature at which the column material can deform plastically. This is then followed by applying compressive forces at a specific speed at both column ends. This caused the heated column to swell.

The blind bolting system described above is a mechanical system which uses expensive components and requires the use of oversized holes. The system also needs special installation equipment which can be more expensive and cumbersome. In contrast, the simplest form of blind bolting system is the flowdrill connection.

2.4 Analysis of Frames with Semi-rigid Connections

This section reviews some of the analytical programs developed by different researchers to study the influence of semi-rigid connections on the behaviour of global frames. Apart from the semi-rigid connections, modern programs also include the effects of geometrical and material nonlinearities. With all these parameters taken into consideration, a more accurate prediction of the true frame behaviour can be obtained. This is useful as the development of current limit states design methods needs a more realistic evaluation of the true behaviour of structures particularly the behaviour at ultimate load levels.

Jones developed a finite element program to investigate the effect of end restraints on a limited individual column model. The parameters of the column such as residual stresses and initial out-of-straightness can be varied in order to study the influence of such parameters to the column behaviour. The connection M-ϕ characteristics are modelled using B-spline technique.

Chapter 3

Nonlinear Inelastic Analysis of Semi-rigid Plane Frames Using Finite Element Method

3.1 Introduction

The nonlinear finite element analysis of plane frame steel structures has become a useful research tool to study the response of real structures. Important aspects such as the effects of semi-rigid connections, geometrical and the material nonlinearities that are present in real frames can now be incorporated into the finite element programs.

3.2 Modelling the Semi-rigid Connections

Numerous forms of semi-rigid connection models which can be employed for analytical studies are discussed in detail by Jones et al.. The B-Spline model occupies a large amount of computer storage. On the other hand the polynomial models can sometimes cause a negative stiffness which the program cannot accommodate.

In the study of non-sway frames carried out in the University of Sheffield, it is observed that, when the individual column model is employed, a small variation in M-ϕ curves can cause a significant different in the behaviour of the structure. However as the portal or multi-storey plane frame model is adopted, it is seen that the variation of $\pm 20\%$ in the M-ϕ curves does not cause any significant effect to the column behaviour. This implies that the precise form of M-ϕ model is less important if a frame is analysed.

In view of the above mentioned aspects, the multi-linear model is used in the current analysis of plane frame structures. This model is simpler to incorporate into the program than other methods such as the B-Spline, exponential, cubic and power models.

In view of the above mentioned aspects, the multi-linear model is used in the current analysis of plane frame structures. This model is simpler to incorporate into the program than other methods such as the B-Spline, exponential, cubic and power models.

Input data for the trilinear model includes the connection rotations ϕ_1, ϕ_2 and ϕ_3 as well as the associated connection stifnesses k_1, k_2 and k_3 respectively.

The stiffness of the connection is evaluated from the data file based on the value of current connection rotation calculated in the program. It is then used in the evaluation of the beam-column element stiffness matrix and the shape functions in which the effects of semi-rigid are incorporated.

The loading and unloading of the connection are determined by checking the values of moments at every load steps in all connections. Any increase in moment indicates loading and any decrease in moment indicates unloading. Tests indicate that unloading occurs with a stiffness which is close to the initial elastic loading value.

3.3 Connection Offset

In the finite element model, the connection can be idealised as located at a concentrated point which may be taken as the intersection between beam-to-column centroid. If this model is adopted the associated forces from beams are transferred directly to columns and the problem of load eccentricity is not present. However, in reality, the connection is made to the column flange and hence the connection can be modelled more accurately as located at an offset from the column centre-line. The presence of this connection offset between the column centre-line and the actual connection location can induce eccentricity moment which may cause significant effects to the column.

A nodal moment equal to the eccentricity moment $M = R \times e$ is applied at the intersection of beam and column centrelines, where R is the beam reaction and e is the eccentricity between column centre-line and column face. The inclusion of this moment will incorporate the offset effect.

3.4 Spread of Yield

A short computer program has been written in the section properties subroutine to evaluate the spread of yield over the cross section at every Gauss points. The spread of points is known, the approximate development of yield spreading along the element can also be quantified.

3.5 Inclusion of Geometrical Imperfection

The shear formulation in this program has been modified to include the effects of P-□and P-Δ in a column. This method has been adopted by Nair, Springfield and Adam and Chen in order to analyse the second order effects in multi-storey frames. Generally, the shear forces due to secondary effects can be obtained by the following equations :

$$V = -\left(\frac{M_{ab} + M_{ba}}{L}\right) + \frac{P\delta}{L} \quad \text{or}$$

$$V = -\left(\frac{M_{ab} + M_{ba}}{L}\right) + \frac{P\Delta}{L}$$

where

M_{ab} , M_{ba} = internal moments in an element

P = member load

L = is the length of the element

The larger deflections and the nonlinear response resulted from the inclusion of P-$\delta$and P-Δ effects are observed. It shows how these secondary effects can influence the response of lateral deflections, in which deflections are important for checking serviceability limit state of a structure.

As a result of modifying the shear formulation above, a further modification has been made to incorporate initial out-of-straightness in columns. Initial work on the program required the application of imaginary lateral loads to induce the initial column shape, which requires several trial and error analyses in order to obtain the required imperfection. The purpose of applying the lateral forces is to increase lateral deflections as column bending increases due to axial loads acting through the initial out-of-straightness. This method, however, can be replaced by the new method which only requires the co-ordinates of the initial column shape associated with the imperfection δ or Δ.

The co-ordinates of the deformed shape of the column can be defined by using a sine wave equation defined as

$$y = e_o \cdot \sin(\pi x) / L$$

where e_o is the initial central deflection, L is the column length and y is the initial deformed shape at a distance x along the column. The value of e_o for SHS tubular columns as investigated by Davison and Birkemoe is approximately L/6000. On the other hand, the mean value of e_o is reported as L/6384 . These values show that the initial out-of-straightness in tubular columns is very small.

The presence of the δ or Δ imperfections in a column, in combination with the axial load, has the effect of introducing additional moments, $P\delta$ and $P\Delta$ respectively. Consequently, additional shear forces $\frac{P\delta}{L}$ and $\frac{P\Delta}{L}$ will develop which will induce additional deflections.

3.6 Treatment of Applied Forces

In this program, four types of applied loads are acceptable which are:

1. A vertical point load acting at a node.
2. A horizontal point load acting at a node.
3. A uniformly distributed load which is applied as a total uniform load acting along an element. In the program this load is simplified into a series of point loads acting at the nodes.
4. A concentrated moment acting at the top end of column element. This moment is also used to model an eccentricity moment due to connection offset.

3.7 Nonlinear Analysis

The formulation of the program has been explained in the above sections. This section will demonstrate the basic procedure of executing the nonlinear analysis in a plane frame structure using the SERIFA program. The main response as a result of this analysis is the load-displacement characteristics. In addition, other important parameters such as the magnitude of collapse loads, the distribution of internal forces, the development of yield spreading and the evaluation of stiffness loss in the structure can also be quantified.

In general, the investigation on the behaviour of structures based on the nonlinear analyses requires that the loadings in the analyses must be applied starting from zero loads and continued up to the collapse loads. Such history of loadings will include the application of incremental applied loads. Consequently, at every load step, the problem of equilibrium is to be solved prior to the next load increment using the Newton Raphson iteration method.

Once a position of equilibrium is achieved, the next load increments can be applied to check for the next load level that can be sustained by the structure. The similar Newton-Raphson iteration processes are performed to obtain the corresponding equilibrium points. In this figure only four load steps are shown for simplicity and clarity of the diagram. In reality more load steps are required to give the true nonlinear load-deflection response. This procedure is also known as the incremental-iterative Newton-Raphson due to the incremental loads and the iterative iterations at every load step.

In the procedure of this program, the nonlinear analysis is completed when the applied load has reached the maximum load carrying capacity of the structure for which the equilibrium condition must be satisfied. The maximum load carrying capacity is known as the collapse load. Any further load increment beyond the collapse load will cause the collapse of the structure in which the structure is no longer in equilibrium. Finer load increments in the higher load level will give a more accurate prediction of the collapse loads.

3.8 Convergence Criteria

In reality, it is impossible to obtain zero values of vector of unbalanced forces at all nodes in the structure. Hence, a criterion to minimise the vector of the unbalanced forces to a required accuracy can be obtained by specifying the convergence criterion.

The two convergence factors to be evaluated in order to achieve equilibrium in the structures are as follows:

(1). The load convergence factor is given as

$$\sqrt{\frac{\sum_{i=1}^{n}\{\Delta Fi\}^2}{\sum_{i=1}^{n}\{Fi\}^2}} \times 100 \leq Tolerance$$

(2). The displacement convergence factor is given as

$$\sqrt{\frac{\sum_{i=1}^{n}\{\Delta Di\}^2}{\sum_{i=1}^{n}\{Di\}^2}} \times 100 \leq Tolerance$$

where

$\{\Delta Fi\}$ = vector of unbalanced forces

$\{Fi\}$ = vector of total applied forces

$\{\Delta Di\}$ = vector of incremental nodal displacements

$\{Di\}$ = vector of total nodal displacements

The convergence is said to be accomplished when both the convergence factors are equal to or smaller than the specified tolerances. In the case of this study, the tolerance of 0.001 was specified in the program for both the load and the displacement convergence factors. Consequently an accurate point of equilibrium is obtained in which internal forces and external forces are in a balanced condition and hence resulting accurate prediction of load-deflection response. A smaller tolerance may be employed and will give more accurate analysis results. However this will require more iterations for each load step which eventually requires a longer computing time. If larger tolerances are used fewer iterations are involved but less accurate results are obtained. Therefore the appropriate tolerances suggested by other researchers should be considered [3-25].

3.9 Failure Criteria

Instability occurs when the vector of displacements $\{D\}$ in Equation $\{D\}=[K_T]^{-1}\{F\}$ becomes very large. This occurs when the tangential stiffness $[K_T]$ is infinitely small which in turn results in a infinitely large inverse stiffness matrix $[K_T]^{-1}$. The corresponding determinant stiffness matrix $[K_T]$ becomes zero or negative indefinite. A zero determinant indicates the exact collapse load whilst a negative determinant indicates that the collapse load of the structure has been exceeded. At this stage the Newton-Raphson iteration process fails to converge which implies that the internal forces can no longer be in equilibrium with the external forces. Structurally, the structure has lost its stiffness and consequently has lost its equilibrium and can no longer sustain additional loads. It is then not possible to increase the load increments and the program is terminated.

The terms of collapse load and collapse of structures used in this study are in accordance with Moy [3-26]. According to Moy, the greatest load at which equilibrium can be maintained is the collapse load. In other words, the collapse load is defined as the maximum load the structure can sustain just before the structure collapses. At the collapse load level the structure is in a state of deformed equilibrium. Analysis results such internal forces, moments and displacements are available.

3.10 Conclusions

A formulation of the nonlinear analysis program based on the finite element method has been presented in this chapter. The formulations in the program include the incorporation of semi-rigid connections, P-delta effects, initial geometrical imperfection and the effects of geometrical and material nonlinearities.

Chapter 4
Ultimate Strength of SHS Beam-columns with Semi-rigid Connections

4.1 Verification in Non-sway Modes

The test frame models have been analysed by the SERIFA program. The results of the analysis are compared directly with the experimental results.

4.1.1 Response of Semi-rigid Frame with Columns Bent about the Major Axes - Response of Frame 1

The response of beams and columns as part of semi-rigid frames with the columns bent about the major axes is presented in Figures 4.1 to 4.2.

Response of Beams

Figure 4.1 shows the progression of load-deflection responses of beam B6 during the beam loading phase. The responses were obtained using the design strength of $\square_y$ = 285 N/mm^2. The load deflection curves are plotted based on the quarter point load versus mid-span deflection of each beam. The comparisons at different beam locations show that the predicted load-deflection responses as obtained from the analysis are in good agreement with the experimental results. This implies that the program is able to provide good predictions of realistic behaviour of beams with semi-rigid connections.

