CHAPTER 6

RESULTS AND DISCUSSIONS

6.1 INTRODUCTION

The cross section of the box girder may take the form of single-cell, multispine or multicell. The single and multicell box girders made of reinforced or prestressed concrete with vertical or inclined webs are preferred as economic and aesthetic solutions for over crossings, under crossings, viaducts, etc. The present trend in concrete box girders is to use thinner webs and flanges in order to reduce self-weight. Inclined webs in box girder bridges are used to reduce the width of the bottom flanges and provide a more efficient cross section. They also provide a more pleasing appearance to the bridge.

The typical box girder behaves like a beam, but its longitudinal flexural action is accompanied by transverse bending and is affected by distortion and warping of the cross section. To study the behaviour of box girders, laboratory testing of prestressed concrete box girders upto failure is very complicated and cumbersome. Also, it is not possible to study all the parameters experimentally. So, nonlinear finite element analysis becomes the only possible method of accurate analysis of prestressed concrete box girders through elastic, inelastic and ultimate load ranges. The ultimate strength and post-cracking load deformation behavior are very much influenced by many parameters. With the aid of the finite element modeling suggested in chapter 3, it is possible to study the behaviour of prestressed concrete box girders,
which leads to optimum design. The finite element model is developed using the composite shell elements and various constitutive laws for material nonlinearity.

Design for the collapse limit state requires consideration of crack patterns and modes of failure. Hence, in order to meet the need for more experimental work on scaled down models of box girders, to obtain information on the behaviour of cracked sections, the effect of cracking on stiffness, patterns of cracking, ultimate loads, effective widths of top flanges, effect of prestress etc., experimental investigations are conducted on four prestressed concrete box girders and the results are compared with the results obtained from the finite element analysis, using ABAQUS 6.5.

The new construction technology designed for construction of box girders is effective in achieving the desired dimensions of the slender box sections with a small deviation (less than 5%) without any distortion in the specimens. The ease in casting and accuracy in dimensions achieved confirms that the new technology can be effectively used in casting the concrete box girder models.

In the previous chapter, details of experimental investigation to study the behaviour of various types of prestressed concrete beams were presented. In this chapter, the results obtained from the analytical and experimental work are discussed. To assess the accuracy of the theoretical predictions, a series of comparisons are made between the analytical and experimental results. Four single cell reinforced concrete box girder models with end diaphragms and top flange width varying as 530 mm, 730 mm and 1080 mm are tested in the elastic range under non-uniform load and upto failure under symmetric load. These results are compared with the results obtained from finite element analysis. All the four girders are analyzed using
the Software ABAQUS 6.5. For validation of the finite element model explained in chapter 3, a few important aspects depicting the behavior of the box girders as obtained both from the experiments and analytical investigations are compared. These aspects are,

- Load versus deflection curves
- Span versus Deflection curves
- Failure load
- Stress plots

In addition to the above these aspects, the existing methods of practices are studied and suggestions are made for practicing engineers. The correlation of experimental and numerical data depends on the use of accurate linear and nonlinear material properties, as they are appropriate.

Some of the results of the study provide background for suggestions that are made for inclusion in the code of practice. Also, a set of modification factors are given to determine the peak top flange stress from the simplified method for preliminary designs. It is hoped that, this study will assist the design process and further physical understanding of the behaviour of prestressed concrete box girders.

### 6.2 STRUCTURAL RESPONSE OF BOX GIRDER BRIDGES

The structural response of box girder bridge consists of the primary actions viz., longitudinal bending, transverse bending, torsion causing distortion or deformation and warping of cross section. While, under self-weight and other symmetrical loadings, the section primarily experiences longitudinal and transverse bending, all of the above responses are present in case of asymmetrical loads, which is common in box girder bridges.
In the present study, the load versus deflection curves, deflected profiles along the span and longitudinal stresses of the prestressed concrete box girders are plotted.

6.3 LOAD-DEFLECTION CURVES

The deflection of the beam is a measure of the stiffness of the beam. Smaller the stiffness of the girder larger is the deflection. The deflection profile of the girder is a continuous curve, at each load stage, without any kinks in the profile. Hence for comparison of deflections, the deflection at any particular section along the span can be used. Mid span deflections are widely used by researchers and the same is followed in the present work. The load-deformation relationship can be used to realistically predict the behavior of structures. Load versus deflection curves for mid span deflection obtained both from the experiments and the analytical works are compared.

