FLEXURAL TORSIONAL BUCKLING BEHAVIOUR OF DOUBLE LONGITUDINALLY PROFILED FLANGED BEAMS (DLPB)

Sukanya S[1],Rajeevan B[2]
Assistant Professor, Veda Vyasa Institute of Technology, Karadparamba, Malappuram, Kerala, India
Postal Address: Susquare Villa, Providence-Kannadikkal Road, Vengeri P O,Kozhikode-
E-mail:sukanyasuresh97@gmail.com
Professor, Government College of Engineering Kannur, Kerala,
IndiaPostalAddress:GovernmentCollegeofEngineering,Mangattuparamba,
P. O.Parassinikadavu, Kannur,
E-mail:rajeevan@gcek.ac.in

ABSTRACT

The beams with flanges of varied thickness along the beam length is referred to as longitudinally profiled flanged beam. The present study investigated the flexural behavior of longitudinally profiled steel beams, subjected to flexural torsional buckling using ANSYS®. The longitudinally profiled beam was subjected to flexural torsional buckling by applying a couple at both beam ends. The tension flange was restrained in its position that ensured flexural torsional buckling only in the compression flange. By varying the depth-to-thickness ratio of the web from 42 to 100, the moment capacity was increased by 6.3%. The flange width to thickness ratio was varied from 8 to 15, which decreased moment capacity by 11.3%.

A comparative study was conducted to evaluate the moment of resistance of a longitudinally profiled beam with a normal steel beam. For a constant web thickness of 12 mm, the normalized moment capacity was 24.6% higher than the normal steel beam and 16.5% higher for a constant width-to-thickness ratio of 10. For an overall span of 3m, the normalized moment capacity showed 6% more moment capacity than the normal steel beam and 23.4% higher than the normal steel beam for grade of steel 690 MPa. For the various parameters, i.e., depth-to-thickness ratio of the web, width-to-thickness ratio of the flange, span of the beam and grade of steel, the moment capacity of a double longitudinally profiled beam was more when compared with the moment capacity of a normal steel beam.

Keywords: profile; flexure; torsion; buckling; beam; steel; double; normal

1.INTRODUCTION

Longitudinally profiled flanged beam is a beam with flange of varying thickness. By varying the thickness, economy is achieved by constraining the quantity of steel required. phenomenon most common in unbraced beams, flexural torsional buckling is a state o instability that has a profound effect on compression flange. When a load is applied on beam one of its halves is subjected to compression and the other to tension. Due to flexural torsion buckling, the tension flange is prevented from rotating around the axis. This results in failure of the member by local buckling of the compression flange. This is predominant in the bea not supported laterally or unrestrained, so the beam moves in the lateral direction.

There are three modes of failure or instability of a beam subjected to compression flexural buckling, torsional buckling, and flexural torsional buckling1

Flexural torsional buckling is similar as out-of-plane flexural buckling of beams. When lateral restraint is provided along the flange, torsional mode of instability occur instead of flexural torsional buckling2. The flexural torsional buckling behavior is attributed to free or retrained torsion.

When a beam is subjected to free torsion, the beam is prone to in-plane displacements and rotations due to twisting3 . Sudden buckling of the beam occurs on the principal plane of bea that has more stiffness and bend on that plane. Warping restraint is a boundary condition that occurs in a beam due to twisting. This results in out-of-plane distortions and leads to flexural torsional buckling strength of channel sections. The load carrying capacity is higher when warping is restrained than when warping is free at the boundaries of channel4 . Flexural torsional buckling occurs in beams when the bracings are not provided or when the bracings are placed with large spacing between them5. The inelastic strength of the beam is highly dependent on the stiffness of the restraint provided centrally and to the beam slenderness6

Beams with stiffeners reduce moment carrying capacity with increasing ratios of height to thickness of the web and breadth to thickness of flange7. A model with non-uniform cross section was recommended when subjected to flexural torsional buckling8 . In a web tapered

beam, the width of flange and the web is constant whereas the web depth is varied throughout the span. Such beams distribute stresses evenly throughout the beam9 . The support conditions influence the flexural torsional buckling behavior of profiled beams. To account for various conditions, the coefficient of friction was altered. Higher friction coefficient resulted in higher strength but higher deflection10 . A typical example of beam with non-uniform cross-section is a stepped beam. Steps provided at regions of higher moment makes the distribution of stresses uniform. The moment was more dependent on flange thickness than on flange width11 . Failure of a beams with longitudinally profiled flange occurred at the compression flange (top flange) due to local buckling of the beam12

2. RESEARCH SIGNIFICANCE

From the studies, it is evident that the use of longitudinally profiled flanged beam minimizes the quantity of steel required compared to a normal steel beam of uniform cross- section for carrying the same moment. The studies on behavior of longitudinally profiled flanged beam had rarely been studied. In an I-section, the web resists the shear and the flange resist the bending moment. Generally, the bending moment in a beam keeps varying throughout the length. Hence the provision of a flange with varying thickness resists moment efficiently with minimum and exact quantity of steel. Studies have been conducted on various cross-sections like web tapered beam, stepped beam which were efficient and economical in resisting the moment coming on the beam. The various studies show that the type of load, geometry, geometrical imperfections, grade of steel all have a significant effect on flexural torsional buckling. A beam of non-uniform cross section has a better response to flexural torsional buckling that members with uniform cross section. The longitudinally profiled beam has non-uniform cross section. The effect of flexural torsional buckling of longitudinally profiled flanged beams were investigated in the present study.

3. MODELING AND VALIDATION

The total span of the beam was 6000 mm, including a span length of 5600 mm and 200 mm overhang at both ends. The dimensions of the model have been tabulated in Table 1 and depicted by Fig 1. The longitudinally profiled beam was modeled using ANSYS® MECHANICAL 2019 R3 finite element modelling and analysis (FEM) software. Frictional coefficient of 0.5 at the two supports between the contact of the beam and the lateral bracing.

The Young’s Modulus and the Poisson’s ratio were adopted as 209950 MPa and 0.3, respectively for longitudinally profiled flanged beam and 207885 MPa and 0.3 for normal steel beam. The yield strength and ultimate strength of the steel were adopted as 502 MPa and as

621 MPa, respectively for longitudinally profiled flanged beam. The yield strength and ultimate strength of the normal steel beam were 450 MPa and 510 MPa, respectively. Multilinear isotropic hardening was used to incorporate the material property in the linear as well as nonlinear range based on the available tensile-coupon test results12, shown in Fig 2.

All the sections of the beam were modelled using 8-node 3D solid element SOLID185.

