Study of nonlinear static finite element analysis and plastic behaviour of steel beam section

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Abstract - *In the most recent researches, it has been a trend to focus on understanding the behavior of structures, i.e. to understand structural capacity. But, it is not possible to understand structural capacity without having sufficient insight on the demand in terms of seismic performance induced on the structures. This is one of the major uncertainties a structural engineer has to face. These uncertainties can be addressed to a sufficient extent by proving adequate amount of ductility to a structure. Inelastic behaviour of structural members greatly influence overall seismic performance of the structure. In particular, global structural ductility is greatly dependent on individual member ductility that in turn, depends on section ductility. Thus, in the moment frames, reliance is placed on curvature ductility capacity in plastic hinges expected to be formed at the beam ends. So the major attention is focused on Steel Moment Resisting Frames and in particular on the member behavior. Evaluation of the section curvature ductility is thus vital in undertaking any form of nonlinear analysis of structure. In absence of a definite analytical approach to analyze inelastic buckling of multiple component plates of a section (e.g., both flanges and the web in an I-section), a nonlinear finite element analysis based on the numerical methodology using commercial finite element package ABAQUS software that allows the prediction of the initiation of local buckling, and to evaluate the curvature ductility capacity of doubly symmetric I- section beams is proposed to study analytically.*

Key words: Curvature Ductility, Inelastic Buckling, Local Buckling.

1. Introduction

In the design practice it is generally accepted that steel is an excellent material for seismic-resistant structures, due to its performance in terms of strength and ductility, as it is capable of withstanding substantial non-elastic deformations. This is generally true because, in percentage, the number of failure of steel structures has been always very small as compared to other constructional materials. Earthquakes are generally considered to be the most devastating and destructive of all natural disasters. Every day, minor ground shakings are registered in some or the other part of the world, but due to their small intensities, they do not cause significant damages. This poses a challenge to the designers, to have a balance between seismic demand and structural capacity. In the most recent researches, it has been a trend to focus on understanding the behaviour of structures, that is, to understand structural capacity. But, it is not possible to understand structural capacity without having sufficient insight on the demand induced on the structures. This is one of the major uncertainties a structural engineer has to face. As far as the seismic design philosophy is concerned, partial or total collapse of structure is not acceptable. These uncertainties can be addressed to a sufficient extent by proving adequate amount of ductility to a structure. Among currently available design concepts, ductility based design of structures assures a minimum expected level of performance, when a structure is subjected to strong ground motion. The ability to predict ductility demand, and available ductility, under seismic loads are key concerns in seismic design. It is also important to ascertain the amount of structure ductility provided, along with other equally important criteria like strength, stability and deformation checks are essentially needed.

2. Ductility in general

In common practice, the term ductility is used for evaluating the performance of structures, by indicating the quantity of seismic energy which may be dissipated through the plastic deformations. The use of the concept of ductility allows the possibility to reduce the seismic design forces and to produce some controlled damage in the structures, in case of strong earthquakes. In the practice of plastic design of structures, ductility defines the ability of a structure to undergo deformations after its initial yield, without any significant reduction in the ultimate strength.

The ductility of a structure allows us to predict the ultimate capacity of a structure, which is the most important criteria for designing structures under conventional loads. Recent development of advanced design concepts, as the capacity based design method, is based on the objective to provide sufficient ductility to a structure, in the same way as rigidity and strength. For this the evaluation of ductility needs to be done.

The aim of this paper is to study the prediction of the initiation of local buckling, under moment gradients. This, in turn, will help in predicting the moment curvature relationship of the available cross-sections and thereby understanding the influence of flange and web slenderness for a cross section. The resulting available moment curvature relationship of different types of the cross sections can be readily used for monotonic loading cases.

3. Literature Survey and background to research

Literature studied in relevance to the objectives of the present study. There are various studies carried out by the researchers on plastic behaviour of steel beam section, different models with different geometry have been developed that are based on conceptual representation using finite element method. In (2012), D'Aniello et al. [1] carried out work on monotonic and cyclic tests on three hot-rolled cantilever I-beams Sections. The same cantilever beams are modelled in this study. For the validation purpose, only one wide flange beams is considered.