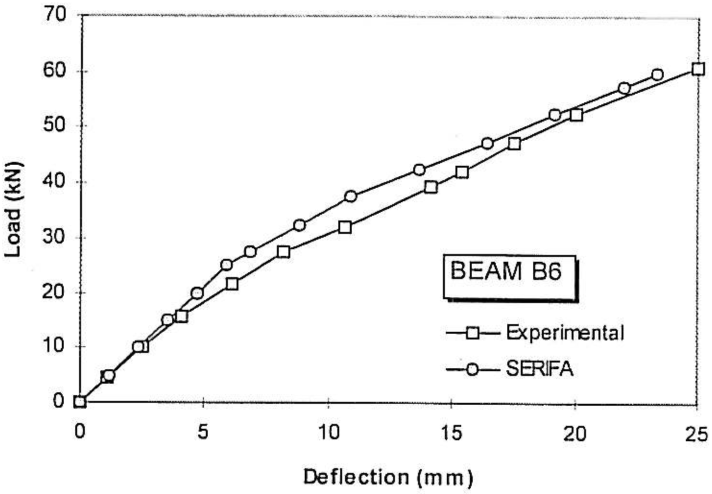


Figure 4.1 Comparison of the experimental and analytical beam load-deflection responses for frame 1

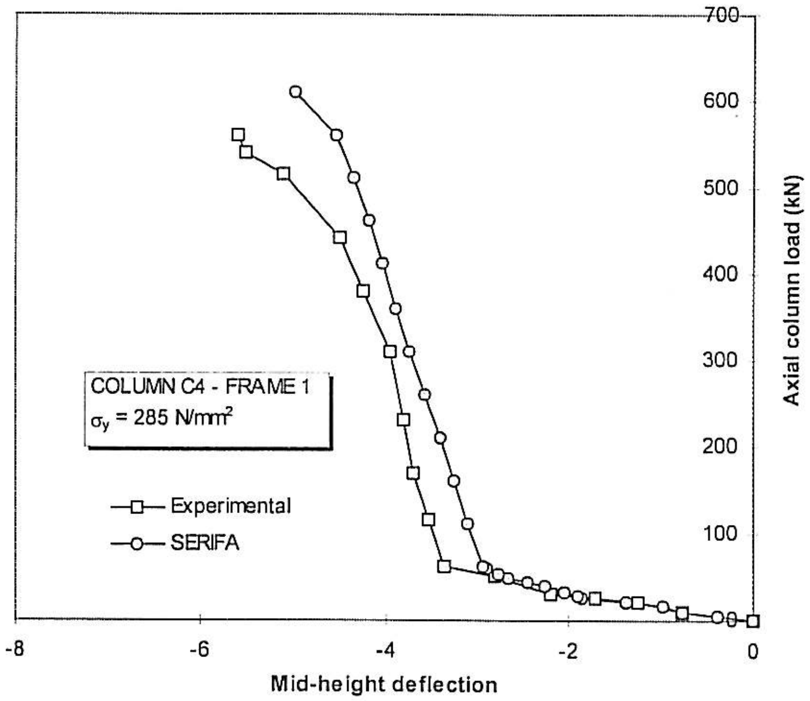


Figure 4.2 Comparison between experimental and analytical load-deflection response using □y = 285 N/mm^2 - Column C4/frame 1

4.2 Behaviour of Internal Beam-columns with Semi-rigid Connections

In presenting the parametric study results, this section discusses the two fundamental responses of load-moment and load-deflection of the internal columns based on the parametric study on frame 3. These responses coupled with the knowledge of behaviour presented in Chapter 5 can provide further understanding of the column behaviour and its significance in design.

4.2.1 Response of Load-Deflection

Figure 4.3 shows the load-deflection characteristics of an internal column with 4m height and the corresponding slenderness ratio of 51.1.

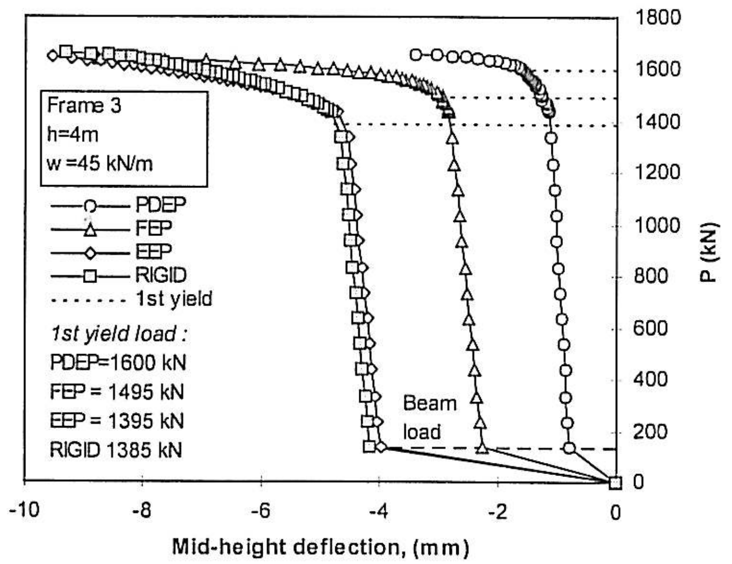


Figure 4.3: Column mid-height deflection

It can be seen that the slopes of load-deflection curves in the elastic range are almost similar for PDEP, FEP, EEP and RIGID connections. This implies that in the elastic range, the stiffness of the column is not much influenced by the connection stiffness. However, a closer observation to the final slope of the load-deflection curves in the inelastic range shows that there is a role played by the various types of connections. It can be noticed that after the formation of first yields, the slopes are much steeper with EEP and RIGID connections as compared to the FEP and PDEP connections. It is evident from this response that the important of connection stiffness is more significant in the inelastic range of the column. The significant contribution of the stiffer connection is to delay the rate of stiffness loss in the column. This response coupled with the shedding of the detrimental moment permits the column with the stiffer connections to sustain larger axial loads in the inelastic range. Hence, the columns with stiffer connections are able to provide larger reserves of strength as compared to the columns with less stiffer connections. On the other hand, it is also seen that the use of less stiffer connections such as PDEP and FEP connections can enhance the column axial failure load to almost close to that of EEP and RIGID connections. This is due to the fact that the use of less stiff connections have some beneficial aspects such as less detrimental moment and the ability to delay the first yield. This in turn permits the column with PDEP connections to sustain high axial failure load.

Figure 4.4 shows the load-deflection response of the more slender column with 7m height and the corresponding slenderness ratio of 89.4.

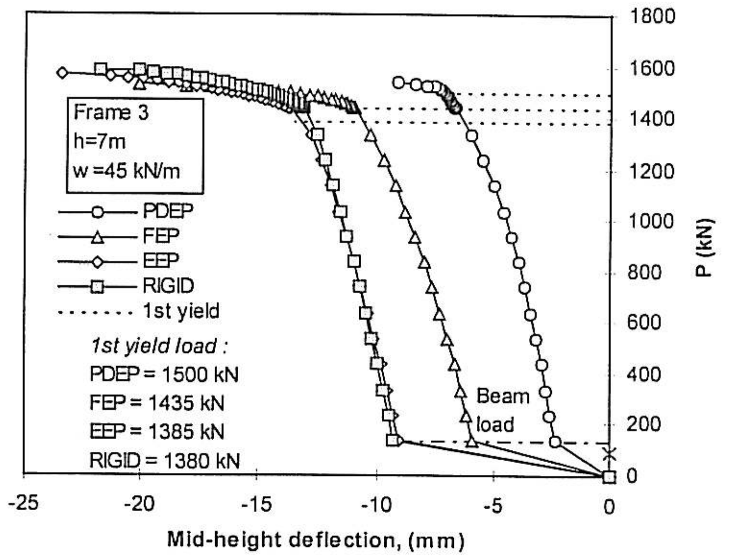
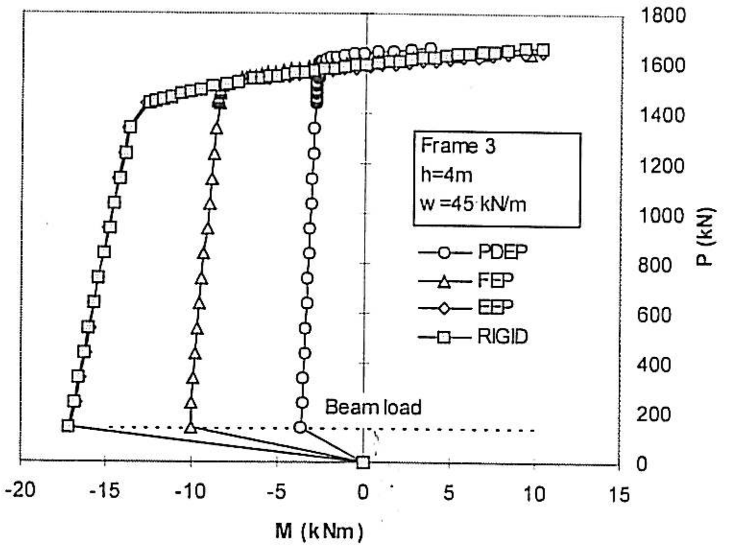


Figure 4.4 Load-deflection response of columns with height of 7m using different connection types

It can be seen that in the elastic range, the slopes with EEP and RIGID connections are slightly steeper than the slopes with FEP connections. However as can be seen from the figure, the stiffening effect is very small and has little significance to the column. Hence, it indicates that increasing connection restraint has little influence on the performance of the slender column in the elastic range.

4.2.2 Response of Load-moment and Moment Shedding

Figure 4.5 shows the load-moment response of the column with slenderness ratio of 51.1.



(b). Moment shedding response of columns with height of 4m using different connection types

Figure 4.5

Similarly, Figure 4.6 shows the load-moment response of the column with slenderness ratio of 89.4.

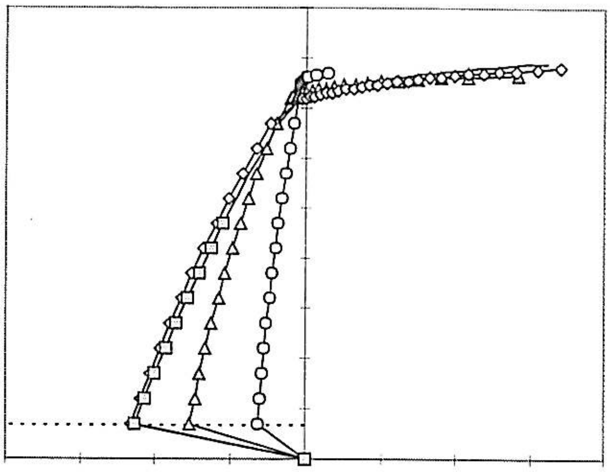


Figure 4.6. Moment shedding response of columns with height of 7m using different connection types

As can be seen from the two load-moment figures, the plots of axial loads against end moments show that the initial detrimental moment is progressively shed with increasing axial load. Eventually, dramatic moment shedding is observed when the column start to undergo yielding.

After the detrimental negative moment is relaxed to zero, the moment will then act as a reverse moment (positive moment). This reversal of moment acts in the opposite direction to the column rotation. This in turn results in the column being restrained further at its ends.

The results of the comparison suggest that:

- The rate of moment shedding is dependent on the rate of stiffness loss. The more rapid is the loss of stiffness, the quicker is the rate of moment shedding.

- Columns with higher slenderness tend to have higher stiffness loss in the elastic range due to geometric deformation. Hence, it can be concluded that the higher the slenderness of the columns the more rapid is the rate of moment shedding.

As a result of a smaller detrimental moment and rapid rate of moment shedding, a slender column will undergo a large restraining moment effect at the collapse load level. This suggests that the more slender columns tend to have larger restraining effect from the reversal of moment. This behaviour is also noted by Gent and Milner.

4.3 Conclusions

The parametric study on the behaviour and strength of SHS beam-columns at ultimate load level has been presented. It is seen that the study on the behaviour of elastic-plastic columns at higher load level leads to a more realistic design method. Moreover, the problem of the detrimental moment acting at the column end which normally occurs at a lower level diminishes as the column loses its stiffness at higher load levels. As a result, the design procedure based on the response at the ultimate load level is nothing more involved than the simple analysis and design of individual members of beams and columns.

Chapter 5

Conclusions and Recommendations

5.1 Introduction

The analytical studies conducted have contributed to a better understanding of the behaviour of inelastic columns, particularly the development of moment shedding and the redistribution of moments at the ultimate load levels. It is seen that, as a result of the redistribution of moment, the beams and columns can be treated as isolated members.