It is generally used for steel elements to capture the flexural behavior accurately. SOLID185 can also be used since the width to thickness ratio of the flange is small. The support conditions were simple supports. Torsion of the beam was prevented to obtain the full moment capacity of the section. A displacement of 34.42 mm and 24.56 mm at the point of loading (Fig 4.7) in case of double longitudinally profiled flanged beam and normal steel beam, respectively. The analysis was hence displacement controlled. It was found that at a mesh size of 27-30 mm, the results were similar and converging. 30 mm was chosen as the mesh size.

The number of steps adopted was 20 for both the linear and nonlinear analysis. The experimental results adopted12 were compared with the results of the finite element analysis and shown in Fig 3.

The deviation of the finite element results from the experiment was because the whole experimental setup could not be simulated in ANSYS®. In case of test, the experiment was stopped when the peak value of hydraulic jack was reached but in ANSYS®, the loading was continued and hence the curve was so obtained. Also, the variation in material property with the flange could not be adequately modeled in ANSYS®. This led to the variation of finite element analysis with the test results of Liu et.al.

4. FLEXURAL TORSIONAL BUCKLING

Flexural torsional buckling reduces the moment carrying capacity of the beam. The support conditions were provided to allow flexural torsional buckling to occur in the beam.

The horizontal displacement of the bottom flanges and the vertical displacement of the web were prevented13

A couple of magnitude 1 kNm was applied at both beam ends. The loading and support conditions of the beam are schematically represented in Figure 4.

5. RESULTS AND DISCUSSIONS

The effect of depth-to-thickness ratio of the web, width-to-thickness ratio of the flange and the total span on flexural-torsional buckling of double longitudinally profiled beams were studied. By changing the thickness of the web, the effect of depth-to-thickness ratio was investigated from 50 to 126. The width to thickness ratio of the flange was varied from 8 to 15, wherein the thickness refers to thickness of the flange at the midspan. The spans were varied from 3 m to 6 m and its effect has also been studied.

5.1 Effect of depth-to-thickness ratio of web

The web depth to thickness ratio (h t w w /) of the doubly profiled longitudinal beam wasvaried from 50 to 126. The effect of depth-to- thickness of the web on flexural torsional buckling at constant width and thickness of the flange and span is shown in Figure 5 and by

Table 2. The normalized value of moment to the plastic moment was reduced with increase in thickness of the web. As the web thickness is increased, the beam was less prone to local buckling than beams with thinner webs. Thus, the moment corresponding to yield point increased. It was observed that the ratio of moment to plastic moment increased by 6.3% as the web depth-to-thickness ratio was increased from 50 to 126.

5.2 Effect of width-to-thickness ratio of flange

The effect of width to thickness ratio of flange (b t f f /) was evaluated by varying the width of the flange and is shown by Figure 6 and Table 3. By varying the ratios from 8 to 15 and keeping the web thickness and depth a constant, the plastic moment was reduced with increase in width-to-thickness ratio of the flange. The moment capacity of the section was reduced after the yield point. As the width-to-thickness ratio of the flange was increased, local buckling is initiated early14

. The moment capacity of the beam reduced by 11.3% as the width- to-thickness ratio was increased from 8 to 15. Buckling occurring early indicates that the capacity of the beam reduced, hence width-to-thickness ratio and moment capacity thus had an inverse relationship.

5.3 Effect of span

The span of the beams (Lo) was increased from 3 m to 6 m whereas the other cross- sectional dimensions were kept constant. The effect on flexural torsional buckling was analyzed is shown in Figure 7 and Table 4. As the span of the beam was increased from 3m to 4m, the moment capacity of the beam rapidly increased by 15%. Beyond a span of 4 m, the capacity of the section was reduced with increase in span by 13.3%. The extent of buckling of beam was highly influenced by the unbraced length of the beams. For beams with larger spans, the flexural torsional buckling was more significant with increasing span and hence the moment capacity was reduced beyond a span of 4 m. In shorter beams, deformations were confined principally to the vertical plane. This was due to attainment of plastic hinge at higher moment. Thus, beams with a span of 3m showed a had greater moment capacity than slender

beams in the elastic region. For beams with span above 3m, significant deformation and

twisting occurred in addition to deformation and twisting. This led to inelastic instability and

hence had lower moment capacity.

5.4 Effect of steel grade

The moment-rotation of the double longitudinally profiled flanged beam was evaluated

by varying the grades of steel, with the other parameters a constant. The numerical value of

steel grade depicted in the figure refers to the yield stress and was varied from 345 MPa to 690

MPa15,16

. The variation of moment-rotation with steel grade on flexural torsional buckling is shown in Figure 8 and Table 5. As the yield stress was increased, the moment also increased and hence the normalized beam moment-rotation values increased. The normalized beam

capacity slightly reduced, by 7.8% as the grade of steel was increased from 345 MPa to 690 MPa.

5.5 Comparison of moment for Depth-To-Thickness Ratio of web, h t w w /= 42

The normalized moment-beam end rotation curves of the double longitudinally profiled flanged beam were compared with a normal steel beam for a constant web thickness of 12 mm and is shown in Figure 9. The dimensions of the double longitudinally profiled flanged beam

(DLPB) adopted were a span,

Lo= 6000.5 mm, overall depth

H=505.5 mm, width of flange

bf=201.5 mm, thickness of flange at supports

tfs=15.5 mm and thickness of web at midspan

tfm=23.5 mm. The various dimensions of the normal steel beam (NSB) were overall depth

H=497 mm, span

Lo= 6002 mm, width of flange

bf=201.5 mm, thickness of flange

tf = 23.5mm. The normalized moment capacity of double longitudinally profiled flanged beam (DLPB) was 24.6% higher than the normal steel beam (NSB).

5.6 Comparison of moment for width-to-thickness ratio of flange, b t f f / =10

To evaluate the flexural torsional buckling behavior for a constant width-to-thickness ratio of 10, the normalized moment-beam end rotation curves of the double longitudinally profiled beam was compared with that of the normal steel beam and is shown in Figure 10.

The dimensions of the all the beams (DLPB and NSB) were same as above, but with a width of flange bf=235 mm for both the double longitudinally profiled flanged beam and normal steel beam. The moment capacity of the longitudinally profiled flanged beam was also higher than normal steel beam (NSB) for the width-to-thickness ratio of 10. For the double longitudinally profiled flanged beam (DLPB), the normalized moment capacity of was 16.5% more than the normal steel beam (NSB).

5.7 Comparison of moment for span of the beam, Lo = 3 m

The flexural torsional buckling behavior of longitudinally profiled flanged beams were analyzed by comparing the normalized moment-beam end rotation curves of the double longitudinally profiled beam with a normal steel beam for an overall span

Lo =3m and is depicted by Figure 11. The moment capacity of the longitudinally profiled flanged beam was higher than normal steel beam (NSB). The normalized moment capacity of the double longitudinally profiled flanged beam (DLPB) was 6% more than the normal steel beam (NSB).