In any structural design, the two important criteria, which are of concern is the limit state of serviceability and the limit state of collapse. Upto the service load, the deformations play an important role while studying the behaviour of structures. Beyond service load, the limit state of collapse is considered.

In prestressed concrete structures, no tension is allowed in any part of the structure during the service life of the structure. Hence the zone of interest in the load versus deflection curve of the structure is generally upto the cracking load. The final failure load gives an indication of the overall strength of the structure. Between the cracking load and the failure load, the load-deformation characteristics, which are generally neglected in design calculation, have considerable significance. In the present study, the
deformation characteristics till failure are studied. The failure load is found for predicting the reserve strength.

6.3.1 Symmetric loading

For the box girders with 4.3 m effective span, the load versus deflection responses from the test program are plotted and compared with the finite element results. The girders are subjected to four symmetric point loads each placed on the top flange at points where web meets the flange, at a distance of 500 mm away from the mid span section of the beam on either side. The girder is tested till failure under this load.

Figures 6.1 to 6.4 shows the load versus deflection curves for the prestressed concrete box girders BG1, BG2, BG3 and BG4, subjected to symmetric loading. The experimental and analytical curves follow closely throughout the load range. The analytical curves indicate less deflection than the experimental value. The finite element solution gives less deflection than the experimental values i.e. the finite element analysis gives a stiffer beam.

In the girder BG1, shown in Figure 6.1, both the experimental and analytical curves agree very closely upto a load of 50 kN and then, there is some deviation afterwards. But still for a given deformation, the corresponding load agrees well within permissible limits. The numerical model predicts an ultimate load of 85 kN and captures well the nonlinear load deflection response of the girders upto failure. The ultimate load reached in the test is 80 kN. It is clear that the response of the model is linear until the first crack is formed at 45 kN. The difference between the analytical deflections and the experimental values are more pronounced beyond this point.
Figure 6.1 Load-Deflection curves for BG1 under symmetric load

Figure 6.2 shows the load-deflection curves for box girder BG2. In this case, the load deflection curves agree closely with the experimental results up to a load of 40 kN. The analytical method gives less deflection than experimental values.

Figure 6.3 shows the load-deflection curves for BG3, which has the experimental values very close to the analytical values up to 16 kN and the analytical values are greater than the experimental values. But, beyond this load, the deviation increased and similar to the previous two girders, this girder also shows lesser deflection analytically.

Figure 6.4 shows the load-deflection curves for the box girder BG4. The agreement between the two curves is fairly good up to about 40 kN load beyond which there is deviation. In this case also, the analytical values of deflection are less than the experimental values.
Figure 6.2  Load-Deflection curves for BG2 under symmetric load

Figure 6.3  Load-Deflection curves for BG3 under symmetric load
From Figure 6.5, it can be clearly seen that the nonlinear part of the curve is steeper in case of BG1 indicating maximum stiffness. As the load increases, the deflections exhibit a nonlinear growth as the overall stiffness of the structure decreases.
From Figure 6.6 it can be observed that the increase in prestress has reduced the deflections up to cracking and beyond which both the beams behaved similar to a reinforced concrete box girder. However, the prestressing force has increased the stiffness of the member.

![Experimental load-deflection curves for BG2 and BG4 under symmetric loading](image)

**Figure 6.6 Experimental load-deflection curves for BG2 and BG4 under symmetric loading**

Finally, it can be concluded that, the load-deflection curves follow very closely up to initial cracking of the girder, near about 40 kN load and beyond that there is slight deviation between the two curves. The variation is more in the third girder.

### 6.4 DEFLECTION PROFILES

The deflected profiles along the span of the box girders are plotted for both symmetric and asymmetric loads and presented in the following sections.
6.4.1 Symmetric loading

The deflection profiles under symmetric loading of various girders as obtained from ABAQUS6.5 are shown in Figures 6.7 to 6.10. The colour bands clearly indicate maximum deflection at mid span and the deflection decreases towards the end diaphragms. The presence of rigid diaphragms prevents excessive parabolic deflections at the ends, by imposing monolithic action of all the component members.

