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INFLUENCE OF PRETWISTING ANGLE ON THE BUCKLING CAPACITY OF STEEL COLUMNS: A REVIEW

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ABSTRACT

A column is a vertical compression member designed to transmit compressive loading. It is generally seen that when a slender member is loaded in compression, it will bow sideways or buckle, and if the load is then increased further the column will eventually fail in bending. Buckling is a mode of failure that is mainly observed in compression members due to structural instability. A pretwisted column has its strong flexural plane weakened and its weak flexural plane strengthened, leading to a favourable effect on buckling strength of the pretwisted column. A linear buckling analysis study was conducted for boxed and unboxed sections for columns with varying twist angles to study the effect of twist angle variation on improvement in buckling capacity. The studies reviewed that buckling capacity increased upto an optimum twist angle value and further reduced. It was found that pretwisting is effective to increase the buckling capacity of columns.

KEYWORDS: Pretwisting, Slenderness Ratio, Buckling, Local Failure

INTRODUCTION

A column is essentially a vertical member designed to transmit a compressive load. Being a compression member, it is reasonable to suppose that a column would fail by crushing of the material when the load reached a high enough value, but for most columns failure occurs at a lower load than the crushing strength; this is because most columns are relatively slender, i.e. they are long in relation to their lateral dimensions. It is generally seen that when a slender member is loaded in compression, it will bow sideways or buckle, and if the load is then increased further the member will eventually fail in bending.

The ratio of the effective length of a column to the least radius of gyration of its cross section is called the slenderness ratio (expressed with the Greek letter lambda, λ). This ratio affords a means of classifying columns. Slenderness ratio is important for design considerations. A short steel column is one whose slenderness ratio does not exceed 50; an intermediate length steel column has a slenderness ratio ranging from about 50 to 200, and are dominated by the strength limit of the material, while a long steel column may be assumed to have a slenderness ratio greater than 200 and its behaviour is dominated by the modulus of elasticity of the material.

If, on the other hand, a stocky column is used, one with a low length to breadth ratio, then a crushing mode of failure is more likely than a buckling mode. Thus the normal compression elements, length and lateral dimension play a part in determining the mode of failure that will result. Also, for a given section, there will be a critical length of the compression member below which it will be crushed and above which it will buckle.

The shape of a column is also very important. If a long thin flexible rod is loaded longitudinally in compression, it is noticeable that it deflects readily near the midpoint of its length with a considerable amount of displacement. The phenomenon is called buckling and occurs when the stresses in the rod are still well below those required to cause a shearing type failure.

Columns and struts may therefore be described as either short or slender depending on its mode of failure. A short column or strut will fail internally by yielding in the case of ductile materials, such as mild steel, or by shearing in the case of brittle materials such as concrete.

Slender columns are becoming increasingly important and popular because of the following reasons:

- The development of high strength materials (concrete and steel),
- Improved methods of dimensioning and designing with rational and reliable design procedures
- Innovative structural concepts – specially, the architect's expectations for creative structures.

BUCKLING

Buckling is characterized by a sudden sideways failure of a structural member subjected to high compressive stress, where the compressive stress at the point of failure is less than the ultimate compressive stress that the material is capable of withstanding. Mathematical analysis of buckling often makes use of an "artificial" axial load eccentricity that introduces a secondary bending moment that is not a part of the primary applied forces being studied. As an applied load is increased on a member, such as a column, it will ultimately become large enough to cause the member to become unstable and is said to have buckled. Further load will cause significant and somewhat unpredictable deformations, possibly leading to complete loss of the member's load-carrying capacity. If the deformations that follow buckling are not catastrophic the member will continue to carry the load that caused it to buckle. If the buckled member is part of a larger assemblage of components such as a building, any load applied to the structure beyond that which caused the member to buckle will be redistributed within the structure. The strength of a column may therefore be increased by distributing the material so as to increase the moment of inertia. This can be done without increasing the weight of the column by distributing the material as far from the principal axis of the cross section as possible, while keeping the material thick enough to prevent local buckling.

A slender column or strut will fail by buckling, where a relatively large bending distortion will develop along its length. The member does not collapse immediately but remains in bent equilibrium unless the yield strength of the material has been exceeded. The buckling phenomenon is an example of unstable equilibrium, whereas the behaviour of a short strut is that of stable equilibrium.