Section classification is one of the key points of IS 800 & other relevant standards, which use different terminology and slightly different limit ratios of width to thickness to check whether the local buckling initiates before elastic limit, between elastic and plastic limits or after the plastic limit. Most of the design codes do not evaluate the available ductility with different types of sections. Instead they classify the sections to be plastic, compact, semi-compact and slender depending upon their geometrical properties without specifying the limit of deformations that whether these sections can undergo without reduction in their flexural capacities. So the attention is focused on the classification criteria of steel members and steel sections adopted by principal steel design code like IS 800.This preliminary work has allowed the understanding that the main factors that influence the classification is curvature ductility capacity. This parameter is to be defined and studied, making it possible to underline the necessity of a section classification criterion in terms of section ductility. A cantilever beam experiment is analysed using the full shell model and its appropriateness is established. From the validation of the model, it is concluded that if appropriate constitutive relation is used, the results of the full shell model would be very close to experimentally-observed results, in flexural members.

Aim of Study

To study nonlinear static finite element analysis and plastic behaviour of steel beam section.

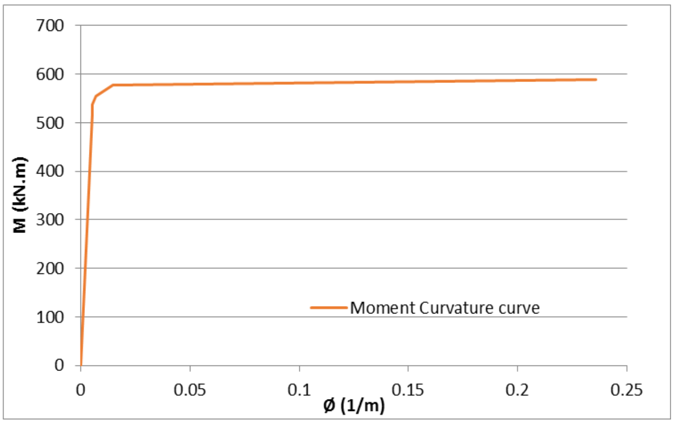

Fig.1: Moment Curvature Relationship for ISWB-500 (Elasto-Plastic)

Objective of Study

Following objectives were finalized for the present study:

1. To The model that studied for the advanced analysis is validated for the in-plane behaviour of steel frames made of hot rolled and shallow I-sections.
2. The imperfection patterns based on buckling modes that would cause the greatest reduction in strength and which resemble the standard codal imperfections can be used.
3. The bilinear and tri-linear elastic plastic isotropic strain hardening material model is used for the static analysis respectively with the Von Mises yield criteria used for defining the yield surface in structure.

4. Finite element modelling using abaqus

Finite Element Method (FEM) is widely used in analysis and design of structures. It can be used with different degrees of sophistication for different purposes. Common applications are linear elastic analysis and geometrically non-linear elastic analysis of frames. Such analyses provide load effects, and together with code-specified limiting criteria, form a design method for structures. These methods are well established for frames using beam elements. The imperfections needed to perform a non-linear analysis are bow and sway imperfections.

SHELL ELEMENT MODEL:- In this model, the entire frame structure is modelled using the shell elements. This approach is advantageous as the shell elements can model all the influencing factors such as the elastic/plastic local buckling, element and member initial imperfections, inelasticity and large deformation at any location of the structure. The model with the entire frame modelled using shell elements is among the best in the advanced analysis methods in terms of modelling accuracy, but is complex and time consuming. Four-node shell element with three translation and three rotation degrees of freedom at each node is used in this study.

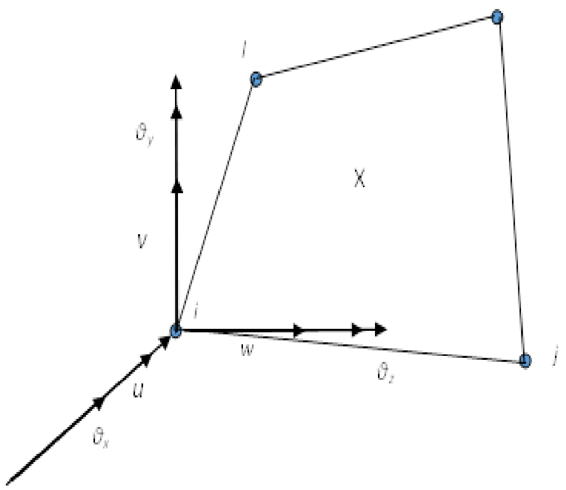

Fig.2: Shell element (X shows integration point)

