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PROPOSED SIMPLE EQUATIONS FOR THE DESIGN OF STEEL I-BEAMS

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ABSTRACT

Design of laterally unsupported steel I-section beams according to ASD (Allowable Stress Design) and LRFD (Load and Resistance Factor Design) techniques requires the use of multiple equations. These equations depend on the section compactness, the laterally unsupported length of the beam, the geometric properties of the cross section and the yield strength of the steel. Furthermore, local buckling and lateral torsion-flexure buckling significantly affect the behaviour of steel I-section beams. The laterally unsupported length of the beam on the other hand, affects the critical moment initiating lateral flexure-torsion buckling. According to most codes of practice, three distinct zones are established for the behaviour of laterally unsupported steel beams; the moment resistance or the allowable bending stress. These two behaviours are defined by different equations in each zone in all the codes, which takes lots of effort and time. In this thesis, a single equation which defines the allowable bending stress for laterally unsupported steel I-section beams is proposed to cover all these zones. Another single equation which defines the moment resistance for laterally unsupported steel I-section beams is suggested to cover all these zones. The proposed equations results are compared to those obtained using the design provisions of the ECPSCB-ASD (2001) and the ECP-LRFD (2008). The equation is proposed to replace the discontinuous definitions currently adopted by the two previously mentioned codes defining the allowable bending stress and the nominal moment resistance for laterally unsupported steel I-section beams. Results obtained using the proposed equations are also compared to those obtained using CAN/CSA-S16 (2007) and AISC-LRFD (2005). The proposed model has been also verified against experimental results available from literature. The results of the proposed equations are well-matched with those provided by the design codes of practices as well as the finite element model results.

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CHAPTER 1

INTRODUCTION

The design of laterally unsupported steel I-beams according to ASD (Allowable Stress Design) and LRFD (Load and Resistance Factor Design) techniques requires the use of multiple equations which depend on many geometric and material parameters. Most codes of practice define three distinct zones with the moment resistance characterized by a different design equations for each zone. The first and second zones are affected by elastic and elasto-plastic lateral-torsional buckling, respectively. The third zone is only governed by steel yielding.

Lateral-torsional buckling is an important phenomenon in the design of steel I-beams. Most beams used in steel construction have a greater stiffness about the axis which resists the bending moment than that in the perpendicular direction. This results-in the possibility of lateral-torsional buckling which arises from lateral deflection and twisting. The resistance to this type of buckling depends on the lateral bending and torsional stiffness of the cross section, bracing type and hence the laterally unsupported length, arrangement and stiffness, the bending moment distribution along the length of the beam, the position of load with respect to the cross section, the material properties, the magnitude and distribution of residual stresses, the initial twist of the cross section, and finally the initial bow along the length of the cross section. Lateral-torsional buckling is a limit state that may often control the design of steel girders. Design specifications usually provide buckling solution derived for uniform moment loading and account for moments gradient along the unbraced length with a moment gradient factor C_b applied to these solutions.

On the other hand, local buckling is an important mode of failure for the I-section steel members. Local buckling is mainly affected by the flange outstand-to-thickness and the web-to-thickness ratios. These two ratios define the "compactness" of the steel I-sections. Codes of practice place limits on these ratios such that the critical stress initiating local buckling would not be reached before the yield stress is reached at selected locations.

1.1 Aim of the Research

The objective of this research is to propose a simple design equation for steel I-beams. The proposed equation shall cover all the three distinct regions of the beam's behaviour: fully plastic, elasto-plastic lateral buckling and elastic lateral buckling. The proposed equation is verified against results of an extensive parametric study performed using a numerical model which is based on the finite element technique.

The research started by collecting a data-base encompassing the results of the experimental investigations performed on beams, and related to the topic of this research. The results of this collected data-base are then used to verify a proposed nonlinear finite element model. The model can predict the behaviour and failure of steel I-beams including plasticity, local buckling, lateral buckling, etc. The numerical model, thus, accounts for: material and geometric nonlinearities, instability and post-buckling behaviour. The model is used to perform a wide-range parametric study covering most of the related parameters affecting the behaviour and failure of steel I-section beams, I-section cantilevers, mono-symmetric I-section beams and I-section columns. The results of the proposed equation are verified against the finite element model results and codes.

The outcome of this research is presented in a from of two simple equations for steel I-section beams: one for the allowable stress design (ASD) concept and the second for the load and resistance factor design (LRFD) concept. Another single equation is introduced to extend the beam equation to include columns as well as the beam-column members for design with the LRFD concept.

1.2 Outlines of the Thesis

This thesis is divided into seven chapters. Chapter 1 introduces the general description of the current complexity related to beam design together with the main aim of the research and the summary of the thesis.

