

# The concept of maximum load health and social care essay



The increased popularity of the use of steel web tapered Tee section beams is explained by the two main advantages they provide. Firstly, the use of such beams is more preferable by architects, as in cases where the beams have to be visible they provide a more elegant and aesthetically pleasing solution (Fisher and Smida, 2000). Secondly, they are frequently used in an effort to minimise the total weight and subsequently minimize the cost of a structure (Raftoyiannis and Adamakos, 2010). Secondary advantages include standard fabrication processes and better corrosion protection. Figures 1 and 2 depict a couple of examples of the use of steel web tapered tee cantilevers in real life structures. Although these types of sections are mainly used for the advantages mentioned, when it comes to the beam's performance, the absence of a flange on the tapered side of the web significantly affects the beam's behaviour under loading. Therefore, when this type of beam is used as a cantilever under bending load, since one side of the web is unrestrained and the web carries the bending stiffness of the beam, global or local instabilities may occur (Smida, 2004). It is generally accepted that relatively short beams tend to buckle locally and long beams tend to buckle laterally (Ellison and Corona, 1998). However, a series of other factors affecting their buckling behaviour will be discussed later, as for example whether the tip of the cantilever is latterly unrestrained or not. As Spellman and Whiting (2010) explained, buckling of a structural member can be simply defined as a sudden failure of the member when it is subjected to a high compressive stress, provided that the compressive stress at the point of failure is less than the ultimate compressive stress that the material of the member is capable of withstanding (also known as failure due to elastic instability). The buckling failure mode occurs as a global or a local instability. However, in <https://assignbuster.com/the-concept-of-maximum-load-health-and-social-care-essay/>

some cases the two above mentioned instabilities may be combined, resulting in a coupled instability. Lateral torsional buckling (LTB) occurs as global instability. The buckling mode of the member involves an out-of-plane deflection and a twist about the shear centre of the cross-section, both occurring simultaneously (New Steel Construction, 2006). LTB may occur in an unrestrained beam. Local buckling occurs as a local instability.

Considering for example a steel member of T-shaped sections acting as a cantilever, when loaded, local buckling failure mode develops as out-of-plane ripples, or buckles, limited to a small region along the length of the web of the member, without overall lateral displacement or twisting (Fisher and Smida, 2000). This project focuses on the investigation of the local buckling behaviour of steel web tapered tee section cantilevers. However, most researches about local buckling concern plate elements. Hence, there is a need of implementing the laws of plate theory in order to analyse steel beam sections for local buckling. Plate elements can be assembled into complete members by either the basic rolling process (hot rolled sections), or by folding (cold formed sections) or by welding. According to Stiemer (2012), the efficiency of such sections is due to their use of the high in-plane stiffness of one plate element to support the edge of the other plate element connected to the former. This results to the control of the out of plane behaviour of the supported plate. Therefore, structural steel elements, rolled or welded, can be taken as if they were made of plate elements which can be either internal or external. This assumption is also supported by Trahair et al. (2008), who explained that the cross-section of a tee beam is composed of two slender plate elements (flange and web) which may buckle locally and cause premature failure of the member before it reaches its yield strength.

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Hence, the local instability of the web of the section, where in the case of local buckling the failure is likely to occur is similar to the local buckling failure of a steel plate with one side free along its length (Fisher and Smida, 2000). As Causevic and Bulic (2011) explain, plate elements of relatively thin cross sections are likely to buckle locally when subjected to in-plane compression, generated either by compressive forces acting on the cross-section or by bending i. e. the cross-section of a beam subjected to vertical loading (pointload and/or uniformly distributed load). Moreover, as indicated in Figure 4, a slender plate element does not fail due to the elastic buckling occurrence, but it exhibits significant post-buckling behaviour by being able to withstand additional load. In other words, in the elastic range the buckled portion of the web plate shed load and becomes ineffective in carrying more load, while the portion of the plate close to the support has post-buckling strength and stiffness reserves (Rhodes, 2002). Based on this fact, it can be understood that the amount of resistance a plate will exhibit is dependant not only on its slenderness but on its yield strength and any residual stresses mainly from fabrication as well. However, what is important is to identify the critical load under which local buckling is initiated even though the plate will not fail yet. Hence, the plate's behaviour has to be investigated in the elastic range. It is important to point out that the resistance of a plate of intermediate slenderness is significantly affected by any geometrical imperfections which are likely to exist, while the resistance of a stocky plate depends primarily on its yield stress (Trahair et al. 2008). The smallest value of load producing local buckling is called critical or buckling load (Ventsel and Krauthammer, 2001). Jones (2006) defined buckling load more specifically, as the load at which the following effects take place: the current