SHELL: Default number of integration points is used in the shell elements. The model with initial imperfection for the nonlinear analysis is obtained by superposing on the perfect model, the appropriately scaled (to obtain the desired magnitude of imperfection) buckling modes from the Eigen value buckling analysis. The critical location identified from the full beam element model analysis is taken as the location of the maximum imperfection amplitude. By using these imperfections with the maximum imperfection provided at the critical location, one would obtain the result with the maximum impact of the imperfections for the structure being evaluated. A longitudinal residual stress pattern, assumed to be constant throughout the member, is used in the study. Since this study is limited to the planar behaviour of frames, restraints are applied at all the nodes at the intersection of the two flanges and the web by arresting the translational degrees of freedom in the out-of-plane direction of the web. First an Eigen value buckling analysis is performed using the sub-space iteration technique or the Lanczos iteration technique to obtain the mode shapes.

The first mode shape corresponding to each mode of buckling (local and overall) are scaled and superposed on to the original geometry of the structure to obtain a model with initial imperfection before the start of the nonlinear analysis. The nonlinear analysis is performed with 'modified Riks' solution technique to capture the nonlinear and post peak response.

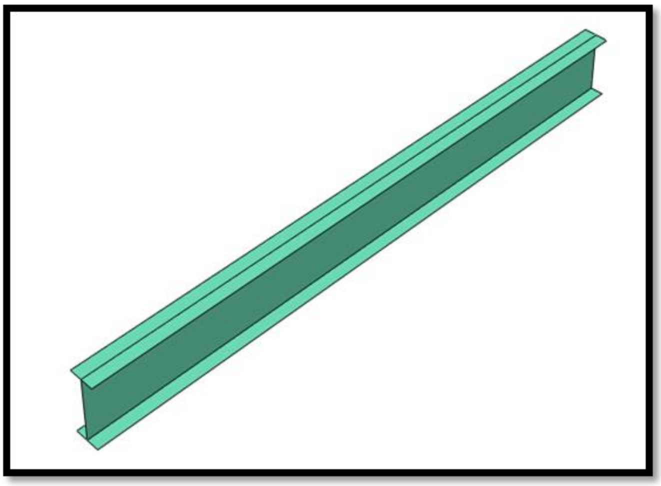

Fig.3: Basic model of typical I-section beam

Geometric Details: Commercial finite element package ABAQUS is used for modelling the geometry and analysing the results in this study. In this first module the different parts which constitute the model are geometrically realized. ABAQUS has several element types suitable for numerical analysis: solid two and three-dimensional elements, membrane and truss elements, beam elements, and shell elements. The major aim of the analysis was to predict the formation of inelastic local instabilities in a cross section and their corresponding rotation capacity. Beam, membrane and truss elements are not appropriate for the buckling problem. Solid three dimensional elements ("brick" elements) may be suitable, but the solid elements have only translation degrees of freedom at each node, and require a fine mesh to model regions of high curvature. A finer mesh does not necessarily imply more total degrees of freedom, as one must consider the number of elements and the degrees of freedom of each element. The most appropriate element type for this study is the shell element. The various elements, analysis techniques, loading and boundary conditions, material models and imperfection details used in the study are explained in the following sections. S4R quadrilateral shell element, which is available in ABAQUS element library, is used for modelling the I-section plate elements in the shell element regions of the models.

S4R is a 4-node, quadrilateral, stress/displacement shell element with reduced integration and a large strain formulation. This element has six degrees of freedom (3 translations and 3 rotations), one integration point (located at the centroid of the element) and has five section points through the thickness. For these reasons, through the use of the graphic processor CAE in the ABAQUS module, the geometric characteristics of whole beam (l=6000 mm) have been modelled. The elements used for the representation of the beam were the three-dimensional extruded shell.

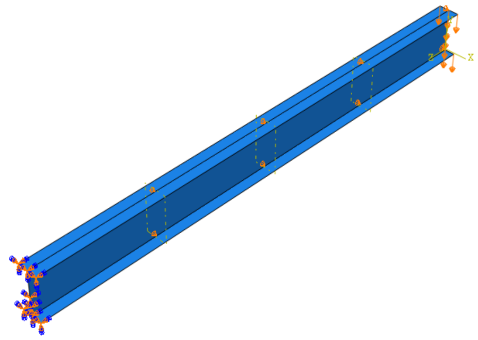

Fig.4: Numerical model of a beam showing support and loading.