Chapter 2 presents a literature review of the previous work in the field of lateral-torsional buckling of steel I-section beams. The method of analysis and some of the previously performed experimental work and researches in the same field are also outlined.

Chapter 3 presents a verification analysis of the equation proposed for the design of steel I-section beams based on the LRFD technique versus AISC-LRFD (2005), CAN/CSA-S16 (2007) and ECP-LRFD (2008) specifications. Modification in the proposed equation factor (n) is presented to suit every case of the previously mentioned codes of practice. The proposed equation for design of steel beams based on the ASD technique is also verified versus ECPSCB-ASD (2001). All cases of the verification included two types of steel grade: 24/36 & 36/52 with various sizes of the cross section and wide range of the unsupported lengths. The verification was performed in this chapter for a simply supported beam.

In Chapter 4, a description of the finite element model is presented. The analytical technique used to obtain the solution of the problem is explained. Details of the finite element model are described. Verification analysis is performed for the numerical model using previous experimental work to check the validity of the finite element model and to investigate the accuracy of the adopted analytical technique. The developed model is then used to perform a wide range parametric study for steel I-section beams where results of the proposed equations are compared to results obtained numerically. The analysis at this stage is only performed for simply supported beams.

Chapter 5 introduces an extension of the work of Chapters 3 and 4 to cover beams with other configurations and boundary conditions. This include I-section cantilever subjected to an end concentrated load (C_b=1.67), and a distributed load (C_b=2.3). Another proposed configuration is for mono-symmetric I-section steel beams.

Chapter 6 presents a proposal to extend the beam equation to beam-columns. Finally, a summary of the thesis and its main conclusions are introduced in Chapter 7. Proposals for further research along the same line are also presented.

CHAPTER 2

LITERATURE REVIEW

Local buckling and lateral-torsional buckling significantly affect the behaviour of steel I-section beams subject to flexure. The design of laterally unsupported steel I-section beams, according to ASD and LRFD specifications requires the use of multiple equations which depend on the section compactness, the laterally unsupported length of the beam, the geometric properties of the cross section and the yield strength of the steel. Flange outstand-to-thickness and web height-to-thickness ratios define the I-section compactness. The laterally unsupported length of the beam affects the critical moment initiating lateral-torsional buckling. Most codes define three distinct zones with the moment resistance defined by a different equation in each zone. The first and second zones are affected by elastic and elasto-plastic lateral-torsional buckling, respectively. The third zone is governed by steel yielding. Several researches studied the phenomena of lateral-torsional buckling of steel I-section beams and the moment capacity of such beams. A brief description for the work done by different researches in the same field of interest of this thesis is included in this chapter.

2.1 Lateral-Torsional Buckling

Lateral-torsional buckling represents an important aspect in the design of steel beams. Most beams that are used in steel construction have greater stiffness about the axis which resists the bending moment than in the perpendicular direction. This results in the possibility of lateral-torsional buckling. Lateral-torsional buckling is the result of lateral deflection and twisting. The resistance to this type of buckling is dependent on the lateral bending stiffness of the cross section, the torsional stiffness of the cross section, the type of bracing (lateral or torsional), the position of the bracing along the length of the beam, the position of the bracing on the cross section, the stiffness of the bracing, the bending moment distribution along the length of the beam, the position of the load on the cross section, the material properties, the magnitude and distribution of residual stresses, the initial twist of the cross section, and finally the initial bow along the length of the cross section. To prevent lateral-torsional buckling, beams are braced against twisting of the cross section. Two types of bracing are commonly used: (1) torsional bracing, which resists twisting of the cross section directly; (2) lateral bracing, which resists twisting of the cross section by limiting the lateral deflection at a point away from the level of virtual rotation. In practice, it is common for beams to be braced laterally at the top flange by the floor or roof system. Figure 2.1 indicates some types of definite lateral support, Salmon and Johnson (1996). Previous studies have shown that, for cantilever beams, torsional bracing is more effective than lateral bracing, assuming that both types have infinite stiffness, Nethercot and Al-Shankyty (1979); Kitipornchai and Richter (1983).

The strength or resistance of a beam in flexure is limited by some combination of local and overall buckling resistances. Figure 2.2 shows the general behavior of a wide flange beam, Yura et al. (1978). Basically, the behavior of the beam is divided in three response regimes: elastic, inelastic, and plastic ranges. In the elastic range, elastic buckling controls the behavior. In the inelastic range, some or all of the cross-section is yielded but only a small amount of inelastic deformations is available prior to failure. In the plastic range, the cross-section reaches the plastic moment M_p, and maintains this load level as the member tends to undergo large plastic deformations allowing for moment redistribution, Aktas (2004).