The axial load to cause buckling is called the critical load (P). For a given load, a critical length may also be deduced. In the case of slender structural columns or struts, the critical buckling load and the critical length depend upon a number of factors, such as the shape and size of the cross-section, the relationship between the length of the column and its lateral dimensions and the degree of fixity at both ends.

For a strut of given length which is pinned at both ends, the minimum load at which buckling will occur may be determined using a mathematical analysis which produces what is known as the Euler Formula. The Swiss mathematician, Leonhard Euler (1707 – 83), calculated the load at which a column would buckle if it were axially loaded and pinned at its ends.

Thus, the Euler buckling load for an axially loaded pin ended column is given by:

$$P_E = \frac{\pi^2 EI}{L^2}$$

P_E = the Euler buckling load

E = Young's modulus for the material

I = the least second moment of area of the section

L = the length of the strut between the pinned ends

The magnitude of the buckling load given by this formula is the appropriate value for initially straight struts which are pinned at both ends and are subject to an axial load only. In the cases where one end is fixed and the other end is pinned, or where both ends are fixed, the effective length has to be determined by multiplying the length between supports by an effective length factor.

1. Both ends pinned — Effective length = actual length x 1.0
2. Both ends fixed — Effective length = actual length x 0.5
3. One end pinned other end fixed — Effective length = actual length x 0.7
4. One end fixed,
   other end completely free — Effective length = actual length x 2.0

Sections normally used in steel structures are I-sections, Channels or angles etc. which are called open sections, or rectangular or circular tubes which are called closed sections. These sections can be regarded as a combination of individual plate elements connected together to form the required shape. The strength of compression members made of such sections depends on their slenderness ratio. Higher strengths can be obtained by reducing the slenderness ratio *i.e.* by increasing the moment of inertia of the cross-section. Similarly, the strengths of beams can be increased, by increasing the moment of inertia of the cross-section. For a given cross-sectional area, higher moment of inertia can be obtained by making the sections thin-walled. Therefore, the buckling of the plate elements of the cross section under compression/shear may take place before the overall column buckling or overall beam failure by lateral buckling or yielding. This phenomenon is called local buckling. Thus, local buckling imposes a limit to the extent to which sections can be made thin-walled.

In closed sections such as the hollow rectangular section, both flanges and webs behave as internal elements and the local buckling of the flanges and webs depends on their respective width-thickness ratios. In this case also, local buckling occurs along the entire length of the member and the member develops a 'chequer board' wave pattern as shown in Figure 1.

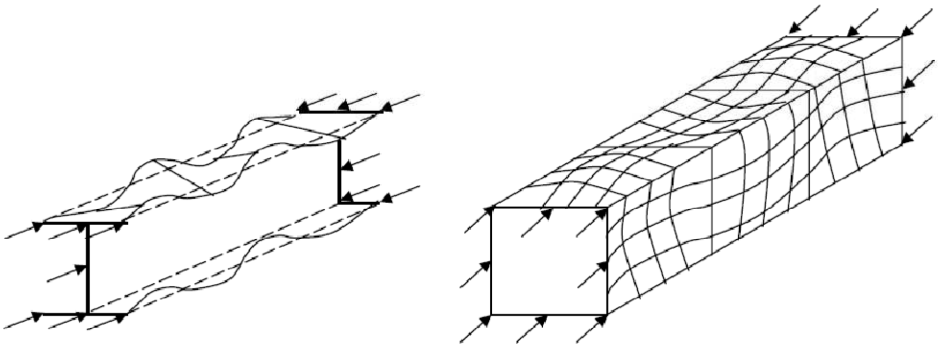

Figure 1: Local Buckling of Compression Members

Local buckling has the effect of reducing the load carrying capacity of columns and beams due to the reduction in stiffness and strength of the locally buckled plate elements. Therefore it is desirable to avoid local buckling before yielding of the member. Most of the hot rolled steel sections have enough wall thickness to eliminate local buckling before yielding. However, fabricated sections and thin-walled cold-formed steel members usually experience local buckling of plate elements before the yield stress is reached. Local buckling involves distortion of the cross-section. There is no shift in the position of the cross-section as a whole as in global or overall buckling. [13]

Beam Columns

Beam columns are member that are subjected to both axial compression and bending while bending is as important as axial compression. The bending may be caused either by moments applied to the ends of the member or it may be due to transverse load. The lateral loads or end moments cause deflections which are further amplified by axial compression causing moments, along the member. These additional deflections add significantly to the moments, which may result still further deflection.