5. Analytical results:

The validation of test Results: A cantilever beam experiment is analysed using the full shell model and its appropriateness is established. From the validation of the model, it is concluded that if appropriate constitutive relation is used, the results of the full shell model would be very close to experimentally-observed results, in flexural members.

In this part results are discussed by considering analyses of finite element analysis (FEA) models of beam sections with a range of flange and web slenderness using shell elements are presented. In addition to element slenderness, dependence of moment curvature relationship on other factors like degree of sectional imperfections and strain-hardening characteristics of steel are studied first.

Factors controlling curvature ductility

1.1 Imperfections

First, a perfect specimen without any imperfection is simulated and compared with available experimental results [D'Aniello et al, 2012]. The numerical estimate of ductility capacity is much higher than that observed experimentally. This is because; all real specimens have some amount imperfection that limits the ductility. Thus, imperfection is explicitly introduced in numerical model. It is seen that the extent of imperfection (d/400, d/300, and d/200) has large influence on the rotation ductility of the specimen.As a result; the amount of imperfection greatly affects the moment carrying capacity of a "plastic" section, particularly in the strain-hardening range (Chart 2). Thus, it is important to include suitable amount of imperfection in finite element models to have proper estimate of moment capacity and section curvature (or member rotation) ductility's. Imperfection considered in this study is local imperfection; the effect of global imperfection is not studied presuming that frame members in seismic applications will be laterally supported. Imperfections are introduced in FEA models as a scaling factor of appropriate buckle shape.

1.2 Strain-hardening of Steel

Generally, the ultimate moment capacity of a section cannot be reached due plastic buckling of component plates in the strain hardening range. Over strength, above the full plastic moment capacity Mp, is possibly high as seen in (Chart 2).

1.3 Flange Slenderness

Local buckling is initiated in the compression flange of doubly symmetric I-sections under flexural actions. To mobilise a plastic hinge in the member, it is important to control local buckling of the flanges. In other words, loss of moment carrying capacity is triggered by local buckling of flanges. This, in turn, controls ductility capacity of the section. The normalised moment rotation curves in the (Chart 3) are for the different values of flange slenderness (b/t), but a fixed value of web slenderness (d/t) of 60. It is observed that increase in flange slenderness decreases both (i) moment carrying capacity, and (ii) curvature ductility.

1.4 Web Slenderness

The normalised moment rotation curves in the (Chart 4) are for the different values of web slenderness (d/t), but a fixed value of flange slenderness (b/t) of 8.5. It is observed that increase in web slenderness decreases both (i) moment carrying capacity, and (ii) curvature ductility.

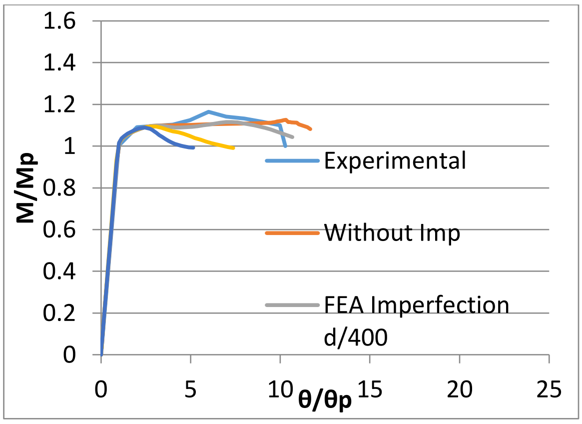

Chart -1: Normalised moment rotation curves with different levels of imperfections for ISMB 500 with (b/t = 8.50 and d/t = 47.53)

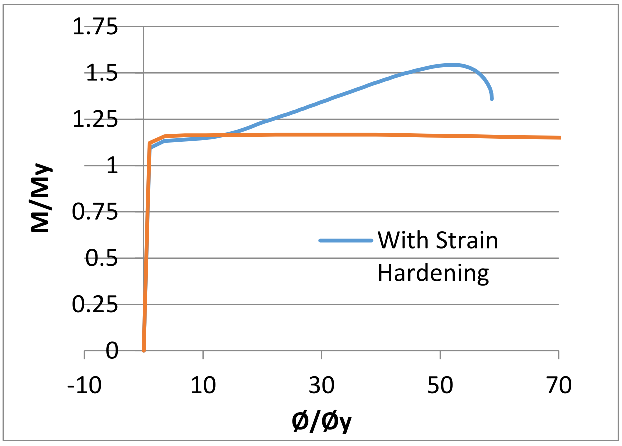

Chart -2: Normalised moment curvature curve with and without strain hardening (b/t=5 and d/t=45)