5.8 Comparison of moment for 690 MPa grade of steel

The normalized moment-beam end rotation curves of the double longitudinally profiled flanged beam were compared with that of the normal steel beam for the steel grade of 690 MPa, depicted by Figure 12. The normalized moment capacity of double longitudinally profiled flanged beam (DLPB) was 23.4% higher than the normal steel beam (NSB).

6 CONCLUSIONS

The following conclusions were made from the present study:

6.1 Flexural Torsional Buckling of Double Longitudinally Profiled Beam (DLPB)

i. As the web depth to thickness ratio of the doubly profiled longitudinal beam when varied from 50 to 126, the ratio of moment to plastic moment increased by 6.3% and was due to the local buckling of thicker webs.

ii. By varying the ratio of width-to-thickness ratio of the flange from 8 to 15, the plastic moment was reduced by 11.3%. The moment capacity of the section was reduced after the yield point. The capacity was reduced due to the uniform bending which occurs in the double longitudinally profiled beam.

iii. As the span of the beam was increased from 3m to 4m, the moment capacity of the beam rapidly increased by 15%. Beyond a span of 4 m, the capacity of the section was reduced with increase in span by 13.3%. As the yield stress was increased, the moment also increased and hence the normalized beam moment-rotation values increased with increase in steel grade.

iv. The normalized beam capacity has slightly reduced, by 7.8% as the grade of steel was increased from 345 MPa to 690 MPa.

6.2 Flexural Torsional Buckling of Double Longitudinally Profiled Beam (DLPB)

i. The normalized moment capacity of double longitudinally profiled flanged beam (DLPB) was 24.6% higher than the normal steel beam (NSB) for a constant web thickness of 12 mm.

ii. For a constant width-to-thickness ratio of 10, the normalized moment capacity of the double longitudinally profiled flanged beam (DLPB), was 16.5% more than the normal steel beam (NSB).

iii. Comparing the normalized moment capacity for an overall span of 3m, the double longitudinally profiled flanged beam (DLPB) possessed 6% more moment capacity than the normal steel beam (NSB).

iv. The normalized moment capacity of double longitudinally profiled flanged beam (DLPB) was 23.4% higher than the normal steel beam (NSB) for the grade of steel of 690 MPa.

v. Thus, for the different parameters examined, the moment capacity of the longitudinally profiled beam was more than the normal steel beam. For a reduced quantity of steel, the moment capacity of a longitudinally profiled beam was improved when compared with the moment capacity of a normal steel beam. The longitudinally profiled beam had better economy and strength compared to a normal steel beam.

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Table 1. Dimensions of beam specimens

Parameters	Dimension of DLPB (mm)	Dimension of NSB (mm)
Overall Length(L_o)	6000.5	6002.0
Overal lDepth(H)	505.5	497.0
Flange Width(b_f)	201.5	201.5
Thickness of flange at end of span(t_{fs})	15.5	23.5
Thickness of flange at midspan(t_{fm})	23.5	23.5

Table 2. Effect of depth-to-thickness ratio of web

h_w/t_w	M/M_p $(\Theta/\Theta_p=3.1)$
50	1.58
126	1.68
Percentage increase in normalized moment capacity	6.3%

Table 3. Effect of width-to-thickness ratio of flange

b_f/t_f	$M/M_p(\Theta/\Theta_p=2.9)$
8	1.78
15	1.6
Percentage decrease in normalised moment capacity	11.3%

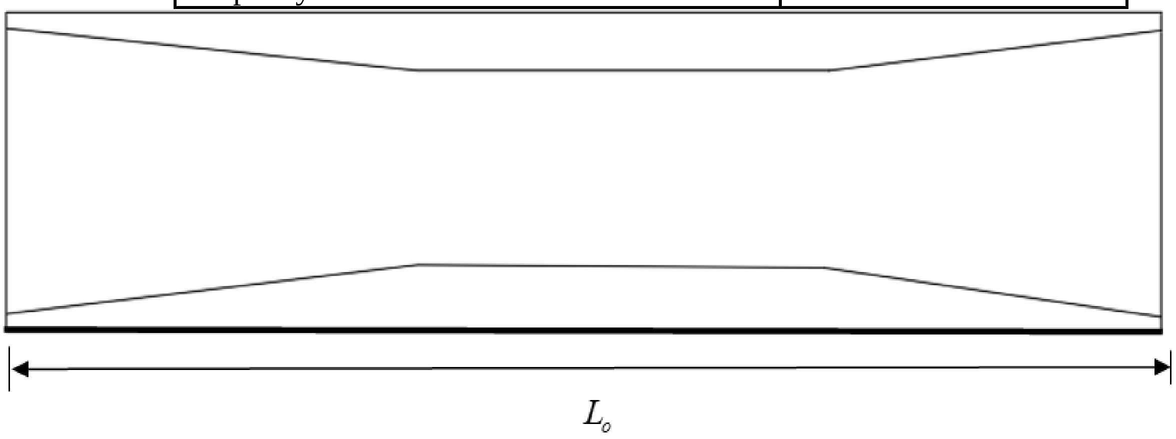
Table4.Effect ofspan

L_o	$M/M_p(\Theta/\Theta_p=1.9)$
3 m	1.18
4 m	1.36
6 m	1.20

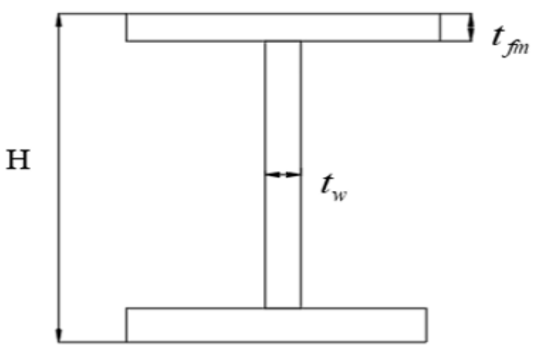
Percentage increase in normalised moment capacity (Lo=3 to Lo=4)	15.3%
Percentage decrease in normalised moment capacity (Lo=4 to Lo=6)	13.3%