Difference between eccentrically loaded column and beam-column

- Eccentrically loaded column: axial pressure is the primary effect while bending (unavoidable imperfections) is the secondary; the research is mainly to discuss the effect of bending on axially loaded column.
- Beam-column: axial pressure is the primary effect while bending (intentionally applied) is also important, the research is mainly to discuss the effect of axial load on bending.

Figure 2 shows a beam-column undergoing lateral deflection as a result of the combination of compression and equal and opposite moments applied at the ends.

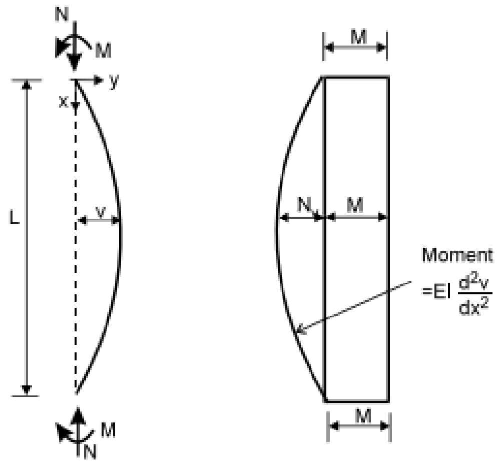

Figure 2: Primary Moment M and Secondary Moment N_V

Pretwisted Steel Columns

Pretwisting is a method of applying an angle of twist along the member's length, such that the principal axes of inertia rotates in accordance with the centroidal axis of the column. The method of applying the pretwisting along the centroidal axis at various twist angles is a challenging task in the real practice. Hence, pretwisting would be considered as an introduced twist to the column which would bear a higher critical load before reaching its ultimate strength. Implementation of pretwisting in a certain member leads to inducing a coupling effect on the weak and strong planes. Moreover, the effect of pretwisting can be explained as a transition between the weak and strong axes of the member (i.e. the weak axis may be strengthened, while the strong axis may become weaker after being pretwisted).

A natural pretwist applied along the centroidal axis also induces a coupling effect in the flexural planes of a pretwisted member. This coupling of flexural planes helps to increase the first buckling load of the member subjected to axial load and thereby reducing the buckling load of second mode of buckling. Since the second mode of buckling does not have much significance, the effect of pretwisting on the increase in axial load capacity of pretwisted columns is beneficial.

A prismatic compression member may buckle due to flexure, torsion or due to both. It is predictable to determine that a prismatic member buckles along the plane of least resistance. In the case of pretwisted members, it is not predictable to evaluate the plane along which the member may buckle due to the coupling of strong and weak axis in every point along the length of the member. However, the column resistance varies at each point along its centroidal axis when its section is permanently pretwisted.

The axial strength and the static performance of the column may be influenced by this pre buckling twist which, in turn, may vary in any arbitrary manner along the length of the member. During buckling, the deformed configuration of the pretwisted column is no longer perpendicular to the axis of least resistance, which results in highly complex nonlinear differential equations, describing the equilibrium of the member. Results obtained from the various studies generally show very wide variation. However, all the studied models agreed that with increasing the angle of twist, the buckling strength of the column increases. Therefore, pretwisting can be considered as a simple way of strengthening thin columns subjected to axial loads or making thinner (lighter and more economical) columns with the same strength.

LITERATURE SURVEY

The concept of pretwisting was introduced in the literature a long time proposed by Zieglar in the year 1948. Previously this technique was massively used in helicopter rotor blades, turbine blades and gear teeth.

Serra (1993) analysed a pretwisted column manually and found out relation between buckling load and angle of twist. This analysis shows that with a simple permanent twisting at the ends a column with constant section can improve its static performance to buckling significantly. Therefore pretwisting can be considered as a simple way of strengthening compressed thin columns or making thinner columns with the same strength. [1]

Bairagi and Kanvinde (1993) said that the concept of pretwisting is similar to the concept of prestressing of flexural members. In the pretwisting technique, a predetermined twist within the elastic range is given to a steel bar placed along the longitudinal axis of the beam. The bar is then anchored to prevent any loss of pretwist. The concrete is then placed and allowed to harden. The bars are removed from the anchor after a proper curing period and the pretwist gets transferred to the beam. This pretwist will induce shearing stresses all over the cross-section of the beam inducing an internal torque. Thus it can be seen that this technique is similar to the prestressing technique. The pretwist is transferred to the adjoining concrete mass through specially designed vane or stud systems attached to the centrally twisted bar at specific spacings. The results say that the ultimate torque-carrying capacity for the pretwisted beams can be made to reach almost the same as that for beams with standard reinforcement. The presence of the steel bar, of course, contributes to the stiffness of the section when compared with those of the plain concrete ones. [2]