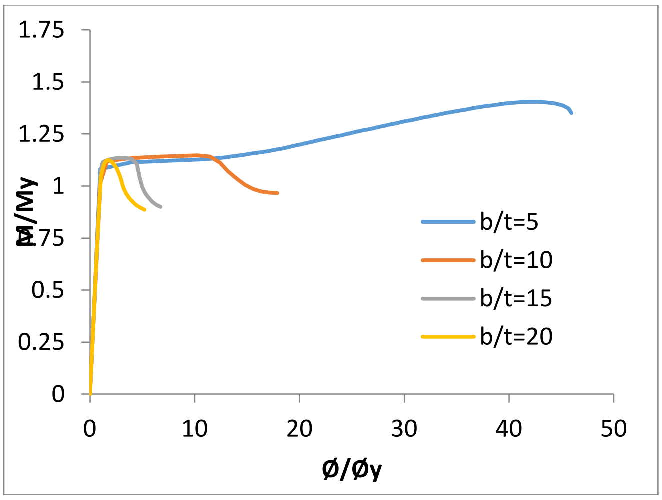

Chart -3: Normalised moment curvature curves varying b/t with constant d/t=60

6. Conclusion

1. It is seen that the extent of imperfection (d/400, d/300, and d/200) has large influence on the rotation ductility of the specimen. Increase in the level of local imperfection decreases curvature ductility capacity, as well as moment carrying capacity, of a section.

2. The study on normalised moment rotation curves for the different values of flange slenderness (b/t), but a fixed value of web slenderness (d/t) of 60 shows that increase in flange slenderness decreases both (i) moment carrying capacity, and (ii) curvature ductility.

3. The plastic strength ratio values obtained at the strain level of 0.125% for (b/t) value of 5,10,15,20 with constant (d/t) of 60 are 1.08, 1.00, 0.95, 0.88 respectively & similarly curvature ductility ratio values obtained at the strain level of 0.125% are 0.99, 0.97, 0.91, 0.86 respectively which shows that increase in flange slenderness decreases both moment carrying capacity & curvature ductility.

4. The study on normalised moment rotation curves for the different values of web slenderness (d/t), but a fixed value of flange slenderness (b/t) of 05 shows that increase in web slenderness decreases both (i) moment carrying capacity, and (ii) curvature ductility.

5. The plastic strength ratio values obtained at the strain level of 0.125% for (d/t) value of 45, 60, 90, 120 with constant (b/t) of 05 are 1.09, 1.08, 1.06, 1.05 respectively & similarly curvature ductility ratio values obtained at the strain level of 0.125% are 0.99, 0.97, 0.94, 0.91 respectively which shows that increase in web slenderness decreases both moment carrying capacity & curvature ductility.

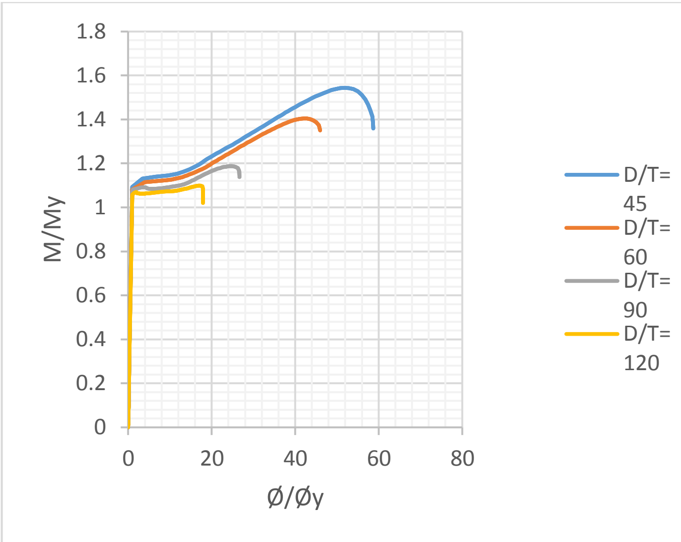

Chart -4: Normalised moment curvature curves varying d/t with constant b/t=05

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