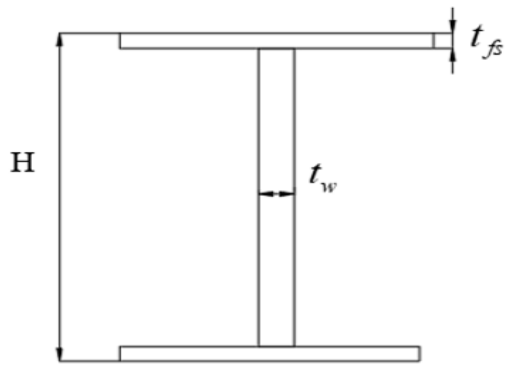
Table5.Effect of steel grade

Grade of steel(MPa)	$M/M_p(\Theta/\Theta_p=2.1)$
345	1.30
690	1.40
Percentage increase in normalised moment capacity	7.8%



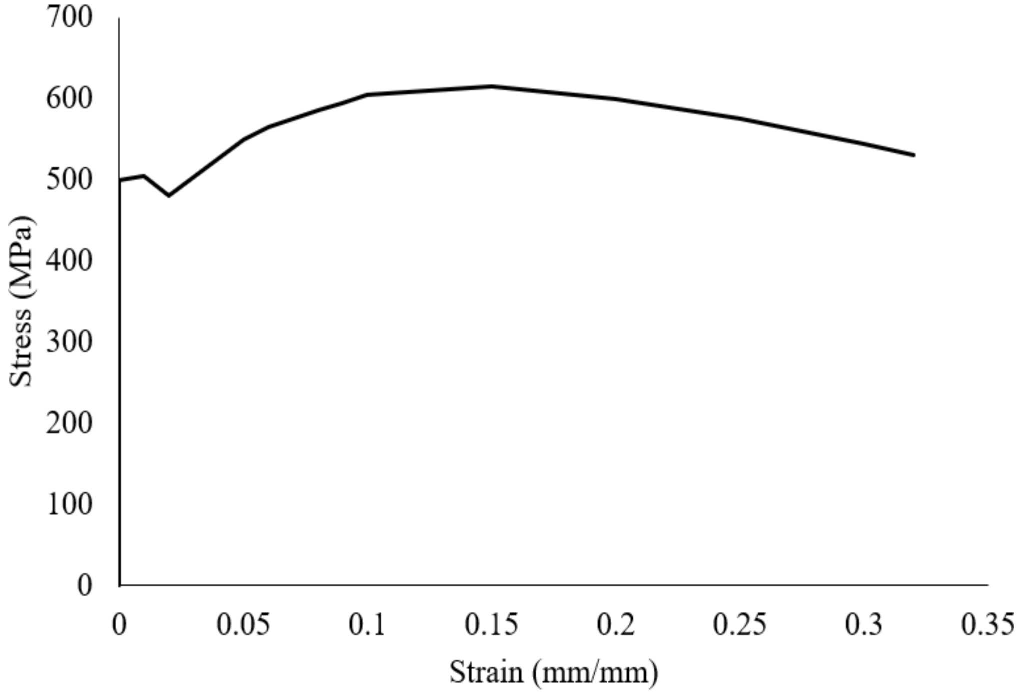
(a) Elevation



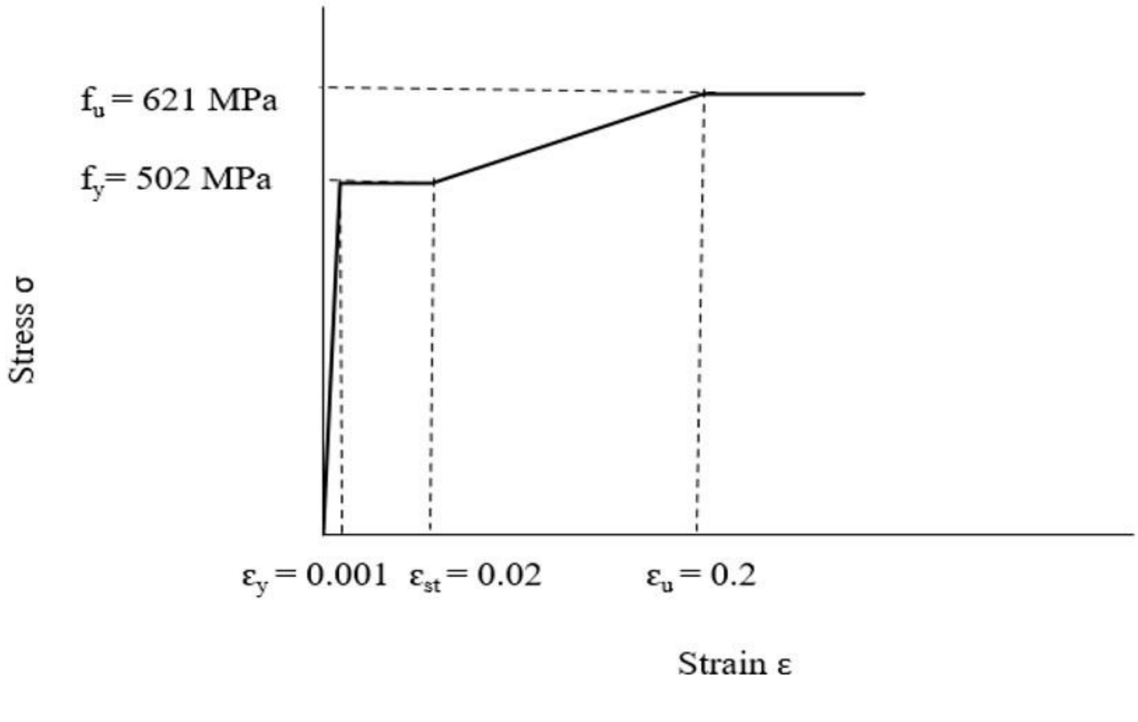


(b) Cross-section at mid one-third span (c) Cross-section at supports

Fig 1. Model of beam (DPLB) with dimensions12



(a)Stress-strain curve of steel from tensile coupon test



(b) Multi-linear material constitutive model for FE modelling

Fig 2. Stress-Strain curves of steel under tension

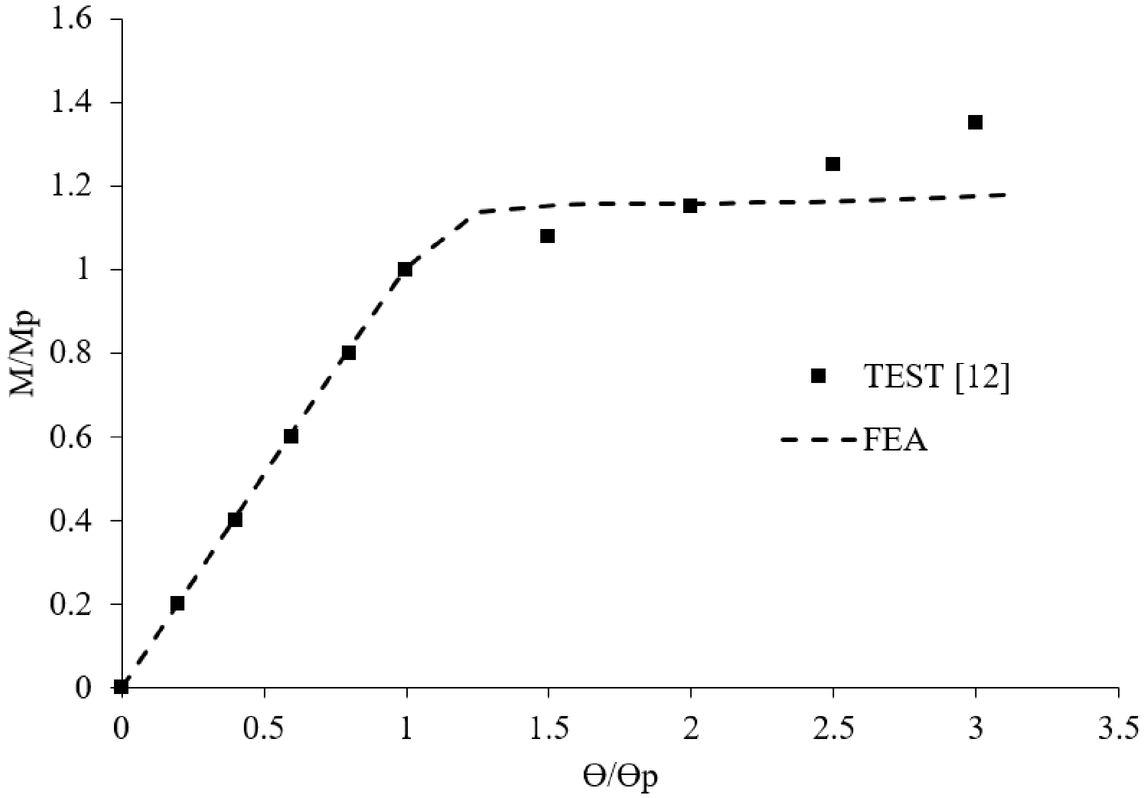


Fig 3. Moment vs Beam End Rotation Curves of longitudinally profiled flanged beam