Madhusudhana et al. (1995) investigated the buckling capacity of uniformly and non-uniformly pretwisted beams with fixed-end conditions. The study found that the optimum twist was 225°. Also, the unidirectional pretwist applied along the member's length was revealed to yield greater buckling capacities than when various pretwist combinations were implemented in opposite directions throughout the centroidal axis from one end of the member to the other. Furthermore, the analysis showed that with uniform pretwisting, the principal axis which governs buckling is the weaker axis (i.e. *y* axis). The study also linked the increase in buckling load of a beam to the position of the pretwist from the centroid of the section to one end, concluding that the maximum buckling load is encountered when the twist is exactly at the center of the beam with a minimum buckling load at the beam end for both opposite and unidirectional twists. Finally, it was also concluded that when a unidirectional twist (symmetric about the center of a discontinuous beam) is applied in two portions, a greater buckling load is achieved.[3]

Celep (1985) studied the stability of simply-supported pretwisted columns subjected to static and periodic axial load. Pretwisting was defined as the rotation of the principal axes of the column around its undeformed axis. The effect of the rigidity ratio (i.e. ratio of the two principal moments of inertia of the section) was highlighted along with the effect of pretwisting on the static performance and the dynamic stability of the column. The analytical model was solved by using the Galerkin's method. The study revealed that as the rigidity ratio of a certain cross-section approaches unity, the effect of pretwisting almost vanishes. For the purpose of the study done by Celep, the first five modes of buckling were considered. The analysis also showed that the first buckling load is not much affected by the rigidity ratio, while the loads from the second and third buckling modes do vary slightly with a change in the rigidity ratio. Moreover, it was shown that as the rigidity ratio is increased, the buckling loads approach each other. It was also revealed that the load from the second buckling mode reaches a minimum before the first buckling load is reached. Furthermore, the study showed that medium pretwisting had the greatest effect on the lowest critical loads. [4]

Tabarrok et al. (1989) performed an analytical study on the buckling capacity of pretwisted columns implementing the principle of total potential energy to derive the desired equilibrium equations and the corresponding boundary conditions. The analysis involved both statically determinate and indeterminate cases. A significant increase in the buckling capacity of the column for the first mode and a faster decrease in the buckling strength of the second mode were observed for almost all boundary conditions. Furthermore, the first and the second mode shape converged as the angle of twist increased. Moreover, the study showed that statically indeterminate cases exhibited more oscillatory nature than the statically determinate cases in the presented graphs of strength ratios of pretwisted and prismatic columns versus the applied pretwist. [5]

Steinman et al. (1991) studied the effect of naturally applied pretwist on the buckling capacity of slender columns both statically determinate and indeterminate cases. The main parameters considered were the applied angle of pretwist and the ratio of second moments of area around the strong and weak axes of the slender columns considered. Four boundary conditions were considered in this study, involving hinged-hinged, clamped-hinged, clamped-clamped and clamped-free. The assumption behind this research was that a column in 3D-space buckles around the stronger flexural plane albeit its original plane of flexure being the weak plane and that pretwisting works on coupling these two flexural planes. It was found that the buckling mode of a pretwisted column resembles that of a prismatic column. The study revealed that for statically indeterminate columns, the optimum buckling capacity is reached for a range of pretwist angles between 90° and 270° followed by a decrease for the angles 270°-360°. The results also showed that the capacity of a pretwisted statically indeterminate column may be twice as much as that of the corresponding prismatic column. [6]

Frisch-Fay (1973) studies the stability of pretwisted columns under an external compressive load to develop differential equations that govern the buckled shape of the column. In his investigation the boundary conditions assumed for the pretwisted columns were spherical hinges provided at both ends. He investigated various parameters in his analysis such as buckling load corresponding to prismatic column (α) pretwist angle per unit length of column (*w*) and ratio of two moments of inertia (I_2/I_1). His findings said that the maximum increase in buckling capacity obtained by the pretwisted column was more than twice compared with the prismatic column provided $k_2<k_1$, where $k_2 = P/EI_1$and $k_2 = P/EI_2$. He also found that for $k_1 = k_2$, the applied pretwisting had no contribution towards the buckling capacity. [7]