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equilibrium state of a plate element changes from stable to unstable and the equilibrium state suddenly changes from the previously stable configuration to another stable configuration with or without an accompanied large response (deformation). Generally it is said that a structural element has buckled if it ceases to deform in its original equilibrium shape (Jones, 2006). In other words, buckling occurs when the path on the load-deflection curve changes direction at the buckling load (or maximum buckling load). The concept of maximum load (or critical load) is of vital importance in structural design as the aim is to design a structural element or a structure to carry a specific load with an acceptable factor of safety against failure. Hence, the analysis of the structural element is essential in order to determine the amount of load it would carry without failures of any nature and in this specific case, without the element buckles locally. As Jones (2006) pointed out, the procedure of calculating the buckling load of plates is the two-dimensional analogy of Euler's buckling load and for that reason, the plate buckling load is also known as Euler buckling load. According to Rhodes (2002) and Stiemer (2012), solutions to plate buckling problems were first obtained by Bryan (1891), which might be referred to as "classical" solutions as they were the first and simplest solutions for basic loading conditions. Bryan presented the analysis of the elastic critical stress for a rectangular plate simply supported along all its four edges and subjected to a uniform longitudinal compressive stress. Since then, a long series of scientific testing of buckling of plates has been carried out as by Schuman and Back (1930) and the theories developed back then have been used ever since. Therefore, through a combination of a series of experimental investigations and analyses, it was proved that the elastic critical stress of a long plate segment

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is determined by the elastic material properties of the plate, its width to thickness ratio and the buckling coefficient  $k\sigma$ , which takes into account the boundary conditions and the distribution of stresses on the plate as will be further explained. Hence, the elastic buckling stress of unstiffened steel plates is calculated using equation 1 as given in BS EN1993-1-5. From equation 1, it is observed that the critical buckling stress in the longitudinal direction is a function of the width  $b$ , in the transverse direction. This relationship between the stress and the width was explained quite simply by Narayanan et al. (2012). The first thing to understand is that as the compressive load on the plate is increased and reaches the critical buckling load, the central part of the plate such as the strip AB tends to buckle. Secondly, considering a transverse strip CD, it can be understood that this strip resists the tendency of the strip AB to deflect out of the plane. Hence, the longer the width more will be the resistance offered by CD to AB. Therefore, the strip AB until buckling behaves like a column on elastic foundation, whose stiffness depends on  $b$ , as shown in the equation. The buckling coefficient  $k\sigma$  is depended on the boundary conditions and the plate's dimension ratios. As Causevic and Bulic (2011) explain, for different support conditions around the plate and different length to width ratio  $L/b$ ,  $k\sigma$  changes significantly having a drastic influence on the critical stress calculation. The first step in determining the buckling factor  $k\sigma$  is the analysis of boundary conditions of the plate element, where it has to be determined whether the plate is either an internal or external element (Causevic and Bulic, 2011). An internal element means that the plate is supported (restrained) to all of its sides whereas an external has at least one side free i. e. the flange of an I-beam or the web of a T-beam. As goes for an <https://assignbuster.com/the-concept-of-maximum-load-health-and-social-care-essay/>

external plate with one edge free, as in can be seen in figure 7, as the length of the plate increases and hence the ratio  $L/b$  increases as well, the buckling factor  $k_{\sigma}$  tends toward the limiting value of 0.425. This clearly shows that plates with free edges do not perform well under local buckling (Kumar and Kumar, 2012). The second step includes the distribution of stresses along the element's cross section generated by the forces acting on it and hence the web plate, as the stress ratio at the boundary fibres,  $\psi = \sigma_2 / \sigma_1$  (Causevic and Bulic, 2011). The stress ratio depends on whether the elements cross-section is subjected to pure compression or bending and on the cross-section shape itself. To explain that, when a symmetrical section is subjected to pure compression, the stress ratio will be 1 as there would be a uniform stress distribution ( $\sigma_1 = \sigma_2$ ) whereas for pure compression of again a symmetrical section, the stresses would be of equal magnitude but opposite signs ( $\sigma_2 = -\sigma_1$ ). However, that is not the case when the section is unsymmetrical or when the section is not under uniform compression as for example a tee section in bending. In that case, the distribution of stresses is uneven and the stress ratio is not 1 or -1 anymore and the stress ratio  $\psi$  accounts for the stress gradient as tensile ( $\sigma_2$ ) over compressive ( $\sigma_1$ ) stress. The effects explained previously must therefore be considered in order to determine  $k_{\sigma}$ . An equation for  $k_{\sigma}$  depending on the case of stress ratio  $\psi$  and the support conditions of the beam and hence the plate element of the section, can be found in tables 4.1 and 4.2 of BS EN1993-1-5 (appendix A) for internal and external compression elements respectively. For a tee section the web which is the part of the section that is of major concern for local buckling occurrence is considered as an external element as it has one longitudinal edge free. Hence, for a cantilever with a point load at the tip, using table 4.2