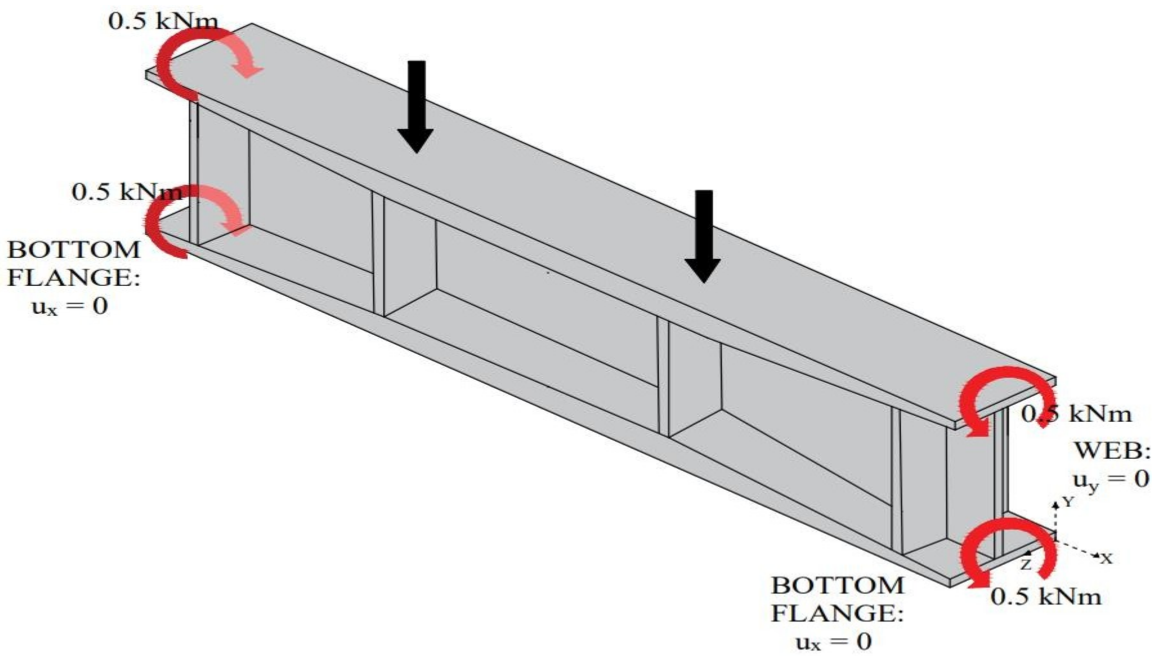


Fig 4. Loading and support conditions

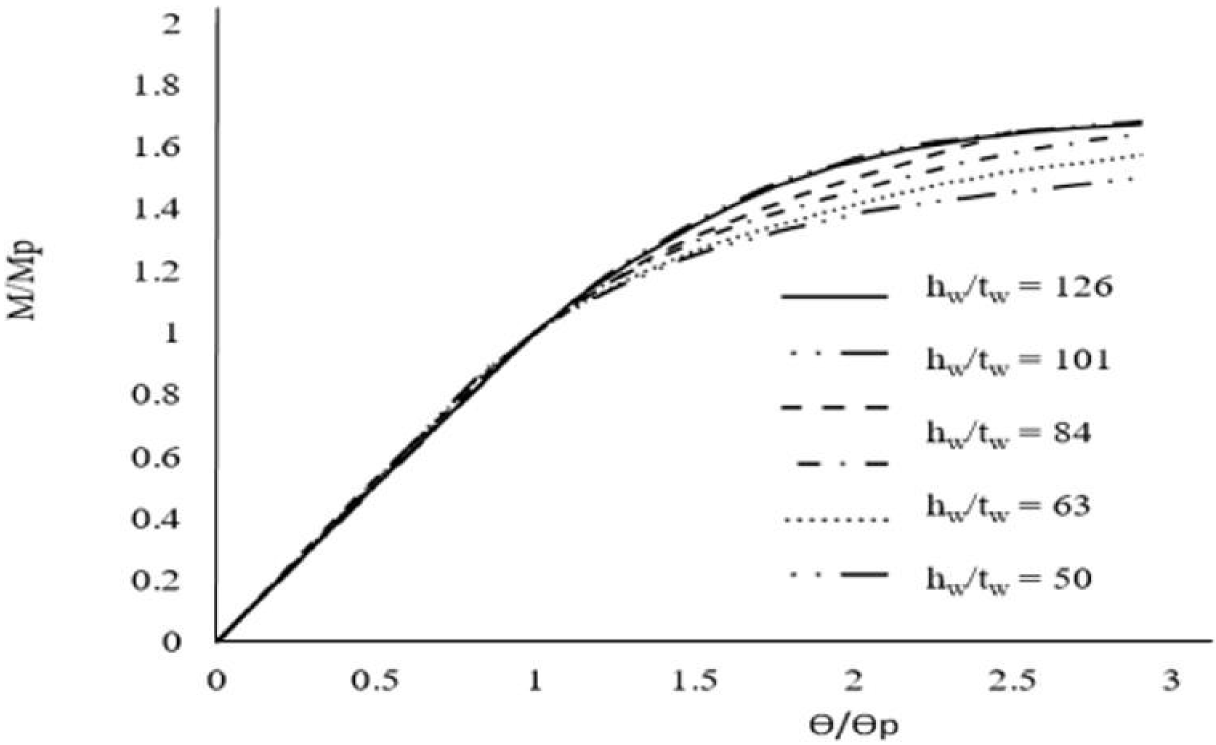


Fig 5. Effect of depth-to-thickness ratio of web

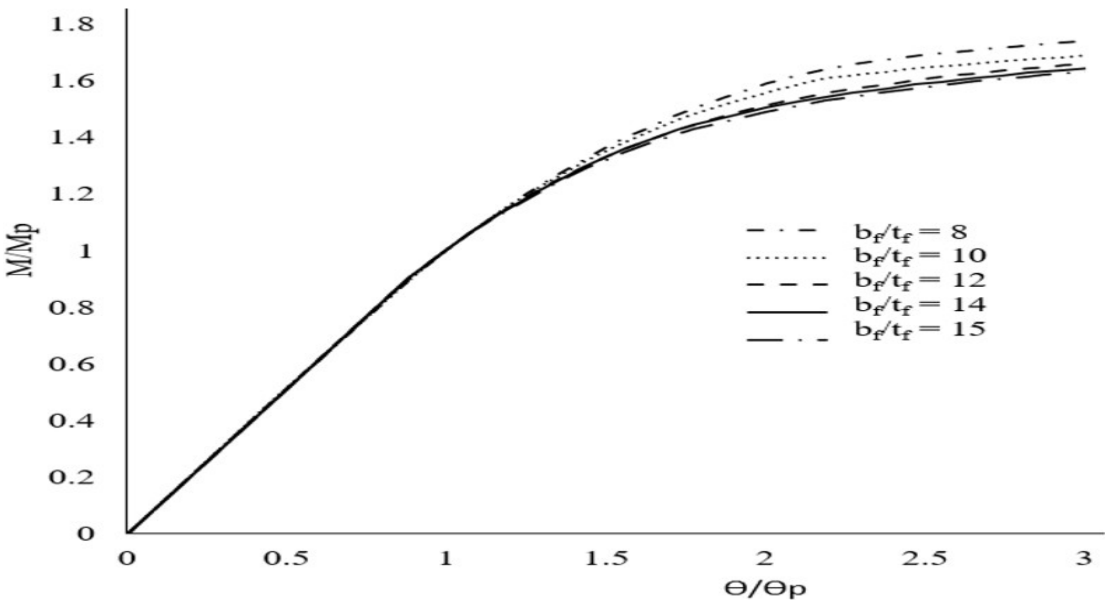