5.2 Conclusions

The main observations and conclusions obtained from the limited parametric studies on low rise multi-storey non-sway frames as conducted in this thesis are:

1. At collapse load, the structure is seen to become statically determinate in which the column behaves as an axially loaded member and the beam as a simply supported member with a certain degree of end restraint moments. As a consequence, beams and columns in frames with semi-rigid connections can be treated as individual members.
2. At the ultimate load levels, the moment acting on the column diminishes as a result of moment shedding and sometime acts as a restraining moment. As a result of this phenomenon, it is not necessary to evaluate the disturbing moment transferred to the column. Hence, based on this justification, the pattern loading necessary to determine the effect of maximum detrimental moment to the columns is not required.
3. The rate of decrease (shedding) of column moment is dependent on the rate of loss of stiffness in the columns. The more rapid is the loss of stiffness, the more sudden is the

moment shedding. The rapid loss of stiffness in the columns has been seen in these studies as being associated with the following parameters:

- *The small values of second moment of area, I.* These parameters normally present in the case of columns buckle in minor axes.
- *The more slender columns.* This parameter causes the rapid loss of stiffness in the elastic range due to the large geometrical deformation.
- *The formation of yielding in columns.* This phenomenon causes the sudden loss of stiffness in the inelastic range as the yielded sections no longer contribute to the stiffness of the column.

5.3 Recommendations

Recommendations for further studies that can be investigated by using the plane frame program are as follows:

- An investigation on the performance of frames as designed by the $\square_{pin}$ design method. This can be performed by first designing the frames with the $\square_{pin}$ design method. Then perform numerical load tests on the designed frames by applying full factored design loads on all beams followed by increasing the loads incrementally up to the frame failure. Frames that are able to carry loads larger than the factored design loads are considered safe and acceptable. Additionally, the behaviour of various frame configurations under these loading conditions can also be investigated.

COMPILATION OF SEMINAR PAPERS

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THE DEVELOPMENT OF THE SIMPLIFIED DESIGN METHOD OF SEMI-RIGID NON-SWAY FRAMES

By

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ABSTRACT

The analytical studies conducted in this research have contributed to a better understanding of the behaviour of inelastic columns, particularly the development of moment shedding and the redistribution of moments at the ultimate load levels. It is seen that, as a result of the redistribution of moment, the beams and columns can be treated as isolated members. The beams can be designed as simply supported with a certain degree of end restraint moment to incorporate the effects of semi-rigid connections and the beam-columns as axially loaded. The understanding of the true behaviour of inelastic columns has enabled the development of the simplified design method of semi-rigid non-sway frames. A straightforward method of designing low rise multi storey buildings for simple, semi-rigid and rigid non-sway frames, known as the approach, is presented in this paper. The α_{pin} design method has been developed based on the response of loss of stiffness and the phenomenon of moment shedding occurring in the beam-columns.

INTRODUCTION

In the current design method of steel columns in multi-storey buildings, procedures for the design are basically based on the interaction equations of beam and columns at lower load levels in which axial load and moments are present. However, due to the moment shedding effects which occurred at ultimate load level in semi-rigidly connected frames, the connections restraining effects outweigh the detrimental moments and therefore the column can be designed as axially loaded member only.

Previous researchers such as Kirby et. al [1], Davison et. al[2] Carr, A.B Abd-Rahman et.al [3,4] and Mohammad[5] have noticed that columns in simple construction may be designed as axially load struts without the consideration of any transfer of moment. This simplified design method is known as the α_{pin} method. The α_{pin} factor gives the net profit ratio of semi-rigid connections on the column compressive resistance and is express as follows.

PARAMETRIC STUDIES

In order to investigate the behaviour and ultimate stregth of columns with semi-rigid end restraints at collapse load level, a single storey one bay frame was examined (see **Figure 1**). The frame was analysed by using a second order elastic-plastic nonlinear finite element program. The sections used were 533 x 210UB x 122 for beam 1 and 203 x 203UC x 86 for all columns. The yield stress and the Young modulus were defined as 265 N/mm^2 and 210 kN/mm^2 respectively. Three types of semi-rigid beam-to-column connections were employed namely Extended End Plate (EEP), Flush End Plate (FEP) and Flange Cleat (FC). The associated moment rotation curves of the beam-to-column semi-rigid connections for the frames were obtained from Buick Davison test[2] (see **Figure 2(a)**). Whereas the moment rotation curves for the column-to-base connections were categorised as flexible (FLEX), moderate (MOD) and stiff (STIFF) and the associated curves are shown in **Figure 2(b).** Furthermore, initial imperfections of L/1000 were applied to both columns.

Figure 3 shows the loading history that has been applied to the frames. Initially all beams were loaded with the total design load. Consequently, with the total beam design load maintained at 160.8kN, the columns were subjected to continuous incremental axial loads (P_1 and P2) from zero to failure.

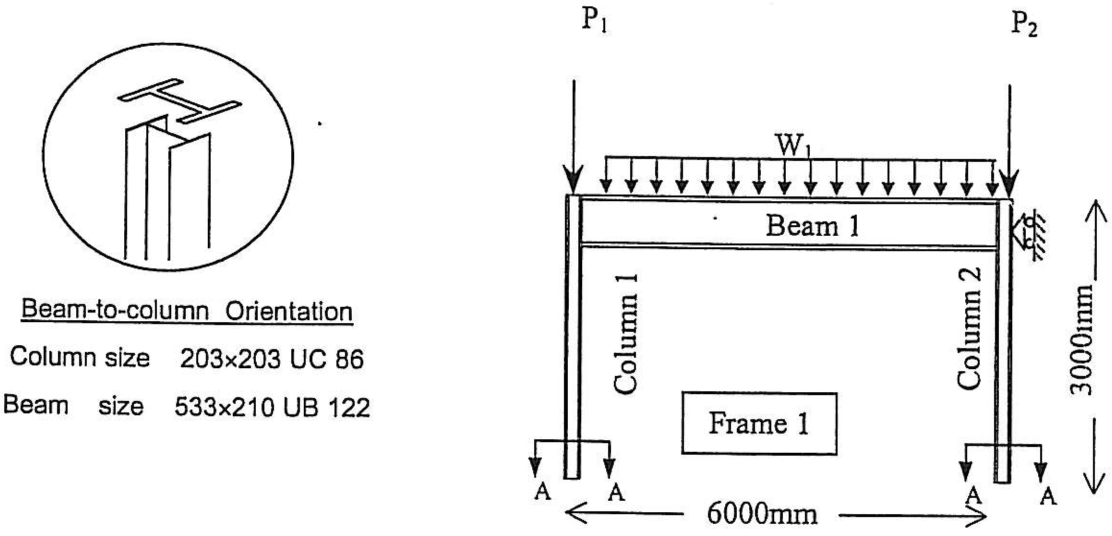
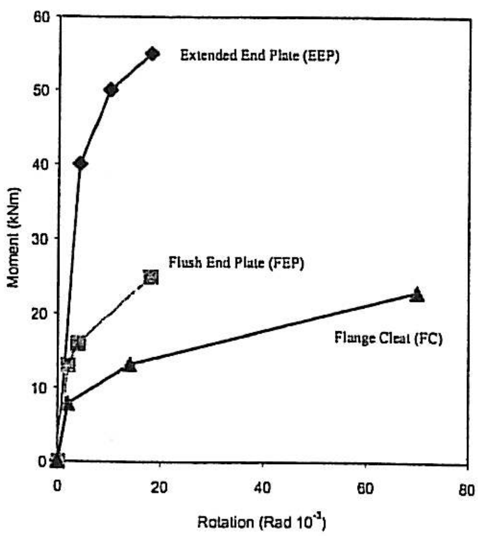
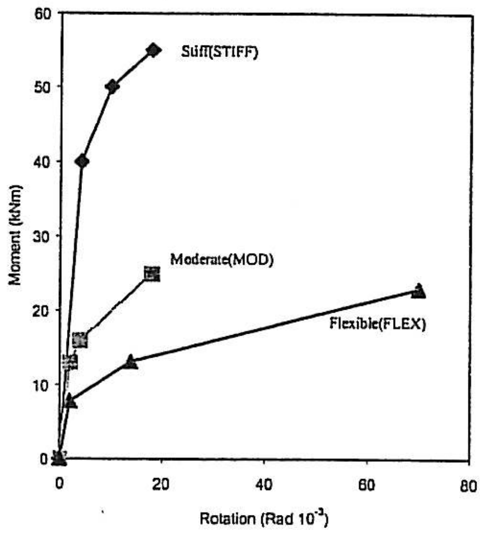


Figure 1 Frame Layout



(a) Beam-to-column connections



(b) Column-to-base connections

Figure 2 Moment rotation curves

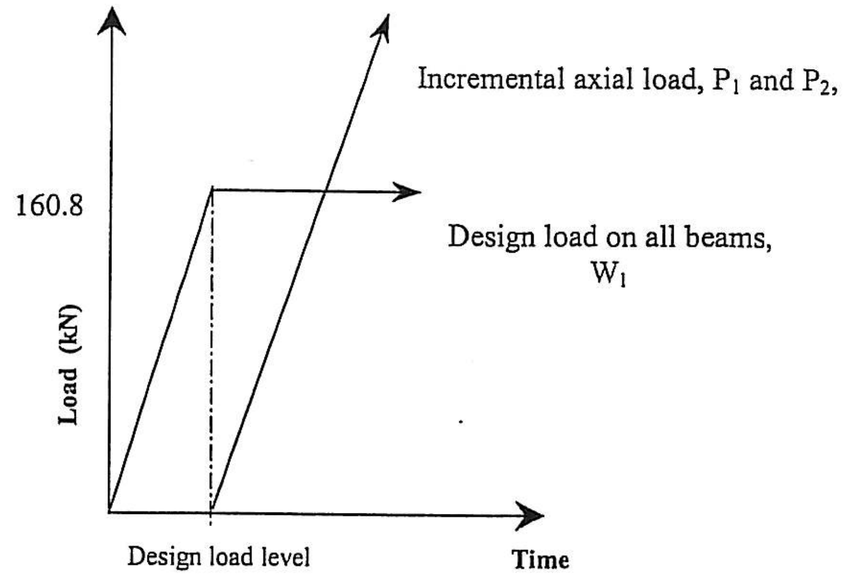


Figure 3 Loading history

RESULTS

Figure 4 shows the load-moment response at the column top end. The important phenomenon to be observed from this figure is the decreasing rate of column top end moment due to moment shedding. This phenomenon has resulted in the change of column bending moment configuration at the ultimate load level.

From the results presented above, it is now recognised that columns with end restraint behave differently at both servis and ultimate load levels. At the lower load level, the moment transfered by the beam causes a detrimental effect to the column. On the other hand at ultimate load level, the column head end momnet instead of being detrimantal has become beneficial by inducing a reversal moment which restrained the column. This significant beneficial effect to the column ultimate failure load is measured in terms of α_{pin} factor. Consequently, in this study, the α_{pin} results are shown in Table 1.

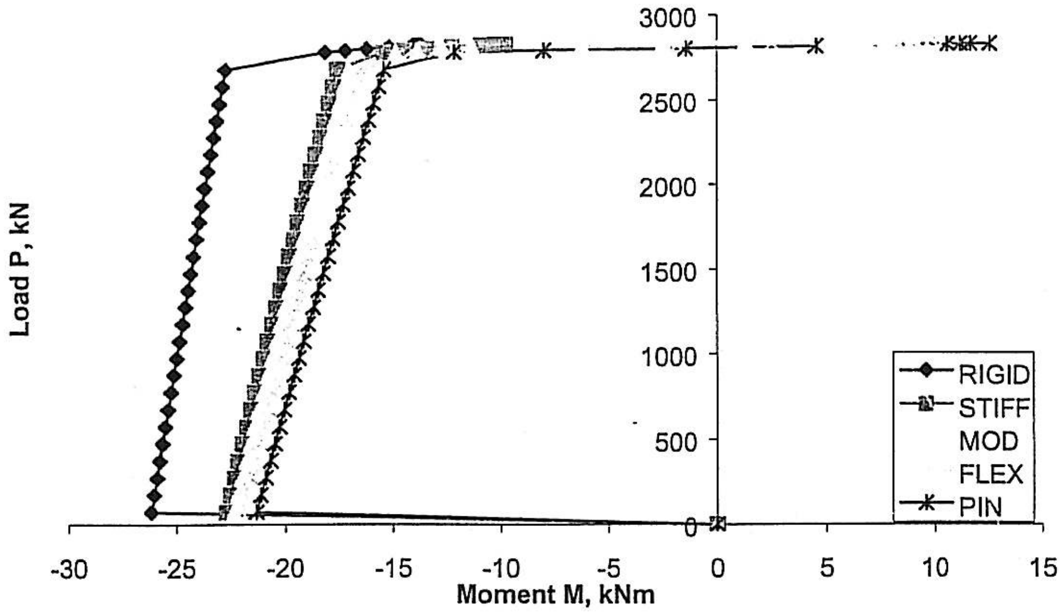


Figure 4 Load-moment response

Table 1 Ultimate failure load of various connection combinations for 3m frames

Types of Connection	Psr, kN	Beam-to-column connections				
		RIGID	*EEP*	*FEP*	*FC*	*PIN*
Column-to-base connections	*RIGID*	2830 (1.03)	2865 (1.04)	2873 (1.04)	2883 (1.05)	2900 (1.05)
	STIFF	2838 (1.03)	2878 (1.05)	2864 (1.04)	2867 (1.04)	2881 (1.05)
	MOD	2905 (1.06)	2871 (1.04)	2855 (1.04)	2864 (1.04)	2877 (1.05)
	FLEX	2895 (1.05)	2846 (1.03)	2850 (1.04)	2859 (1.04)	2873 (1.04)
	PIN	2868 (1.04)	2832 (1.03)	2839 (1.03)	2846 (1.03)	2866 (1.04)

Values in bracket $=\alpha_{pin}$ *value*

In all the cases, it is found that the α_{pin} values are in excess of unity. This indicates that the detrimental moments at lower load level is being overweighted by the beneficial effects from moment shedding. These These results suggest that semi-rigidly restrained beam-columns can be designed safely as axially loaded members using the BS 5950 or EC3 pin ended column curves.