Figure 6.7 Vertical deflection profile of BG1 under symmetric load
Figure 6.8 Vertical deflection profile of BG2 under symmetric load

Figure 6.9 Vertical deflection profile of BG3 under symmetric load
From the Figure 6.11, it can be observed that the girder BG1 has maximum deflection while girder BG3 has minimum deflection. This is because of the increased stiffness of the girder with increased cross section dimensions. The experimental values are shown as points while the straight lines show the theoretically predicted deflection profiles.
From the Figure 6.12, it can be observed that the girder BG2 has more vertical deflection than BG4. This is because of the increased prestressing force in the girder BG4.

![Figure 6.12](image)

**Figure 6.12 Effect of prestressing force on vertical deflection under symmetric load**

### 6.4.2 Asymmetric loading

The girders are subjected to two pairs of unequal point loads each placed on the top flange, at a distance of 500 mm away from the center of the beam on the web flange junctions. The load is distributed in the proportion 0.78 W and 0.22 W on each of the web. The girder is tested within the elastic range under this load. The deflection profiles under asymmetric loading of various girders as obtained, from ABAQUS 6.5 are presented below (Figures 6.13 to 6.16). A total load of 40 kN is applied on the girder both experimentally and analytically and the results are presented below. The deflection profiles of the various girders along the length are presented in this section. The colour bands indicate the variation of deflection under the two webs when subjected to asymmetric loading. Both the webs are not deflected uniformly. The variation is more pronounced near the load points and decreased towards the ends indicating that the end diaphragms provided sufficient rigidity and the prevented distorsion of the cross section.
Figure 6.13  Vertical deflection profile of BG1 under asymmetric loading

Figure 6.14  Vertical deflection profile of BG2 under asymmetric loading
Figure 6.15  Vertical deflection profile of BG3 under asymmetric loading

Figure 6.16  Vertical Deflection profile of BG4 under asymmetric loading
From Figures 6.17 to 6.22, the curves reveal that the webs subjected to heavier load are under more vertical deflection than the web subjected to less load. This indicates that unlike rectangular sections, the box girder sections can resist asymmetric loads by distortion of cross sections. The torsion produced due to asymmetric loading is resisted primarily due to the torsional rigidity of box girders. In case of BG3, the variation of deflection between the two webs is small. This is possibly because of the increased stiffness of BG3. From Figures 6.21 and 6.22, it is clear that with increasing top flange width, the vertical deflections decrease as in the case of symmetric loading.

Figure 6.17 Effect of asymmetrical load on vertical deflection under both webs of BG1

Figure 6.18 Effect of asymmetrical load on vertical deflection under both webs of BG2
Figure 6.19 Effect of asymmetrical load on vertical deflection under both webs of BG3

Figure 6.20 Effect of asymmetrical load on vertical deflection under the both webs of BG4

Figure 6.21 Effect of asymmetrical load under heavily loaded web1
Under asymmetric loading, the vertical deflection decreases as the top flange width increases. When subjected to a total load of 40 kN, the percentage of decrease is about 16% for BG2 and 25% for BG3 under the heavily loaded web1 and 18% for BG2 and 18.5% for BG3 under lightly loaded web2, when compared with BG1. The deflection of less loaded web is about 5-20% of the loaded web showing better distribution of loads even without intermediate diaphragms. It is also observed that the box girder BG3, with widest flange experiences least deflection. This reflects the rigidity provided by the wider top flange. The decrease in vertical deflection is about 35% for both the webs. Vertical deflections along the length are plotted for different load cases for all the box girders. Comparing BG2 and BG4 in Figures 6.23 and 6.24, it can be concluded that the increase in prestress reduces the deflection up to the formation of crack and beyond cracking, both of them behave similarly.
From the above result analysis, it is seen that the proposed analytical model can be used for analysis of prestressed concrete box girders. The variations at higher load stages are due to the inherent variations in actual concrete stress-strain relations, the discretization effect in modeling the continuous medium by discrete finite elements and the assumptions made in deriving the constitutive relations. The variation in the tensile strength of concrete adopted also cause this deviation in the experimental and analytical values. The incremental measurements of strains and deflections were stopped beyond a load because of the practical difficulties. Whereas, analytically, the load deformation curve is obtained until large deformations occurred. With all
the limitations, which will remain until additional information are available in these areas, the method developed in this study can be used to analyze a post-tensioned prestressed concrete box girder. The analysis also shows that the behaviour of prestressed concrete girders is different up to cracking and beyond which both behave similarly.