Fig 6. Effect of width-to-thickness ratio of flange

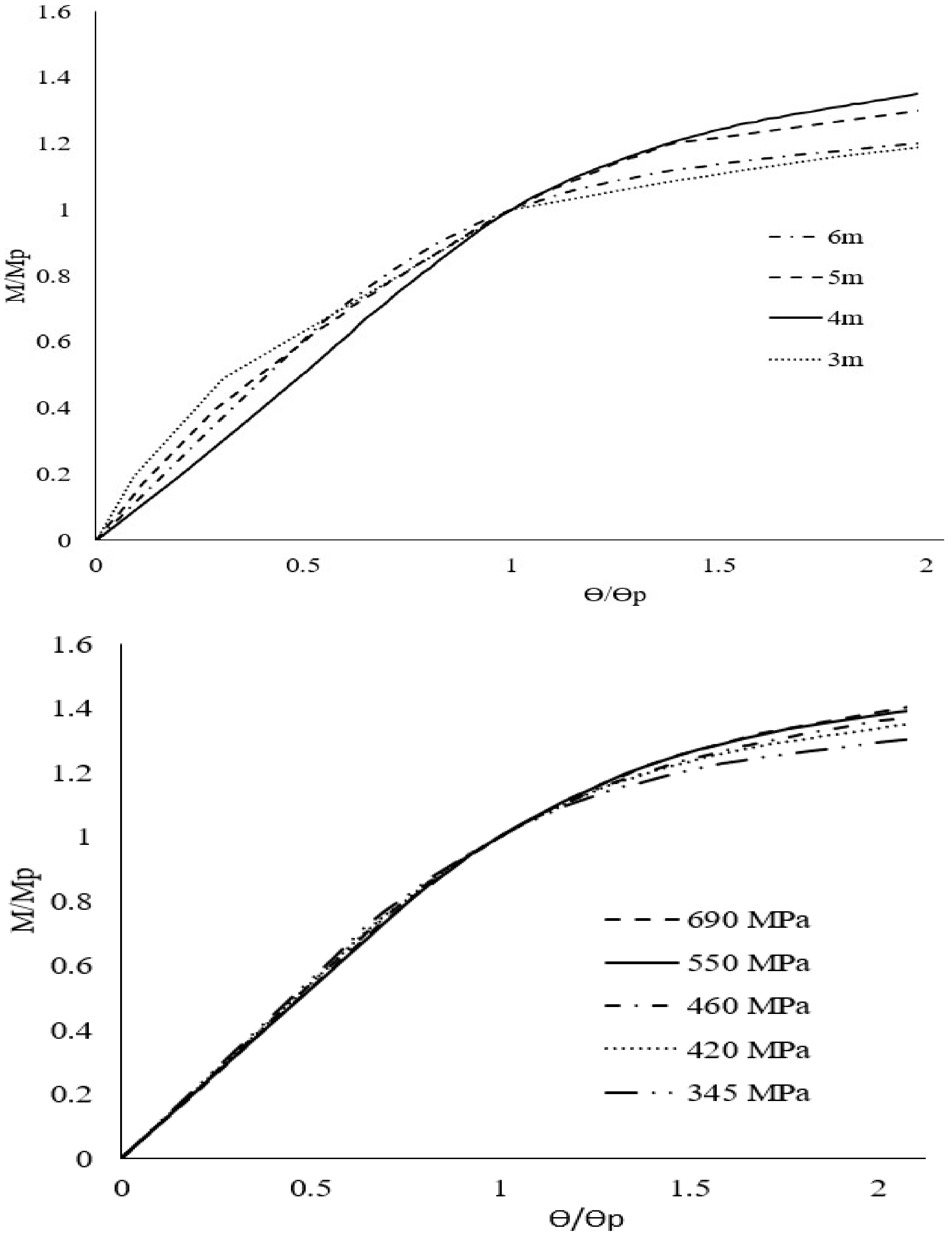


Fig 7.Effect of span

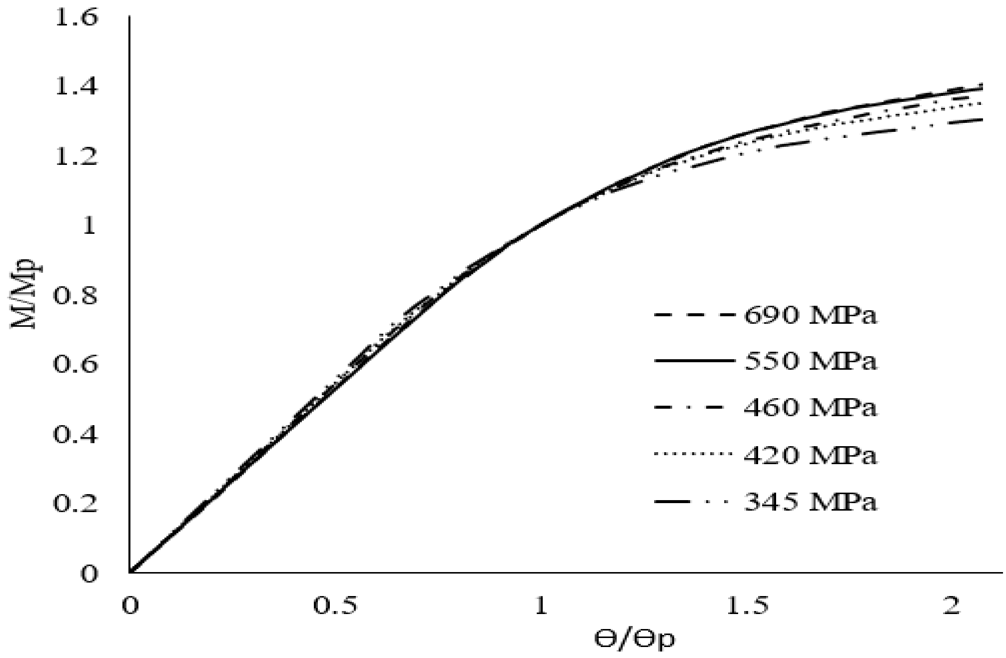


Fig8.Effect of steel grade

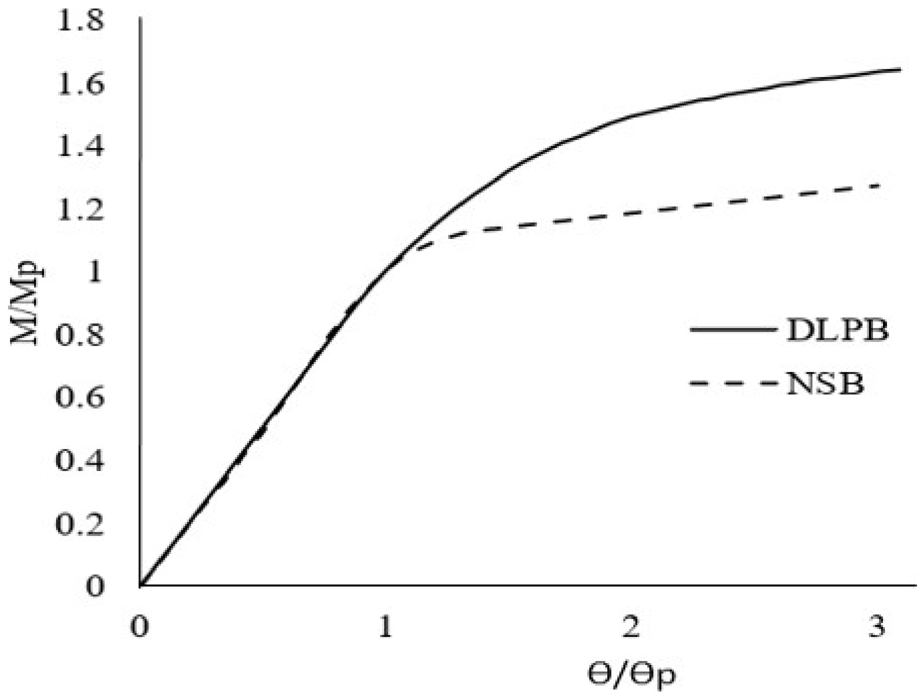