An arrow in **Table 1** shows the trend of the ultimate failure load in which the column failure load increases when stiffer column-to-base connections are employed.

α_{Pin} DESIGN METHOD

It is seen that the study on the behaviour of elastic-plastic columns at higher load level leads to a more realistic design method. Moreover, the problem of the detrimental moment acting at the column end which normally occurs at a lower level diminishes as the column loses its stiffness at higher load levels. As a result, the design procedure based on the response at the ultimate load level is nothing more involved than the simple analysis and design of individual members of beams and columns. This simplified design method, known as the α_{pin} method is described as follows.

Table 2 Effective Length *K* Factor[5]

Support Condition	Effective Length Factor, *K*		
	BS 5090	Theory	Proposed effective length factor, K
	1.0	1.0	1.0
	0.85	0.5-1.0	$\frac{0.7}{1-0.15\left[\frac{P_E}{P_{SQ}}\right]^{1/2}} \le 0.85$
	0.85	0.7	$\frac{0.7}{1-0.15\left[\frac{P_E}{P_{SQ}}\right]^{1/2}} \le 0.85$
	N/A	0.5-0.7	$\frac{0.6}{1-0.20\left[\frac{P_E}{P_{SQ}}\right]^{1/2}} \le 0.85$
	0.7	0.5	$\frac{0.5}{1-0.20\left[\frac{P_E}{P_{SQ}}\right]^{1/2}} \le 0.7$
	N/A	0.7-1.0	$\frac{0.85}{1-0.15\left[\frac{P_E}{P_{SQ}}\right]^{1/2}} \le 1.0$
	$P_E = \frac{\pi^2 EI}{L^2}$ is the Euler Buckling Load, $P_{sq} = A_g.P_y$ is the Squash Load		

The strength of beam-column with length L is evaluated as equal to the strength of pin ended column with the effective length KL. Eventually, the beam-column can be designed as a pin ended if the effective length factor K is known. In order to obtain the effective length factor K, the authors suggest the use of effective length factors as proposed by Carr[6] (see **Table 2**). Then the actual column length is multiplied with this value to get the column effective length. Consequently, the column compressive strength, P_c is obtained from clause 4.7.4 BS 5950:2000. The proposed design method simplifies the current interaction design method given in clauses 4.7.7 and 4.8.3 of BS 5950:2000.

CONCLUSIONS

The parametric study on the behaviour and strength columns with end restraints at ultimate load level has been presented. It is seen that the study on the behaviour of elastic-plastic columns at higher load level leads to a more realistic design method. Moreover, the problem of the detrimental moment acting at the column end which normally occurs at a lower level diminishes as the column loses its stiffness at higher load levels. As a result, the design procedure based on the response at the ultimate load level is nothing more involved than the simple analysis and design of individual members of beams and columns.

ACKNOWLEDGEMENT

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ULTIMATE STRENGTH OF BEAM-COLUMNS USING THE α_{pin} VALUES

by

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ABSTRACT

The parametric study on the behaviour of beam-columns with semi-rigid connections in the elastic and inelastic ranges for non-sway frames has been investigated. The study was carried out using a second order non-linear elastic-plastic finite element program incorporating the use semi-rigid elements. The problem of interaction between the axial load and moment which occurs at the lower load level diminishes as the ultimate load level is approached as a result of moment shedding phenomenon. This response demonstrates that beam-columns can be treated as axially loaded members. In view of the above, the values of the ultimate strength of the beam-columns obtained from the analysis can be utilised to relate the ultimate strength of semi-rigid end restrained beam-columns to the ultimate strength of the pin ended columns. In other words, the failure load of the beam-columns which are treated as axially loaded at ultimate load can be compared directly to the ultimate axial failure load of the pin ended columns. Consequently, the ultimate strength of beam-columns can be presented in terms of the α_{pin} values.

Introduction

A simplified design method called the α_{pin} has been developed at the University of Sheffield, UK (Kirby 1992). The α_{pin} is defined as

$$\alpha_{pin} = P_{sr} / P_{pin} \tag{1}$$

in which P_{sr} is the ultimate axial load (collapse load) of semi-rigidly connected column, P_{pin} is the ultimate axial load of equivalent perfectly pin ended column. Previous studies on the α_{pin} method have been related to the open section columns (Gibbons 1991), (Kirby 1992). The main criteria of this design method is that the effect of acting moments are taken as being outweighed by the column end restraint and hence the column can be designed as axially loaded. Recently, based on this study, the concept of α_{pin} method has been adopted in the scheme design of semi-continuous braced frames published by the Steel Construction Institute, UK (SCI 1997).

The use of the α_{pin} method is further investigated by the authors for possible use in frames with tubular square hollow section (SHS) columns. The results of the study indicate that the α_{pin} design method can be used for simple, semi-rigid and rigid non-sway frames with the tubular SHS columns.

Basis of the α_{pin} Method

The basis of the proposed α_{pin} method is the phenomenon of moment shedding and the response of inelastic columns at the ultimate limit state with the beams remaining elastic (see Figure 1(a)). The response of beam-columns shown in Figure 1(b) was investigated using a second order non-linear finite element program incorporating the semi-rigid elements (Abd-Rahman 2000).

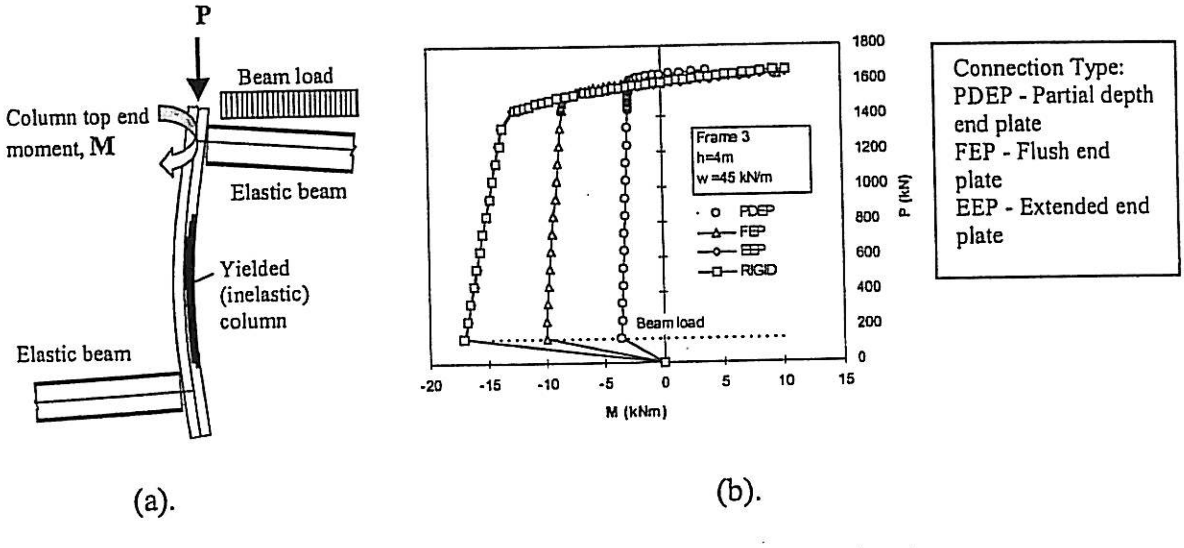


Figure 1 Response of beam-columns at ultimate load level

The results of the study carried out using the second order finite element program show that when the column starts to become inelastic, the column end moment is dramatically shed and relaxed to zero (see Figure 1(b)). This is followed by the redistribution of moments from the yielded column to the neighbouring members. This in turn resulted in a simple configuration of internal forces in the frame members. Eventually, the frame members can be rendered as statically determinate in which the columns can be treated as axially loaded and the beams as simply supported with a certain value of end restraint moment. The results of the analytical studies show that the eccentricity moments due to beam reactions acting at the column face are also subjected to moment shedding and eventually relaxed to almost zero. The phenomenon of moment shedding has been observed in the experimental results of both semi-rigid and rigid frames (Moore 1993), (Baker 1956).

As a result of the above phenomenon, the α_{pin} design method neglects the moments resulted from the eccentric load and that transmitted by the connections.

The idealisation of the frame for the α_{pin} design is discussed as follows:

- Figure 2(a) shows a semi-rigid non-sway frame carrying gravity loads. The corresponding internal moments in the frame due to working load are shown in Figure 2(b).
- In order to obtain the response of the inelastic column with the beam remaining elastic, a series of incremental loads is applied at the column heads of the top storey columns to cause failure of the lower columns. When the lower columns fail as the ultimate load is reached, the corresponding internal moments of the frame that can be rendered as statically determinate are shown in Figure 2(c).
- At ultimate load, the column is not subjected to the acting moment as the moment is already shed to the adjoining members. As a result, the internal moment of the column is similar to the moment of axially loaded columns. Consequently, the internal moment of the beam which is affected by the failed column is in the form of the bending moment of a simply supported beam with end restraints.

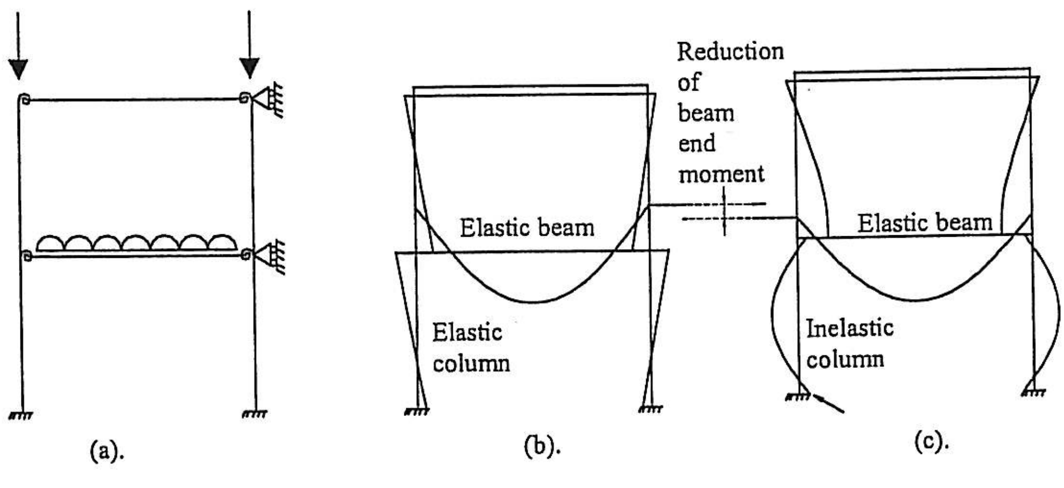


Figure 2 Idealisation of non-sway semi-rigid frame for the simplified design method

Analytical Studies On The α_{pin} Approach

The problem of interaction between the axial load and moment which occurs at the lower load level (see Figure 2(b)) diminishes as the ultimate load level is approached as a result of moment shedding phenomenon. As a consequence, the bending moment diagrams of beam-columns at collapse load are analogous to the bending moment diagrams of axially loaded columns with end restraints (see

Figure 2(c)). This response demonstrates that beam-columns can be treated as axially loaded members.