6.5 STRESS PLOTS

The distribution of stresses in concrete across the depth of the girder under different loads is also verified using the analytical results obtained. The stresses obtained are plotted for the section at the mid span and quarter span of the girder. It was observed that the tension starts occurring at the bottom most fiber at about 40 kN load.

6.5.1 Symmetric loading

Under symmetric loading irrespective of the cross section, the position of neutral axis is horizontal and lie at the level of center of gravity of the section. This is true for all the four box girders and is verified from the longitudinal stress plots. Figure 6.25 shows the stress plot for BG2 obtained from ABAQUS6.5 and it shows that the shear lag effect is reduced away from the load in the longitudinal direction.
6.5.2 Asymmetric loading

However, tilt in neutral axis is observed in the plot of mean longitudinal stresses on a cross section for asymmetrically placed loads. The neutral axis rotates slightly about the centroid of the cross section of the box girder. This shift is because, on the loaded side, a comparatively larger compressive stress block emerges, which in turn requires lesser depth of the section in order to achieve the statistical equilibrium condition. The tilted position of neutral axis can also be attributed to the torsion resulting from eccentric loading. The typical stress and strain plots are shown in Figures 6.26 and 6.27.

Figure 6.25 Typical stress plot under symmetrical loading
Figure 6.26  Typical stress plot under asymmetric loading

Figure 6.27  Typical strains under asymmetric loading
6.6 SHEAR LAG

Shear lag arises when a box girder is subjected to bending with out torsion. A differential longitudinal deflection between points in the top flange occurs due to shear lag. This type of behavior results from shear deformation in the planes of the flanges and leads to a decrease in longitudinal bending stress away from the webs, which in turn, effects the effective widths of the flanges in bending, especially when there are excessive side cantilevers. This longitudinal differential displacement causes a non-uniform distribution of bending stresses, which is called shear lag. The complexity is increased by the effect of shear lag. Shear lag effects have been studied by previous investigators who mostly paid attention to theoretical analysis (Luo, Li and Tang, 2002). At the web and top/bottom deck slab junction, the longitudinal bending stresses got enhanced due to this shear lag effect as shown in Figure 6.28 in which, the longitudinal bending stresses obtained from the measured strains are plotted for all the three box girders. The bending stresses near the web-flange junction are much larger than the stresses at locations away from the web, which is called as positive shear lag. It is not possible to achieve the shear lag effect with the help of simple beam theory, which yields only uniform stress distribution across the flange. It can also be observed from the stress diagrams, that the variation of stress across the depth is almost linear.
Figure 6.28 Longitudinal bending stress (N/mm$^2$) at mid span
   a) Symmetric Loading  b) Asymmetric loading
Considering Figures 6.28 and 6.29, it can be observed that, for wider flanges, the effect of shear lag is more prominent. This could be because the compression or tension effect of the edge shear does not flow very far from the loaded edge. Hence, much of the wide flange overhang remains ineffective. Peak flange stresses are found to be 30% greater than the values given by the simplified method. For accurate values, a nonlinear finite element analysis is to be adopted. The analytical results are compared with the experimental results and are found to match well with a variation of about 5-10%.

It can be appreciated that, the magnitude of stresses are lesser in case of box girder with widest flange as expected. Neglect of the shear lag effect can lead to an under estimation of stresses. For a load of 40 kN, the stress at web flange junction is found to be 15 % greater than that on the edge of the flange for BG2 and BG3. For box girder of type BG3, the corresponding increase is 23 %. Hence, during the design, the effective width of the compression flange has to be taken.

For box girder BG2 as shown in Figure 6.28, decrease in values of longitudinal stress are 18.21% at overhangs, 32.89% at center line of top deck, 20.44% on soffit slabs and 11.65% for webs compared to simplified method.