Pratyoosh and Blandford (1996) observed the lateral-torsional buckling of non-prismatic I-beams. They developed a finite-element model to analyze lateral-torsional buckling behavior of the non-prismatic simple or continuous beams with linear or quadratic tapered webs. The influence of warping deformations of the section and location of the member's loads are included in the formulation of the finite-element model. A parametric study on the lateral-torsional buckling strength is carried out for different forms of degree of taper, loading and support conditions for single-span and two-span continuous beams. They concluded that the buckling capacity increases significantly with the increase of linear taper for simple-span beams subjected to a uniform load applied at the section centroid. However, the maximum buckling loads are obtained if quadratic taper is provided. Linear taper for fixed-fixed supported beams leads to higher buckling capacities for very larger tapers and buckling loads can be increased significantly by providing quadratic taper. Their study also showed that the buckling loads for the two-span beams can be increased significantly when the loads applied on the second span is almost 50-60% greater than that applied on the first span. They recorded obvious increase in the buckling load by quadratically tapering the web depth towards the interior support or by tapering the flange thickness towards this support.

Earls (1998) conducted an experimental investigation to predict the ductility of wide flange beams made from high performance steel (HSLA80) subjected to moment loading. The loaded beams displayed inelastic modes of failure which do not lend themselves to a notional de-coupling of so-called local buckling and lateral-torsional buckling phenomena. Rather, the inelastic modes of failure of the HSLA80 tested beams displayed two distinct inelastic buckling patterns at failure, both of which exhibit localized and global buckling components. The structural ductility of beams is very much dependent upon which of the two mode shapes govern at failure. Cross-sectional proportions, bracing configuration, and geometric imperfections all play a role in influencing which mode governs in the beam at failure. He concluded that currently cross-sectional compactness and bracing for structural ductility in the AISC-LRFD (2005) may not be applicable for HSLA80 beams.

In the current AISC code, members are classified as, non-compact and compact as a means for characterizing their strength and deformation capacity. For sections permitted in plastic analysis, the AISC (2005) requires a compact section. The specification defines a compact section as one that can develop a fully plastic stress distribution while exhibiting sufficient plastic hinge rotation capacity, prior to the onset of local buckling, to accommodate moment redistribution in the structural system.

Vila et al. (2003) studied the effect of residual stresses on the lateral-torsional buckling of steel I-beams at elevated temperature. A numerical investigation of the lateral-torsional buckling of steel I-beams subjected to a temperature variation from room temperature up to 700^{o}C, with the aim of assessing the effects of the residual stresses in this mechanism of failure. For this purpose, a geometrically and materially non-linear finite element model has been used to determine the lateral-torsional resistance of steel I-beams at elevated temperatures. They concluded that Young's modulus decreases faster than the yield strength when the temperature increases. This conclusion along with the fact that the stress–strain relationship at elevated temperatures is not the same as that at room temperature, produce a modification to the lateral-torsional buckling curve at elevated temperatures. To overcome this problem, a new beam design curve has been proposed. They also concluded that the buckling resistance of the beams is less sensitive to the residual stresses when the temperature increases. This is probably a result of the smaller difference between the yield stress of steel and the level of residual stresses that is characteristic of elevated temperatures.

Karl et al. (2005) reviewed the performance and accuracy of the web compactness limits employed by AISC (2001) and AASHTO (2004) specifications. In order to evaluate the performance of the above web compactness limits, a significant number of finite element analyses were conducted with the goal of determining if girders satisfying the above compactness limits would achieve their intended moment capacity M_p. They concluded that AISC (2001) and AASHTO (2004) compactness limits do not adequately account for the behavior of mono-symmetric sections having geometries typical of that used in contemporary bridge construction. However, AASHTO (2004) web compactness limit overcomes these limitations, ensuring that the plastic moment is developed throughout a range of geometric ratios investigated.

2.2 Critical Moment Initiating Lateral-Torsional Buckling

Beams loaded about the strong axis may buckle in lateral-torsional buckling mode. The well-known analytically derived equation for the critical elastic buckling moment M_{cr} of a simply supported beam, loaded by a moment which is constant along the span was published by Timoshenko and Gere (1961):

$$M_{cr} = \frac{\pi}{L}\sqrt{E\,I_y G\,J + \left[\frac{\pi\,E}{L}\right]^2 I_y\,C_w} \tag{2.1}$$

Where E is Young's modulus (Modulus of elasticity), I_y is the moment of inertia about the weak axis, G is the shear modulus, J is the torsional constant, L is the beam span and C_w is the warping constant.