In view of the above, the values of the ultimate strength of the beam-columns obtained from the analysis can be utilised to relate the ultimate strength of semi-rigid end restrained beam-columns to the ultimate strength of the pin ended columns. In other words, the failure load of the beam-columns which are treated as axially loaded at ultimate load can be compared directly to the ultimate axial failure load of the pin ended columns. Consequently, the ultimate strength of beam-column can be presented in term of α_{pin} values as shown in Table 1. Further information on the method of carrying out the analytical studies to obtain the α_{pin} values shown in Table 1 can be obtained from reference (Abd-Rahman 2000).

Table 1 The α_{pin} values

SUMMARY OF α-pin VALUES
INTERMEDIATE COLUMN (C5)
Connection type : EEP (Test 19)

Frame 3 - Case 4

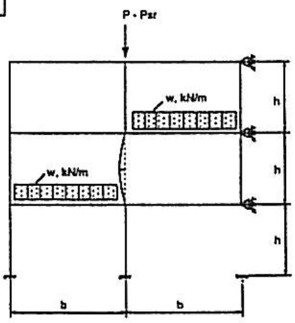
b = 6m

All beams
356 x 171 x 45 UB
Grade 43A

2nd & 3rd storey columns
200 x 200 x 8 SHS
Grade 43A

1st storey columns
200 x 200 x 10 SHS
Grade 43A

Beam elements = 4 nos.
Column elements = 4 nos.



w = 30 kN/m			
	(1)	(2)	(1) / (2)
Column length h (m)	Psr SERIFA e_o = L/3000 (kN)	Ppin BS 5950 (kN)	α-pin BS 5950 e_o = L/3000
2	1680	1650	1.02
3	1680	1600	1.05
4	1670	1530	1.09
5	1655	1430	1.16
6	1635	1290	1.27
7	1605	1110	1.45

w = 45 kN/m			
	(1)	(2)	(1) / (2)
Column length h (m)	Psr SERIFA e_o = L/3000 (kN)	Ppin BS 5950 (kN)	α-pin BS 5950 e_o = L/3000
2	1675	1650	1.02
3	1665	1600	1.04
4	1650	1530	1.08
5	1625	1430	1.14
6	1600	1290	1.24
7	1565	1110	1.41

w = 30 kN/m			
	(1)	(2)	(1) / (2)
Column length h (m)	Psr SERIFA e_o = L/3000 (kN)	Ppin SERIFA e_o = L/600 (kN)	α-pin SERIFA e_o = L/3000
2	1680	1641	1.02
3	1680	1602	1.05
4	1670	1548	1.08
5	1655	1464	1.13
6	1635	1338	1.22
7	1605	1161	1.38

w = 45 kN/m			
	(1)	(2)	(1) / (2)
Column length h (m)	Psr SERIFA e_o = L/3000 (kN)	Ppin SERIFA e_o = L/600 (kN)	α-pin SERIFA e_o = L/3000
2	1675	1641	1.02
3	1665	1602	1.04
4	1650	1548	1.07
5	1625	1484	1.11
6	1600	1338	1.20
7	1565	1181	1.35

Notes :
(i),(ii) : α-pin values based on Ppin (BS 5950)
(iii),(iv) : α-pin values based on Ppin (SERIFA program)

The use of α_{pin} values enable the following assessment to be made:

1. When $\alpha_{pin} > 1.0$

- The beneficial effect of connection restraint outweighs the detrimental effect of the column moment.
- The strength of beam-columns with semi-rigid connections is greater than the strength of the equivalent pin ended axially loaded column.
- The beam-column can be designed as axially loaded member without the consideration of moment.

2. When $\alpha_{pin} = 1.0$

- The beneficial effect of connection restraint is equal to the detrimental effect of the column moment.

- The strength of beam-columns with semi-rigid connections is equal to the strength of the equivalent pin ended axially loaded column.
- The beam-column can be designed as axially loaded member without the consideration of moment.

3. When $\alpha_{pin} < 1.0$

- The detrimental effect of the column moment outweighs the beneficial effect of the connection restraint.
- The strength of beam-columns with semi-rigid connections is less than the strength of the equivalent pin ended axially loaded column.
- The beam-column should be designed using interaction equations considering both the axial load and the detrimental moment.

The justification of designing beam-columns as axially loaded compression members is based on the behaviour of beam-columns discussed in reference (Abd-Rahman 2000). Consequently, based on the α_{pin} values, in-depth studies on the ultimate strength of beam-columns with respect to various types of parameters can be carried out.

4 Conclusion

Results of the study show that beyond yielding the configuration of moment in the frame has changed dramatically due to the phenomenon of moment shedding. The configuration of moments at ultimate load shows that a beam-column with end restraints can be treated as an axially loaded compression member.
Examination of the values of α_{pin} for 180 different cases shows that the values of α_{pin} are always greater than unity. This reflects that the strength of beam-columns with semi-rigid connections exceed the axial load capacity of the column with pinned ends as calculated based on BS 5950.

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TUBULAR COLUMNS IN MULTI-STOREY STEEL BUILDINGS

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ABSTRACT

The use of tubular columns in multi-storey steel buildings is gaining more popularity with architects, engineers and the public. Structures employing tubular members can be more visible and display an elegant aesthetic appearance, and provide more internal space with little intrusion. This paper concerned on the subject areas related to Square Hollow Section (SHS) steel columns in multi-storey buildings. From a structural point of view, SHS columns have been regarded as efficient in carrying axial compressive loads and have been effectively used as columns in low to medium rise buildings. The main obstacle to the use of SHS columns is the cost and complexity of the connection systems. A major development to overcome this problem is the use of flowdrill connection system. The connections can be site bolted without special machines or skilled workers. The paper reports the analytical results from a study to examine the possibilities of using SHS columns with flowdrill connections in multi-storey steel buildings.

Keywords: flowdrill connections, semi-rigid connections, non-sway frames

1 Introduction

The use of tubular columns in multi-storey buildings is gaining more popularity with architects, engineers and the public. Structures employing tubular members can be more visible and display an elegant aesthetic appearance, and provide more internal space with little intrusion. There are various types of tubular sections suitable for applications as structural members in multi-storey steel buildings such as circular hollow section (CHS), square hollow sections (SHS) and rectangular hollow section (RHS). This paper, however, only concerned on the subject areas related to SHS columns in multi-storey buildings.

2 The Use of Tubular SHS Columns in Multi-storey Buildings

The typical use of tubular SHS columns in multi-storey buildings is as illustrated in Figure 1.

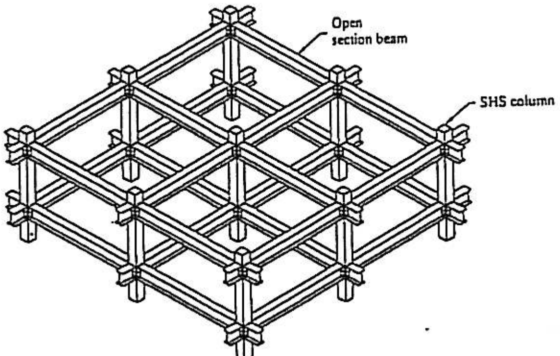

Figure 1 Typical structural system of multi-storey frames with open section beams and SHS columns

From a structural point of view, SHS columns have been regarded as efficient in carrying axial compressive loads and have been effectively used as columns in low to medium rise buildings. Other benefits of SHS columns as opposed to open sections columns are:

- column sizes can be set to be uniform with increasing storey heights by varying the section thicknesses
- equal flexural stiffnesses in both axes
- high strength to weight ratio.

The design of SHS columns follows the same procedures as designing open section columns as specified by BS 5950 [1]. In terms of local buckling, the problem can be avoided by satisfying the limitations of web and flange slenderness ratios specified by Table 7 of BS 5950. The limitations will restrict the use of very thin walled columns which may cause local buckling failure prior to failure by general yielding. McGuire [2] suggested that where tubular columns are used in ordinary bridges and buildings, they are usually of the type for which regular column design procedures apply and the special problems of shell buckling need not be considered.

The main obstacle to the use of SHS columns is the cost and complexity of the connection systems. As compared to open sections, tubular columns are more difficult to manufacture and hence a little more expensive weight for weight. Additionally. traditional connection systems require welding to the column wall which contributes to further difficulty in the fabrication work.

2.1 Connections between Open Section Beams and Tubular SHS Columns

White and Fang [3] conducted one of the earliest experimental studies on connections between open sections and SHS columns in the mid 1960s. The connection types suggested were mainly welded. At that time, welding was the only practical method because of the closed cross section of the tube. Researchers such as Shanmugam et al. [4] extended the scope of the investigations by providing external stiffeners between open beams and SHS columns. Such connections are still very expensive and not ideal for actual constructions. Overall, early developments on open beam to SHS columns were mainly concentrated on welded connection types. The complex fabrication of a welded connection has made its use in practice limited and restricted, more probably due to difficulty in site welding or joints have to be made twice, first in the factory and second on site.
The problem with SHS sections is lack of access to the inside of the tube for the nut, if traditional bolting system is to be employed. A major development to overcome this problem is the use of blind bolting system. A hole may be cut into the hollow section to give access for the blind bolts. This system then allows the installation of bolts from one side only in which nuts are not required as normally used in the traditional bolting. Figure 2 shows the blind bolting system without the need to increase the column wall thickness.

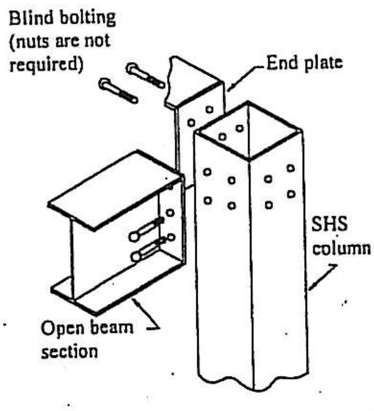

Figure 2 Blind bolting to column walls

The simplest form of blind bolting system is a flowdrill connection.

2.2 Flowdrill Connections

When sections with high wall thicknesses are used, threads may be tapped directly into the column wall but for many SHS sections wall thickness are too small for tapped holes to provide adequate thread length. In this case, flowdrilling can be utilised. In the past, the flowdrill system is used in car production, metal furniture and household electrical appliances. Currently, the technique has been used in the structural steelwork joints due to the development of a new tungsten carbide drill bit that can be used for hard material.

Flowdrilling is a process which consists of forming a hole using a plain tapered cone which simultaneously increases the wall thickness of the SHS section locally. Figure 3 shows the process and the procedures are described as follows:

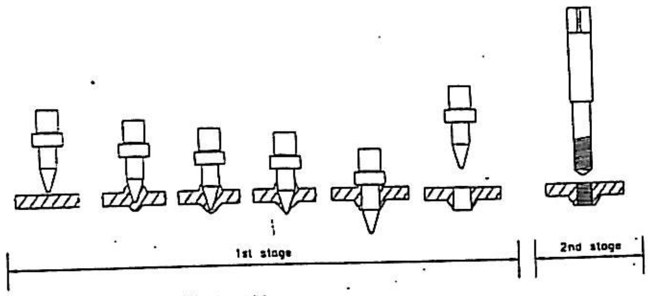

Figure 3 Flowdrilling process

1st. stage:

(i). A special tungsten carbide drill bit that operates at very high speed is forced into the material.

(ii). When the drill comes into contact to the column wall, it produces high heat and consequently softens the SHS section locally.

(iii). The drill bit is then forced through the steel section to produce a hole.

(iv). The softened material displaced by the drill bit will form a conical extension mainly on the inside of the tubular section and finally increases the tube thickness locally by extrusion. The material forming an upstand on the exterior of the tube is removed by a cutter on the first tool to leave a flush surface.

(v). The completed hole is a truncated cone shape with sufficient depth to permit an adequate thread to be tapped for standard bolt application.

2nd. stage:

(i). The thread in the hole is made in a cold formed process by using a different tool.

Currently, the wall thickness of the SHS columns that is suitable for the flowdrill connection system is in the range of 6mm up to 12.5mm. The connections can be site bolted without special machines or skilled workers. The bolts can be fastened from the outside of the columns without access to the internal hollow sections. Beams with conventional end plates can be bolted to the SHS columns easily using Grade 8.8 bolts. Figure 4 shows the installation of the flowdrill connections as viewed from the cross section.