The non-uniform distribution of longitudinal displacements could be because, unlike rectangular section, in plated structures, webs and flanges are inter connected, resulting in frame action, so that relative displacement cannot occur between them. Hence, in case of such sections, the assumption that plane sections remain plane before and after bending may not be justifiable.
Figure 6.29  Longitudinal bending stress (N/mm$^2$) at quarter span

(a) Symmetric Loading  b) Asymmetric Loading

The longitudinal displacements are considerably affected by warping of girder and warping is induced by shear strains. So, it can be anticipated that, a reduction of shear stress can lead to a non-uniform distribution of longitudinal stresses in flanges.
The effect of positive shear lag is clearly exhibited by the Finite Element Analysis, which cannot be obtained by the approximate method. For box girders with wide flanges, the effect of shear lag is very much pronounced. This could be because, the plane sections in box girders with wide flanges are not always plane, owing to the action of in-plane shear strain in flanges. The longitudinal displacements in parts of the flanges remote from the webs lag behind those nearer to the webs. This results in the deflections and longitudinal stresses at the web flange intersection of the girder being greater.

### 6.7 FAILURE LOAD

The failure loads of various prestressed concrete box girders obtained from the experiments and the analysis by the nonlinear finite element analysis are given in Table 6.1 and Figure 6.30.

**Table 6.1 Failure load of box girders**

<table>
<thead>
<tr>
<th>S.No</th>
<th>Box Girder</th>
<th>Experimental Value (kN)</th>
<th>Analytical Value (kN)</th>
<th>% Variation</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>BG1</td>
<td>85</td>
<td>78</td>
<td>8.21</td>
</tr>
<tr>
<td>2</td>
<td>BG2</td>
<td>100</td>
<td>96</td>
<td>4.00</td>
</tr>
<tr>
<td>3</td>
<td>BG3</td>
<td>115</td>
<td>106</td>
<td>7.81</td>
</tr>
<tr>
<td>4</td>
<td>BG4</td>
<td>105</td>
<td>98</td>
<td>6.67</td>
</tr>
</tbody>
</table>

It can be seen that the failure load predicted by the analytical method is in close agreement with the experimental results. The percentage variation between the two varies from 4.00% to 8.21%, which is considered
reasonable, with the present level of development of constitutive relations of concrete and steel and the plane stress approximation made in this study.

![Bar Chart](chart.png)

**Figure 6.30** Failure loads

### 6.8 CRACKING

The experimental crack distribution indicates that, in the post-cracking stage, the beams are characterized by small cracks, well distributed in the zone of maximum moment. The first flexural crack initiated at the bottom of the web and extended vertically upwards, which, widened with further application of the load. The cracks become less uniform as they reach the compression face. Details of crack patterns in all the three girders are shown in Figures 6.31 to 6.34. The cracks extended into the top flange
Figure 6.31 Crack pattern for BG1

Figure 6.32 Crack pattern for BG2
Figure 6.33 Crack pattern for BG3

Figure 6.34 Crack pattern for BG4
in case of BG1. However, in case of the other three specimens, the cracks were denser in the webs and in the bottom flange. The top flanges showed no cracks. In the middle third potion of the girder, a set of transverse cracks formed in the bottom slab perpendicular to the longitudinal axis. These cracks extended over the middle third of the girder. The discrete nature of flexural cracks could not be captured using a smeared cracked model. As the yield strength of the post tensioning cables and the internal reinforcement is reached the neutral axis shifted upwards and eventually the yield strength is exceeded resulting in cracking.

6.9 EFFECT OF PRESTRESS ON DEFLECTION AND LONGITUDINAL STRESSES

The girders BG2 and BG4 are used in the experimental and analytical investigations to study the effect of prestress. Theoretical studies are also conducted using the finite element model suggested in chapter 3 by varying the loading conditions. The details like cable profile, strength of materials is kept the same as that of the experimental beam. The girders are analyzed and mid span deflection is used as a measure and is compared for different girders.

As a first case, the girder is analyzed for an un prestressed girder but with cable and the results are compared with the prestressed girders. The beam without prestress is analyzed for gravity load. Using symmetry, only half of the section and half of the span is analyzed. The girders are also analyzed for prestress of 700 N/mm² and 1000 N/mm². The comparison of results is shown in Figures 6.35 and 6.36 qualitatively. In these figures, the ordinate refers to the mid span deflection of the beam and the abscissa represents the span.
In order to study the effect of cable stiffness, the girder is analyzed with cable but without prestressing tension. Results in this case show that there is a reduction of about 10% in the results, due to addition of cable stiffness.