Fig9.Comparison of moment capacity for h_w/t_w=42

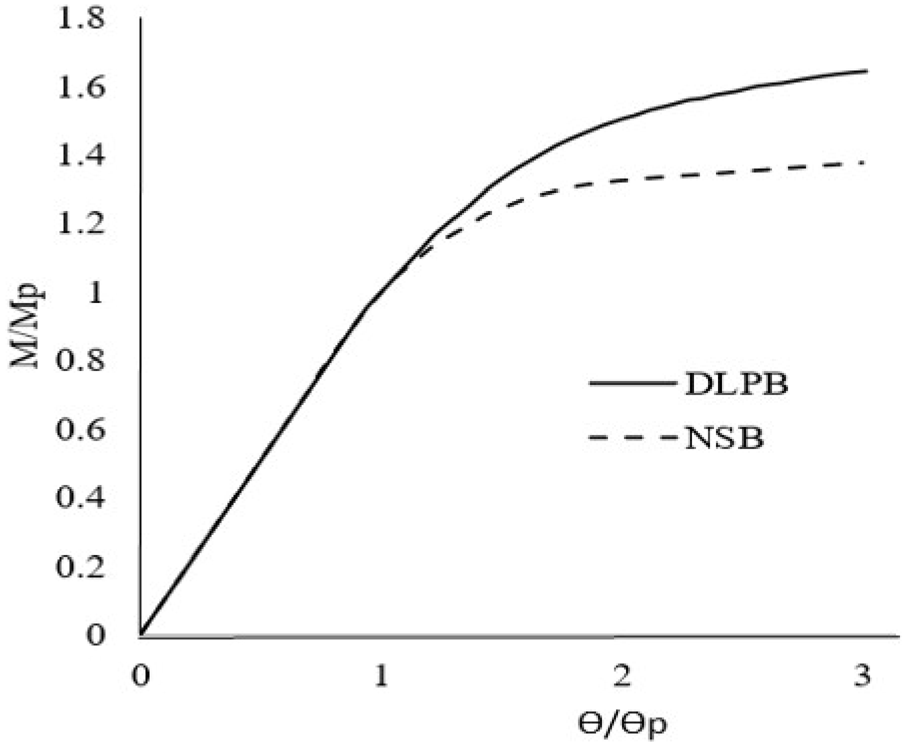


Fig10.Comparison of moment capacity for b_f/t_f=10

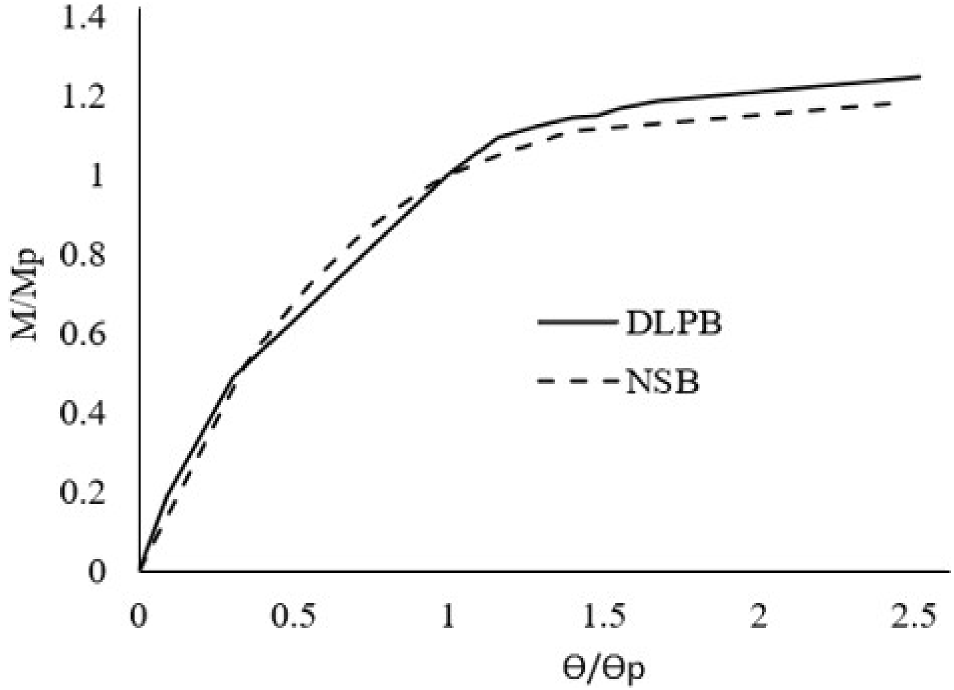


Fig11.Comparison of moment capacity for overall spanL_o=3m