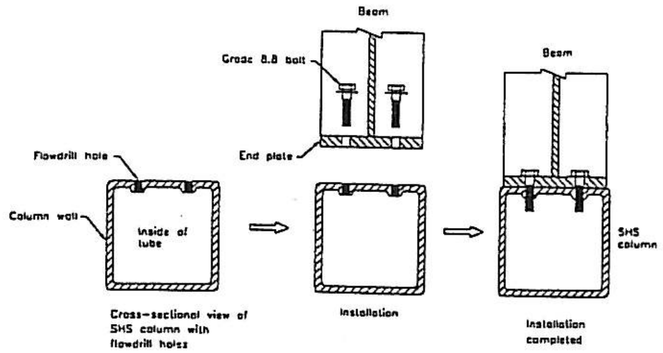

Figure 4 Installation of flowdrill connections

France [5] conducted a total of 25 beam-to-column flowdrill joint tests comprising three major connection types namely partial depth end-plate (PDEP), flush end plate (FEP) and extended end plate connections (EEP). The objective is to study the performance of flowdrill connections for possible use in joining beams and columns in low-rise non-sway and sway frames. One of the experimental results carried out by France is shown in Figures 5(a) and 5(b).

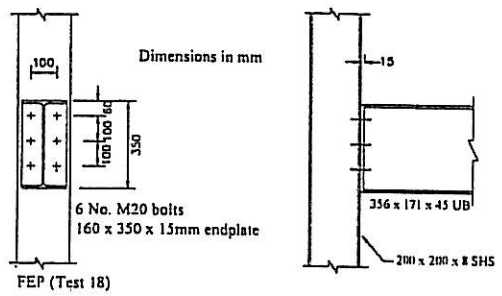

Figure 5 (a).
Flush end plate connections (FEP)

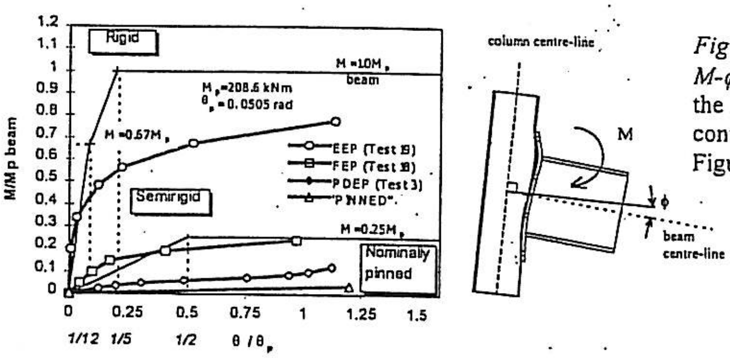

Figure 5(b).
M-ϕ characteristics of the FEP flowdrill connections shown in Figure 5(a).

The joints were subjected to realistic loadings as expected in real buildings. The test results showed that the flowdrill connections exhibited no unexpected failures and that they behave as semi-rigid. Thus, the connections are able to transfer moment between the beams and the columns but simultaneously permit some relative rotations between the connected members.

3 Analytical Studies on the Response of Tubular SHS Columns in Multi-storey Frames

3.1 Parametric Study

The results of experimental work carried out by France have provided an understanding of the behaviour of flowdrill connections which can be exploited further in analytical studies of global frames.

In order to investigate the behaviour and ultimate strength of SHS columns at collapse load level a parametric study using a non-linear semi-rigid program on a one-bay two-storey non-sway frame was carried out (see Figure 6). The storey height h was taken as 4m. The frame bay width b was taken as 6m for all cases. The beams and columns were taken as 356 x 171 x 45 UB and 200 x 200 x 8 SHS respectively. All members were designated to have grade 43A steel with the yield stress value of 275 N/mm^2. The steel was assumed to possess an elastic-perfectly plastic stress-strain relationship with the modulus of elasticity of 205 kN/mm^2. The frame utilised FEP connections as shown in Figure 5(a). The corresponding connection M-ϕ curve of the FEP connections is shown in Figure 5(b).

In phase one of the loading, a uniform distributed load of 30 kN/m was loaded to the right beam which was connected to the top end of the column under investigation. The purpose was to transfer the detrimental beam end moment to the top end of the column

3.2 Response of Beam-Columns

This section discusses the analytical studies on the possibility of adopting flowdrill connections in multi-storey frames. The discussion refers to Figures 6 to 8. With the use of FEP connections, the effective restraint offered by the column and the connection at the beam ends has increased. As a result of beam loading, the corresponding end restraint moment at beam ends is 20.5 kNm. The lower column is able to provide resistance of about -9.7 kNm and the balance is transferred to the upper column (see Figure 8).

Due to larger initial column deflections, an earlier formation of first yield is observed at 1535 kN (see Figure 7). In the elastic range, about 19% of moment shedding has occurred (see Figure 8). This is followed by 79% of moment shedding in the inelastic range. The column end moment at collapse load has been relaxed to -1.6 kNm which corresponds to a total of 98% of the moment shedding. The collapse load is observed at 1592 kN (see Figure 7). The reserve of strength is 3.7% above the first yield load and the corresponding deformed shape of the frame at ultimate load level is shown in Figure 6(b). It is seen that due to the relaxation of moment at column top end, the bending moments at collapse load of the tested column shows similarity to that of an axially loaded case. This response is in accordance with the phenomenon observed by other researchers [6], [7].

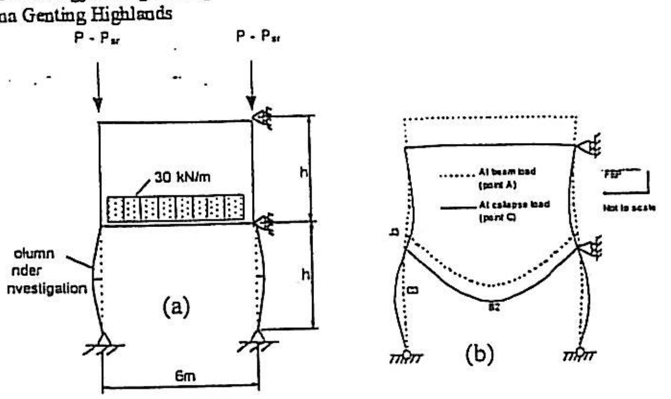

Figure 6 (a). Loading combinations (b). Exaggerated deformed shape of frame with FEP connections

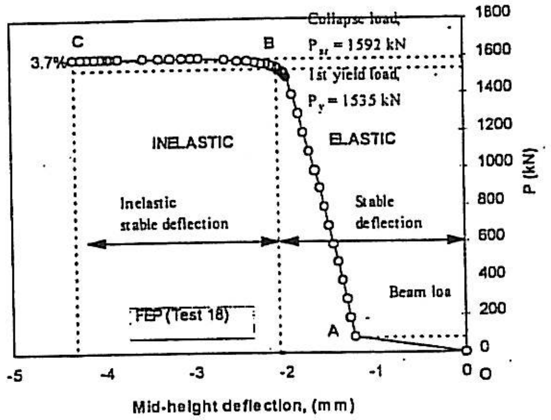

Figure 7 Load deflection response at column mid-height with FEP connections

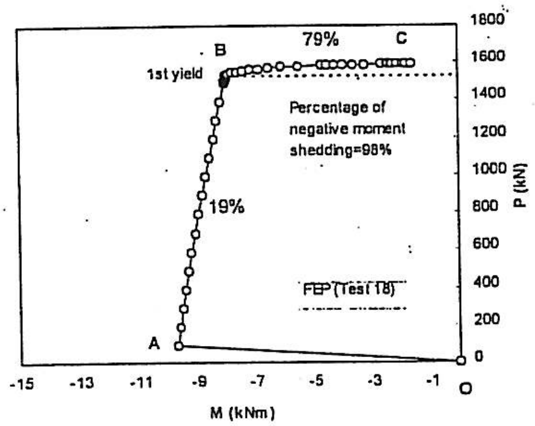

Figure 8 Response of moment shedding at column top end with FEP connections

4 Conclusion

Results of the study show that beyond yielding the configuration of moment in the frame has changed dramatically due to the phenomenon of moment shedding. The configuration of moments at ultimate load shows that a beam-column with end restraints can be treated as an axially loaded compression member.

In terms of stability and strength, preliminary studies on a two-storey and one-bay frame show that SHS columns coupled with flowdrill connections are suitable for low-rise multi-storey non-sway frames.

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DESIGN OF UNBRACED STEEL FRAMES BY WIND MOMENT METHOD FOR LOCALLY PRODUCED STEEL SECTIONS.

By

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For the design of unbraced sway frames at ultimate limit state, it is possible to use advanced methods based on interaction of elastic buckling characteristics and the reduction in stiffness due to plasticity. The joints are assumed to be rigid and full-strength. However, other less-sophisticated methods apparently based on purely elastic behaviour are also available. One approach, termed the "wind-moment" or "wind-connection" method, is often used in the U.K. The method is known as "Type 2 Construction" in the U.S.A. In its simplest form the "wind-moment" method assumes:

- under gravity load, the connections act as pins; this means that the beam members are designed as simply supported with no moments transferred to the column, other than nominal "eccentricity" moments;.
- under wind load, the connections behave as rigid joints, with points of contraflexure at the mid-height of columns and mid-length of beams.

Members are proportional initially to resist gravity load. The internal forces and moments due to gravity load and wind are then combined in appropriate load cases. The design at the ultimate limit state is completed by amending the initial section sizes and other details for the members and connections, to withstand combined load effects. The advantage of the method is its simplicity. The frame is considered as statically determinate with internal moments and forces not dependent on the relative stiffness of the members. The need to repeat the analysis to correspond to changed section sizes is thereby avoided. The beam sections generally have the same size for all the floors since the mid-span moment due to gravity load usually controls the design, thus simplifying the construction of the building.

This paper will elaborate on the analysis and design of the unbraced steel frame bending on major axes. Work example for 2 bay 4 storey height steel frame will be shown and the verification of the design will be carried out by computer software. The result proved that the method could be used for hand calculation and reliable as a design tools.

Note: For Symposium C

Malaysian Science and Technology Congress 2001 – Symposium B, 8-10 October 2001, Melaka

PHENOMENON OF MOMENT SHEDDING IN BEAM-COLUMNS OF MULTI-STOREY STEEL BUILDINGS

By

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Abstract

Semi-rigid connections and their potential benefits to multi-storey steel frame structures have been the focus of many investigations for the past 60 years. Numerous investigations into the global behaviour of steel frames as influenced by the presence of semi-rigid connections have been reported by many researchers. One of the important aspects of frame behaviour observed in recent studies is the effect of shedding moment. Moment shedding is a phenomenon in which the moment at column top end is progressively shed as the column starts to undergo failure. In multi-storey buildings, this kind of phenomenon has been seen in many tests either in small scale or full scale frames.

As a result of moment shedding, a simplified method of designing beam-columns with semi-rigid joints as axially loaded only may be proposed.

INTRODUCTION

The aim of the study presented in this paper is to investigate the behaviour of SHS columns with various types of connections in both elastic and inelastic ranges. This includes the investigation on the following aspects:

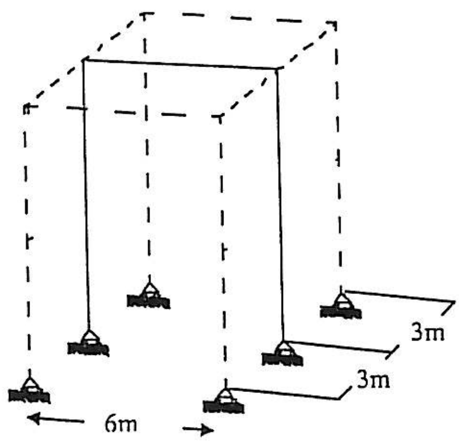
- the phenomenon of moment shedding and its effect to the column performance
- the applicability of the beam-columns to be designed as axially loaded compression members

PARAMETRIC STUDY ON BEAM-COLUMNS WITH SEMI-RIGID CONNECTIONS

In order to investigate the behaviour and ultimate strength of columns with semi-rigid end restraints at collapse load level, a single storey one bay frame was examined (see Figure 1). The frame was employed to investigate the strength of .beam-columns with semi-rigid connections. The storey height was taken as 3m whereas the frame bay width was taken as 6m for all cases. The beam size was taken as $533 \times 210 \times$UB 122 and the studied columns were taken as $203 \times 203 \times$UC 86.

All members were designated to have grade 43A steel with the yield stress value of 275 N/mm^2. The steel was assumed to possess an elastic-perfectly plastic stress-strain relationship and the modulus of elasticity was taken as 205 kN/mm^2.

The moment rotation relationship of beam-column connections for the frame was obtained from Buick Davison test [1]. Four types of semi-rigid connections namely RIGID, extended end plate (EEP), flush end plate (FEP) and flange cleat (FC) were utilised in this study. An initial imperfection of L/1000 was applied to the columns.