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of BS EN1993-1-5, the equation for  $k$  is the following: Eurocode 3 also introduces the method of effective width; according to which, a reduction factor  $\rho$  is introduced in plate elements without longitudinal stiffeners to account for the effective width of the compression zone of the plate. Hence, for outstand compression elements the reduction factor  $\rho$  is: So far it has become clear that the boundary conditions and the type of loading applied are the two factors which govern the stress equation and therefore the determination of the critical buckling load. Hence, the coefficient  $k$  which accounts for these two factors, is the key element to consider for any improvement on the estimation of the critical stress. From research in literature, a couple of examples were found where the buckling coefficient was modified. The first case is Smida's (2004) study on the load capacity of T-shaped sections considering both lateral torsional and local buckling. Their study concluded with a modification to the buckling coefficient equation given by Eurocode, for the same boundary and loading conditions as the ones considered in this project. Their new equation of  $k$  in opposition to equation 2 by Eurocode is: Smida (2004) also considered the method of effective width, where the reduction of the stiffness of the cross-sectional area and the reduction of the global slenderness are taken into account by equations 4 and 5 respectively. Another alternative approach is the one proposed by Trahair et al. (2008) in their textbook "The Behaviour and Design of Steel Structures to Eurocode 3". According to this book the buckling coefficient is a function of the plate's aspect ratio. Hence, for a plate free along the longitudinal edge,  $k$  can be approximated using the following equation: Classification of the cross-section is stated in both BS EN1993-1-1 and BS 5950-1: 2000 and its purpose is to identify the extent to

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which the resistance and rotation capacity of the cross-section is limited by its local instability resistance. In other words, determine whether local buckling influences the capacity of the cross-section without any actual calculation of its resistance to it. BS EN1993-1-1 code of practice defines the four classes of cross-sections quoted below: The classification of a cross-section depends on the width to thickness ratio of the parts of the section which are either totally or partially subjected to compression (due to bending or pure axial compression). The limits which define the class of the section depend on the steel grade and become more severe for higher grades of steel. The reason being that higher grade steel sections are likely to be more stressed and hence, the possibility of local buckling is increased. It is important to emphasize that increased strength of the steel does not provide any improvement in local buckling behaviour provided that the slenderness ratio remains the same (Tata Steel, 2013). Furthermore, in the case when the flange and the web of a cross-section are of a different class, the whole cross-section is classified according to the highest (least favourable) class of its parts. When it comes to designing a T-section subjected to bending, local buckling is one of the main considerations. Therefore, the classification of the section is rather important. Table 5. 2 of the BS EN1993-1-1 code of practice (see appendix A) can be used in order to classify a T-shaped section. The table does not provide a clear way of how to classify such a section, however, as this type of sections consist of two outstand elements, part 2 of the table for outstand elements may be used. Depending on the stress distribution the section can then be classified. As this process tends to be rather complicated, the SCI Advisory Desk (2011) suggests the use of BS 5950-1: 2000 code of practice in order to design T-sections in bending. Table <https://assignbuster.com/the-concept-of-maximum-load-health-and-social-care-essay/>

11 of this document (see appendix A) provides the limiting width to thickness ratio for T-sections, rolled or cut from a rolled I or H section. Depending on the value of the width to thickness ratio the section is then classified to one of the four classes. Slender sections (very wide relatively to their thickness) which of course are more viable to local buckling are almost always class 4 which proves their limited resistance to local instabilities. For tapered tee cantilevers the cross-section has its highest depth at the support and reduces along the length of the cantilever. Hence, the classification of the section along the cantilever's length changes, usually start from class 4 to end up to class 2 or 1. Therefore, for the purpose of this project, the section must be classified at various space intervals starting from the support and moving towards the tip in order to see the variation of the cross-section classification and how it would be related to the experimental outcome.