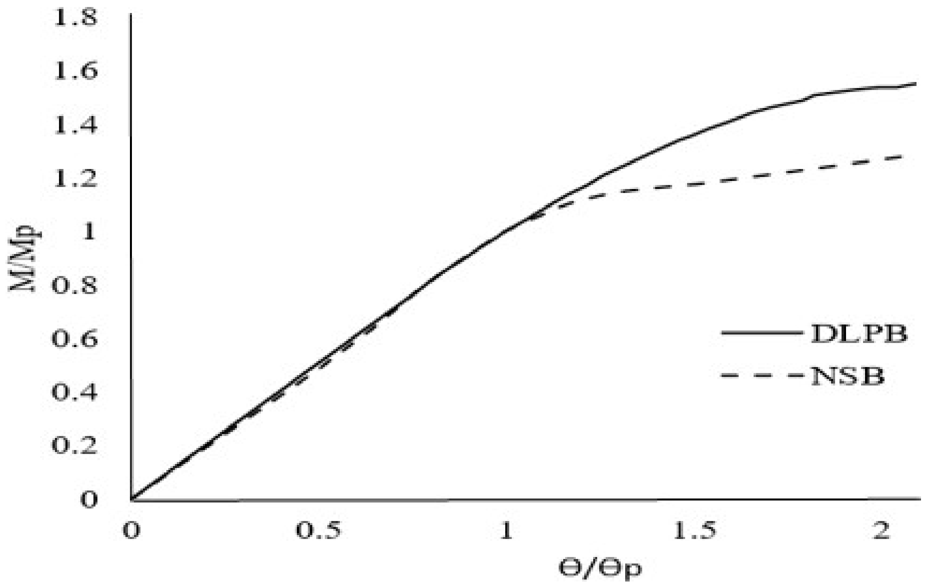


Fig11.Comparison of moment capacity for overall spanL_o=3m

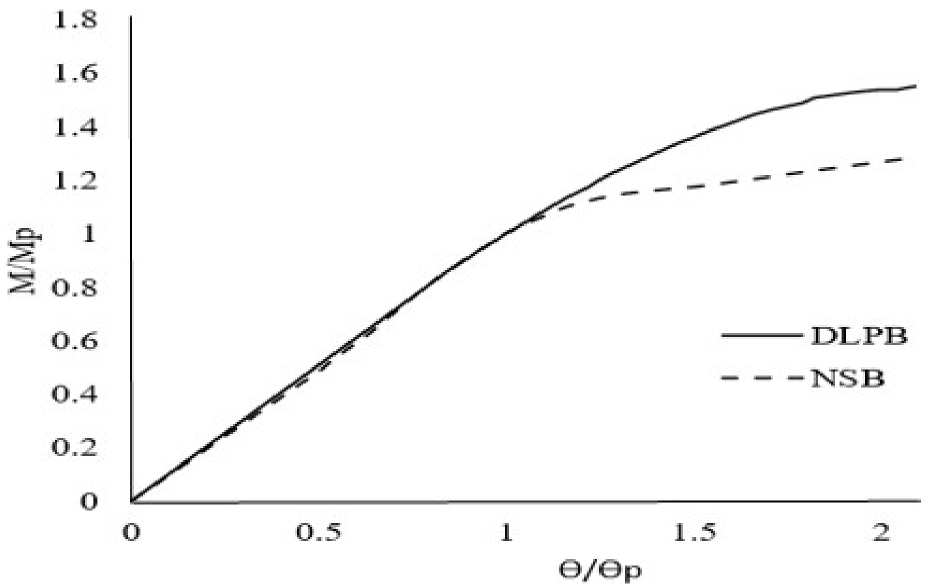


Fig12.Comparison of moment capacity for grade of steel 690MPa