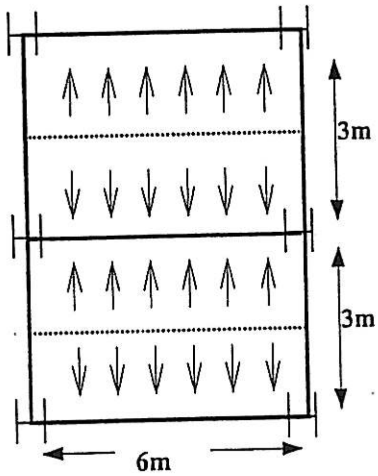


Figure 1 Frame dimensions and loadings

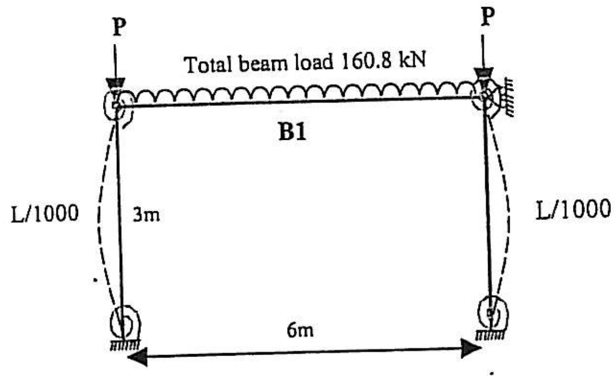


Figure 2 Initial imperfections and loadings

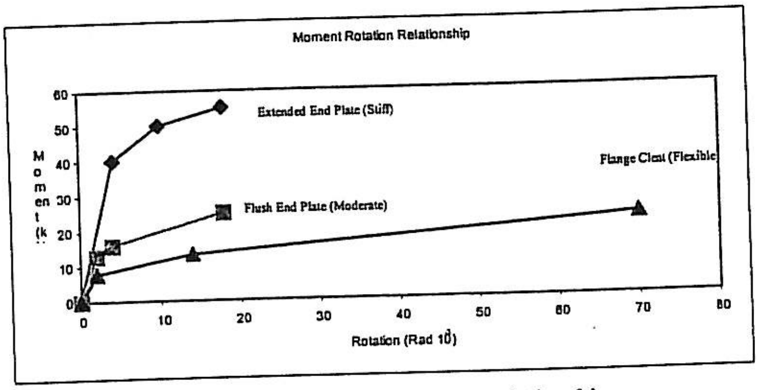


Figure 3 Connection moment-rotation relationship

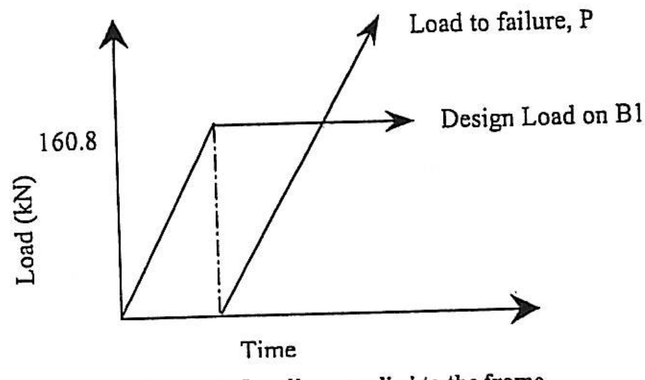


Figure 4 Loadings applied to the frame

In the first stage of the loadings, the uniform distributed load was applied to beam B1 to induce moment at the top end of columns (see Figures 2 and 4). The beam load was kept constant and then followed by the application of incremental axial column loads located at both column heads. Three different connection types with different degrees of rotational stiffness were chosen. The corresponding M-ϕ characteristics of the semi-rigid connections are shown in Figure 3.

RESULTS

Figure 5 shows the load-deflection characteristics of the studied column. It can be seen that the slopes of load-deflection curves in the elastic range are almost similar for all connection types. This implies that in the elastic range, the stiffness of the column is not much influenced by the connection stiffness.

Furthermore, as can be seen from the figure, the stiffening effect is very small and has little significance to the column. Hence, it indicates that increasing connection restraint has little influence on the performance of the column in the elastic range. It can be noticed that after the formation of first yields, the slopes are much steeper with stiffer connections as compared to the more flexible connections.

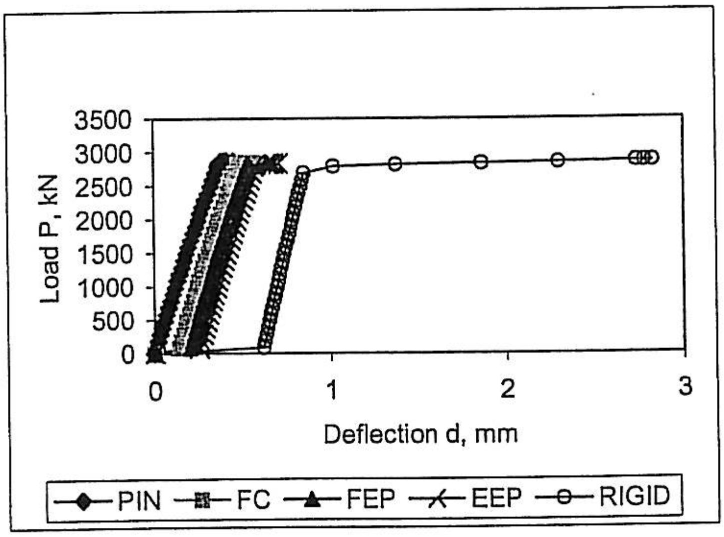


Figure 5 Load-deflection Response

Figure 6 shows the load-moment response of the column. As can be seen from the four load-moment figures, the plots of axial loads against end moments show that the initial detrimental moment is progressively shed with increasing axial load. Eventually, dramatic moment shedding is observed when the column start to undergo yielding.

After the detrimental negative moment is relaxed to zero, the moment will then act as a reverse moment (positive moment). This reversal of moment acts in the opposite direction to the column rotation. This in turn results in the column being restrained further at its ends.

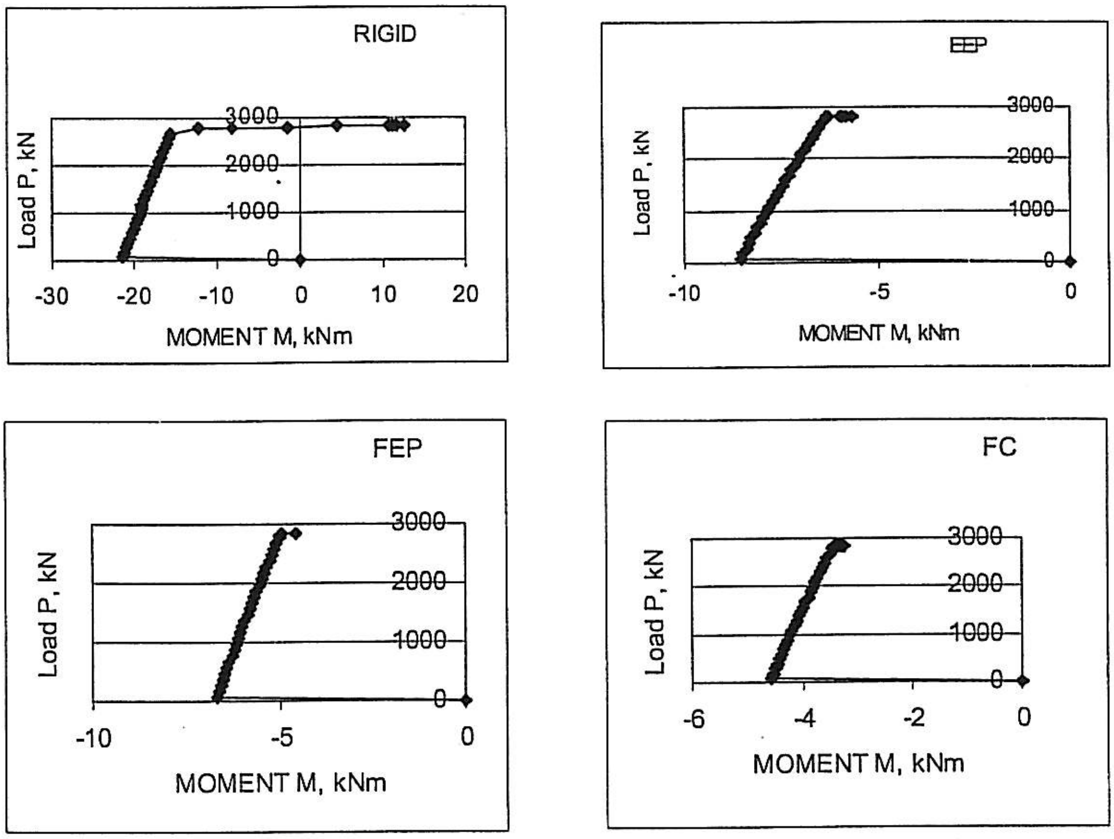


Figure 6 Load-moment Response

Based on the analysis results presented above, it is now recognised that beam-columns behave differently at both service and ultimate load levels. At the lower load level, the moment transferred by the beam causes a detrimental effect to the column. On the other hand, at the ultimate load level, the column top end moment instead of being detrimental has become beneficial by inducing a reversal of moment which restrained the column.

The problem of interaction between the axial load and moment which occurs at the lower load level diminishes as the ultimate load level is approached as a result of moment shedding phenomenon. As a consequence, the bending moment diagrams of beam-columns at collapse load are analogous to the bending moment diagrams of axially loaded columns with end restraints. This response demonstrates that beam-columns can be treated as axially loaded members.

CONCLUSIONS

The parametric study on the behaviour and strength of SHS beam-columns at ultimate load level has been presented. It is seen that the study on the behaviour of elastic-plastic columns at higher load level leads to a more realistic design method. Moreover, the problem of the detrimental moment acting at the column end which normally occurs at a lower level diminishes as the column loses its stiffness at higher load levels. As a result, the design procedure based on the response at the ultimate load level is nothing more involved than the simple analysis and design of individual members of beams and columns.

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Melaka
15/5/02

BEHAVIOUR OF BEAM-COLUMNS WITH SEMI-RIGID CONNECTIONS AND ITS IMPLICATION TO THE DESIGN CONCEPT

by

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ABSTRACT

The parametric study on the behaviour of beam-columns with semi-rigid connections in the elastic and inelastic ranges for non-sway frames has been investigated. The study was carried out using a second order non-linear elastic-plastic finite element program incorporating the use semi-rigid elements. The study described in this paper, however, is limited to a one-bay and two-storey frame only. The results show that the behaviour of beam-columns beyond yielding is significantly different from the behaviour in the elastic range. This is due to the effect of moment shedding which significantly influenced the form of moment transfer from the initial beam loads up to the collapse condition. At the collapse load, the structure is seen to become analogous to a statically determinate system in which the columns and the beams can be treated individually. The results have shown that at collapse load, the column end moment reduced and usually can be relaxed to zero ($\pm$ 0). Therefore, the bending moment diagram of a beam-column at ultimate load is similar to that the bending moment diagram of an axially loaded compression member. On the basis that at collapse load the structure is still in a state of stable equilibrium, the configuration of the bending moments can be adopted as a fundamental feature for a true simplified design method. Based on this behaviour, a simplified design method of designing beam-columns in low-rise multi-storey buildings non-sway frames will be discussed.

1 Introduction

Semi-rigid connections and their potential benefits to multi-storey steel frame structures have been the focus of many investigations for the past 60 years. Numerous investigations into the global behaviour of steel frames as influenced by the presence of semi-rigid connections have been reported by many researchers.

This paper presents a brief overview of the research that has contributed to the understanding the behaviour of semi-rigid connections, columns and frames and its implication to the design concept. The analyses reported in this paper have been carried out using a second order non-linear finite element program incorporating the semi-rigid element. The program known as SERIFA can handle the elastic and inelastic behaviour of semi-rigid frames.

2 Analytical Studies on the Response of Beam-Columns in Multi-storey Steel Frames

2.1 Parametric Study

In order to investigate the behaviour and ultimate strength of square hollow section (SHS) beam-columns at collapse load level, a parametric study using a non-linear semi-rigid program on a one-bay two-storey non-sway frame was carried out (see Figure 1). The storey height *h* was taken as 4m. The frame bay width *b* was taken as 6m for all cases. The beams and columns were taken as 356 x 171 x 45 UB and 200 x 200 x 8 SHS respectively. All members were designated to have grade 43A steel with the yield stress value of 275 N/mm^2. The steel was assumed to possess an elastic-perfectly plastic stress-strain relationship with the modulus of elasticity of 205 kN/mm^2. The frame utilised FEP connections as shown in Figure 2(a). The corresponding connection *M-ϕ* curve of the FEP connections is shown in Figure 2(b).