Recently, Barakat and Abed (2010) conducted an experimental study to investigate the effect of pretwisting on the axial load capacity and stability of fixed-ended pretwisted steel bars with rectangular cross-sections. More than 200 specimens of different cross-sections, lengths, and widths were first twisted with several angles by applying pure torque using a torsion machine, then exposed to axial compression using an MTS machine. It was observed that the pretwisted bars claimed a non-planar deformed shape during buckling. Moreover, at buckling, the axial stiffness of the twisted bars decreased gradually until the critical load was reached. The experimental results revealed that the critical buckling load of a pretwisted bar was always of higher value than that of its corresponding prismatic bar. Furthermore, this experimental study showed that the effect of pretwisting was greater on sections with higher second moments of area for specific pretwist angles. [8]

Abed et al. (2013) then expanded the experimental study by using a nonlinear finite element analysis to include a wider range of pretwisting angles up to 270°. Both the experimental and numerical results concluded that pretwisting increases the buckling capacity of thin columns; the buckling load capacity becomes higher with higher ratios of principle moments of inertia for a specific set of pretwisting angles. It was also observed that the buckling load of a pretwisted bar is always higher than that of the corresponding prismatic bars with unequal principal moments of inertia. Also, the highest increase in buckling capacity of the bars was observed at a pretwisting angle close to 90°. [9]

Abed et al. (2015) evaluated the improvement in elastic buckling capacity of pretwisted columns using linear perturbation analysis. Three different column sections of various lengths initially twisted at angles from 0˚ - 180˚ were analysed assuming fixed-fixed and pinned-pinned end conditions. Results concluded that there is a significant improvement in the critical buckling capacity for different slenderness ratios. It was also observed that effect of various column lengths on improvement of buckling capacity was insignificant. As compared to fixed ended boundary conditions pinned ended column didn’t show considerable increase in the buckling capacity. [10]

Chan et al. (1991) proposed a non-linear finite element procedure for the pre- and post-buckling analysis of thin-walled box section beam columns. The incremental secant stiffness approach was employed in conjunction with tangent stiffness to trace the load vs deflection path of square hollow sections. The box beam column is idealised as an assembly of plates that are further decomposed into a number of element areas. The influence of local plate buckling upon the overall ultimate buckling behaviour of the member is incorporated in the analysis by adopting a set of modified-stress-versus-strain curves for axially loaded plates. Factors such as residual stresses, associated with hot rolled and cold-formed sections, and initial geometrical imperfections are accounted for in the analysis. [11]

Chiew et al. (1987) conducted an experimental investigation of the ultimate load behaviour of thin-walled box columns subjected to concentric or eccentric compressive loading taking into consideration of failure by local, overall and interaction buckling. The column models are fabricated from mild steel sheets hydraulically shear cut into require dimensions and positioned by tack-welding. The results show that for short columns, the failure was caused by the local buckling of the component plates. The behaviour of long columns with low plate width-thickness ratios was, however, dominated by the overall buckling, while those with high b/t ratios failed due to the combined effect of local and overall buckling. [12]

SUMMARY OF LITERATURE REVIEW

- Buckling capacity of pretwisted column show a significant increase with increase in twist angles by inducing a coupling effect on strong and weak planes.
- The unidirectional pretwist applied along the member's length was revealed to yield greater buckling capacities than when various pretwist combinations were implemented in opposite directions throughout the centroidal axis from one end of the member to the other.
- The unique configuration of the pretwisted member induces higher stiffness and better stability than the prismatic member. However the buckling mode shapes show a similar character under an axial compressive load.
- A significant increase in the buckling capacity for the column was found for first buckling and a faster decrease in buckling strength for second mode for almost all boundary conditions. It was also observed that the first and second mode shape converged as the angle of twist increased.

CONCLUSIONS

- The support conditions fixed-fixed or pinned-pinned does not show any effect in buckling load capacity for the case of pretwisted columns.

- For the I and H sections flanges are unstiffened members and webs are stiffened, as a result flanges yields first prior to web by completely losing its stiffness. Thereby there is a lower stiffness for the whole section since its flanges individually has become structurally ineffective. This type of local buckling leads to unstable structure prior to its critical load of failure.
- Various experimental and analytical studies were conducted on I sections to determine the increase in buckling capacity of steel columns. More studies need to be conducted for boxed sections to determine its response towards the axial buckling capacity for increasing twist angles.

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