Equation 2.1 shows that the moment of inertia about the weak axis, the torsional constant and the warping constant are the parameters that affect the resistance to buckling, together with the lateral unsupported length of the beam. I-sections are often used as beams because of the favorable ratio between resistance and weight. A disadvantage of I-sections, however, is that they have a relatively low critical elastic buckling load because of the relatively low torsional constant, the low warping constant and the small ratio between I_y and I_z where I_y is the second moment of inertia about the strong axis.

The AISC LRFD specification (AISC 1999) uses equation 2.1 modified by a factor C_b which accounts for non-uniform moments along the length of the beam. For inelastic buckling, the capacity is interpolated linearly between the elastic buckling moment and the plastic capacity of the cross section. Residual stresses are accounted for explicitly by limiting the elastic stress to the yield stress minus the residual stress. The equation for the elastic buckling moment is

$$M_{cr} = C_b \frac{\pi}{L}\sqrt{E\, I_y G\, J + \left[\frac{\pi\, E}{L}\right]^2 I_y\, C_w} \qquad (2.2)$$

To obtain the design capacity, M_{cr} is multiplied by a reduction factor φ_b to account for design inaccuracies.

Helwing et al. (1997) used the finite element buckling analysis to examine the lateral-torsional buckling of singly symmetrical I-beams. Mid-span point load and uniform distributed load were investigated. They concluded that for single-curvature bending, traditional values for moment gradient factors C_b can be used to estimate the buckling capacity of singly symmetric girders with ($0.1 \leq \rho \leq 0.9$) and where

$$\rho = I_{yc} / I_y \qquad (2.3)$$

I_{yc} is the moment of inertia of the compression flange and I_y is the weak-axis moment of inertia of the cross section. The finite element results of Helwing et al. (1997) demonstrated that the height of the load application on the cross section has a significant effect on the buckling capacity. If the load is not applied at the mid-height of the cross section, they recommended that the moment gradient factors C_b has to be modified with a modification factor.

Park and Michael (2003) studied the lateral-torsional buckling of stepped beams. They used the results of the finite-element buckling analysis for stepped beams under uniform moment to develop new proposed design equations that account for the change in the cross section of stepped beams. Traditional moment gradient factors for prismatic beams were reviewed and found to be sufficiently accurate for some stepped beam cases. Comparisons were made between the proposed equations and finite element model results for doubly and singly stepped beam spans of existing highway bridges. The comparisons indicated that proposed equations produced conservative estimates of the lateral-torsional buckling resistance for these cases.

Johan et al. (2004) illustrated the relation between the critical elastic buckling load and the resistance of a beam (Figure 2.3). The horizontal axis in this figure represents the lateral deflection of the cross-section at mid-span of a simply supported beam loaded by a concentrated load applied at mid-span. They stated that for beams with a much higher critical elastic buckling load, buckling does not significantly influence the beam resistance. The resistance of such beams is dominated by the plastic capacity, whereas for beams with a much lower critical elastic buckling load, the beam resistance is dominated by buckling. In codes such as the Euro code 1993, the influence of the critical elastic buckling load on the beam resistance is expressed as a function of the relative slenderness λ_{LT} as shown in Equation 2.4:-

$$\lambda_{LT} = \sqrt{\frac{M_{pl}}{M_{CR}}} \tag{2.4}$$

They also illustrated in Figure 2.4 the ratio between the beam resistance and the plastic capacity (χ_{LT}) as a function of the relative slenderness (λ_{LT}). For a beam with large slenderness, the critical elastic buckling load (solid black line) dominates the resistance, while the plastic capacity (grey line) dominates the resistance for stocky beams. The influence of initial imperfections is taken into account by the use of a buckling curve. In The Euro code 1993, buckling curve 'a' (dashed line) is used for the design resistance of rolled sections. This buckling curve is based on the results of many tests. As shown, the influence of initial geometric imperfections and residual stresses on the resistance is largest when the critical elastic buckling load is equal to the plastic capacity (at a relative slenderness equal to 1).

Sayed-Ahmed and Loov (2005) and Sayed-Ahmed (2005) proposed a single equation which defines the allowable bending stress for laterally unsupported steel I-section beams to cover the three zones of the steel beam behaviour (plastic, elasto-plastic and elastic). The equation was proposed to replace the discontinuous definitions currently adopted by the codes of Practice. The proposed equation results were compared to those obtained using the design different codes of practice provisions.

lopez et al. (2006) suggested a general expression for the moment gradient factor for the lateral-torsional buckling of steel I-section beams applicable to any moment distribution. Modern steel works design codes provide closed form expressions to compute the moment gradient factor for any bending moment distribution, but changes in the moment factor due to end support restrictions are not considered. They used the finite difference approach and provided new results for the moment gradient