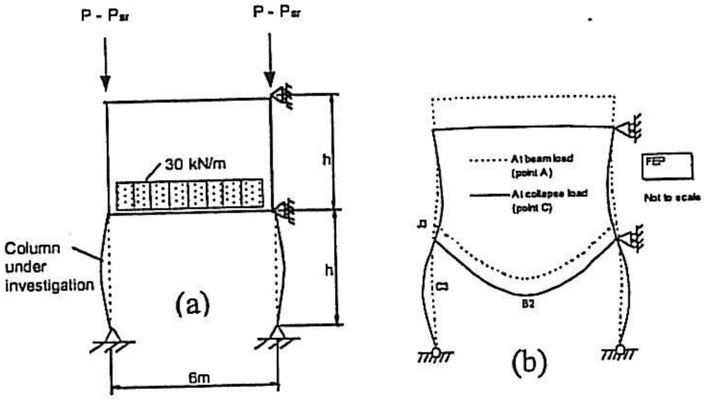


Figure 1 (a). Loading combinations (b). Exaggerated deformed shape of frame with FEP connections

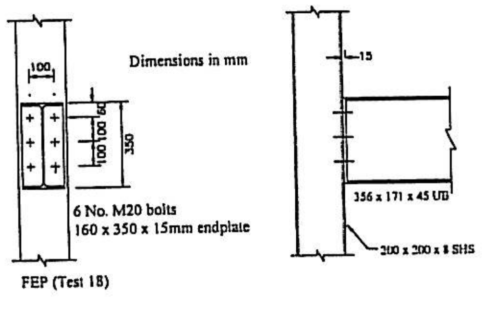


Figure 2 (a).
Flush end plate connections (FEP)

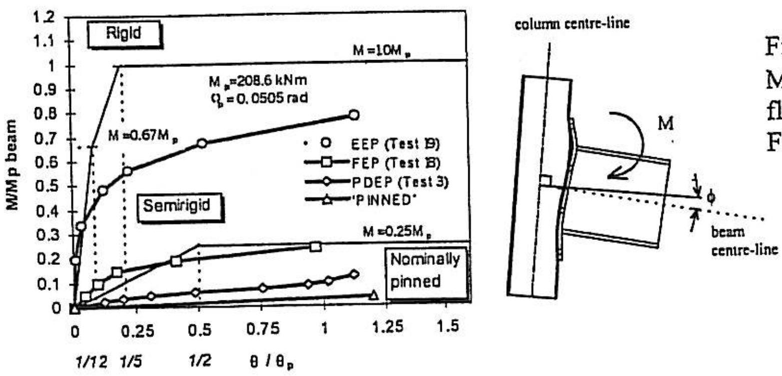


Figure 2(b).
M-ϕ characteristics of the FEP flowdrill connections shown in Figure 1(a).

In phase one of the loading, a uniform distributed load of 30 kN/m was loaded to the right beam which was connected to the top end of the column under investigation. The purpose was to transfer the detrimental beam end moment to the top end of the column

2.2 Response of Beam-Columns

This section discusses the results of parametric studies on the behaviour of beam-columns in multi-storey frames. The discussion refers to Figures 3 to 4. With the use of FEP connections, the effective restraint offered by the column and the connection at the beam ends has increased. As a result of beam loading, the corresponding end restraint moment at beam ends is 20.5 kNm. The lower column is able to provide resistance of about -9.7 kNm and the balance is transferred to the upper column (see Figure 4).

Due to larger initial column deflections, an earlier formation of first yield is observed at 1535 kN (see Figure 3). In the elastic range, about 19% of moment shedding has occurred (see Figure 4). This is followed by 79% of moment shedding in the inelastic range. The column end moment at collapse load has been relaxed to -1.6 kNm which corresponds to a total of 98% of the moment shedding. The collapse load is observed at 1592 kN (see Figure 3). The reserve of strength is 3.7% above the first yield load and the corresponding deformed shape of the frame at ultimate load level is shown in Figure 1(b). It is seen that due to the relaxation of moment at column top end, the bending moments at collapse load of the tested column shows similarity to that of an axially loaded case. This response is in accordance with the phenomenon observed by other researchers [1], [2].

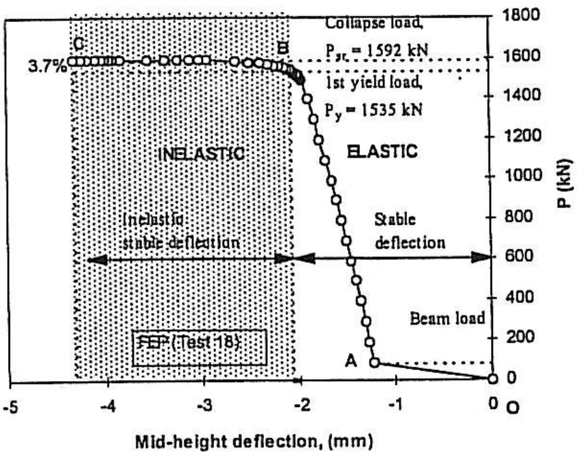


Figure 3 Load deflection response at column mid-height with FEP connections

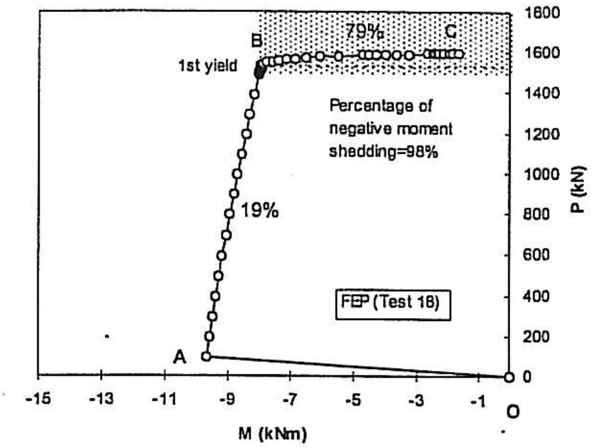


Figure 4 Response of moment shedding at column top end with FEP connections

3 The Simplified Design Method (The α_{Pin} Method)

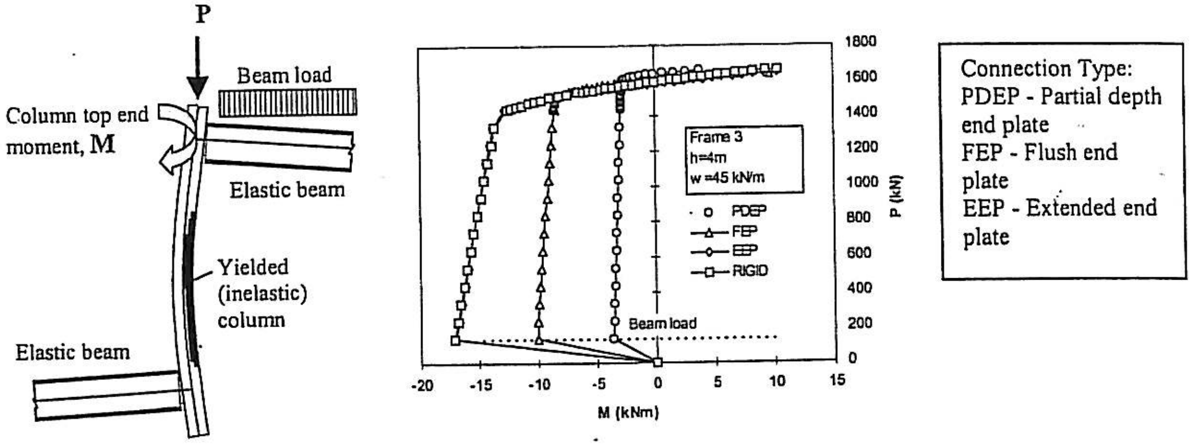
Based on the behaviour of beam-columns at ultimate load level, a simplified design method called the α_{pin} has been developed. The α_{pin} is defined as

$$\alpha_{pin} = P_{sr} / P_{pin} \qquad (1)$$

in which P_{sr} is the ultimate axial load (collapse load) of semi-rigidly connected column, P_{pin} is the ultimate axial load of equivalent perfectly pin ended column. The main criteria of this design method is that the effect of acting moments are taken as being outweighed by the column end restraint and hence the column can be designed as axially loaded. Recently, based on this study, the concept of α_{pin} method has been adopted in the scheme design of semi-continuous braced frames published by the Steel Construction Institute, UK [3].

3.1 Basis of the α_{pin} Method

The basis of the proposed α_{pin} method is the phenomenon of moment shedding and the response of inelastic columns at the ultimate limit state with the beams remaining elastic (see Figure 5(a)). The response of beam-columns shown in Figure 5(b) was investigated using a second order non-linear finite element program incorporating the semi-rigid elements [4].



(a). (b).

Figure 5 Response of beam-columns at ultimate load level

The results of the study carried out using the second order finite element program show that when the column starts to become inelastic, the column end moment is dramatically shed and relaxed to zero (see Figure 5(b)). This is followed by the redistribution of moments from the yielded column to the neighbouring members. This in turn resulted in a simple configuration of internal forces in the frame members. Eventually, the frame members can be rendered as statically determinate in which the columns can be treated as axially loaded and the beams as simply supported with a certain value of end restraint moment. The results of the analytical studies show that the eccentricity moments due to beam reactions acting at the column face are also subjected to moment shedding and eventually relaxed to almost zero. The phenomenon of moment shedding has been observed in the experimental results of both semi-rigid and rigid frames [1].

As a result of the above phenomenon, the α_{pin} design method neglects the moments resulted from the eccentric load and that transmitted by the connections.

The idealisation of the frame for the α_{pin} design is discussed as follows:

- Figure 6(a) shows a semi-rigid non-sway frame carrying gravity loads. The corresponding internal moments in the frame due to working load are shown in Figure 6(b).
- In order to obtain the response of the inelastic column with the beam remaining elastic, a series of incremental loads is applied at the column heads of the top storey columns to cause failure of the lower columns. When the lower columns fail as the ultimate load is reached, the corresponding internal moments of the frame that can be rendered as statically determinate are shown in Figure 6(c).
- At ultimate load, the column is not subjected to the acting moment as the moment is already shed to the adjoining members. As a result, the internal moment of the column is similar to the moment of axially loaded columns. Consequently, the internal moment of the beam which is affected by the failed column is in the form of the bending moment of a simply supported beam with end restraints.

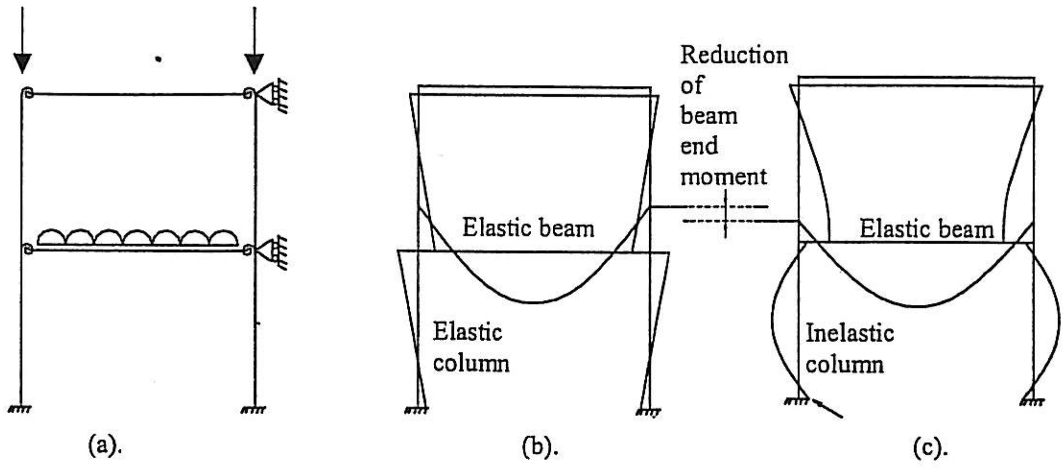


Figure 6 Idealisation of non-sway semi-rigid frame for the simplified design method

4 Conclusion

Results of the study show that beyond yielding the configuration of moment in the frame has changed dramatically due to the phenomenon of moment shedding. The configuration of moments at ultimate load shows that a beam-column with end restraints can be treated as an axially loaded compression member.

5 References

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