The effect of prestressing on the deflection of box girder bridge is quite appreciable as illustrated in the Figure 6.35 below. The dead load deflection is balanced by the prestressing force and the mid span deflection due to combined action of dead load, Live load and Prestress is reduced by 40 to 50%. From Figure 6.36, it can be observed that stiffness of the girder increases due to the presence of cables, thus indicating the necessity of including cable stiffness in the stiffness of the elements.

Figure 6.35  Effect of prestressing
6.10  FLEXURAL STRENGTH

The flexural strength of the members is estimated using the design equations suggested in IS:1343 and when compared with the experimental results, it is observed that the Indian code recommendations are conservative estimates of flexural strength and also no separate expressions are available for over reinforced and under reinforced sections.

The results obtained from ABAQUS 6.5 software are compared with the experimental results and also with the results obtained from IS:1343 codal provisions in Figure 6.37. The flexural strengths estimated by the IS:1343 are found to be 10% to 20% less than the experimental values while the finite element analysis results are 5% to 15% less than the experimental values.
Presently, IS:1343 is the code available in India for design of prestressed components. The shear lag effect causes stresses significantly larger than those predicted by simple beam theories. The design equations suggested in the code IS:1343 do not accommodate for these variations in stresses.

### 6.12 DESIGN PROCEDURES

In design practice, the longitudinal action and transverse action are often analyzed separately. The box-girder bridge is modeled as a beam for
longitudinal action and as a frame of unit width and simple frame analysis is carried to obtain the transverse bending moments. But, in thin-walled box sections warping stresses are developed due to torsion and distortion. To account for the error arising out of the neglect of this warping effect, the results of simplified analysis are sometimes enhanced by some percentage (about 10%). In addition, most of the methods commonly used are limited to homogeneous and linearly elastic members. Also, these methods are developed for single cell sections but are applied to multi-cell sections. Three-dimensional finite element analysis provides an alternative computational method, which addresses both transverse and longitudinal actions integrally.

Because of the warping restraint at the ends, the experimental values for the sections are significantly higher than those calculated from the simple beam theory. In simple beam theory of bending, it is assumed that the cross section of a girder which were plane before bending remain plane after bending. However, for a girder with wide flanges, this assumption is not justifiable. Owing to the action of in-plane shear strain in the flanges, the longitudinal displacements in the parts of the flanges remote from the webs lag behind those nearer the webs. This phenomenon called as shear lag results in deflections and longitudinal stresses at the web flange intersections of a girder being greater than those given by the elementary theory of bending. Peak flange stresses are found to be 30% greater than the values given by the simplified method. It has been observed that the effect of shear lag has quite significant implications as the on the bending stresses which should be considered at design stage itself.

For design purposes, while calculating the deflection or stresses of a wide flange girder, the actual breadth of each flange is to be replaced by a reduced breadth, termed as effective breadth, such that the application of the
elementary theory of bending to the transformed girder cross-section gives the values of maximum deflection and longitudinal stresses.

Defining shear lag factor as the ratio of peak flange stress to the simplified method stresses, the design charts for extracting the ratios for various flange widths are given in Figure 6.38 under symmetric load. The design chart for maximum deflections varying with the cantilever top flange width is given in Figure 6.39 under symmetric load.

![Figure 6.38](image)

**Figure 6.38** Ratio of Width of side cantilever/ top flange between webs \((b_c/b_t)\) vs shear lag factor
Figure 6.39  Ratio of Width of side cantilever/ top flange between webs (b_c/b_t) vs Ratio of deflection w.r.t box girder with out cantilever top flange

6.13 MODIFICATION FACTORS

Modification factors are developed to predict peak flange stresses under symmetric load. The longitudinal bending stress is affected by the \( \frac{b_c}{b_t} \) ratio i.e the ratio of cantilever flange to the width of the top flange between the webs. A set of modification factors have been developed based on the \( \frac{b_c}{b_t} \) ratio under symmetric load. Based on the extensive experimental and analytical studies carried out, for the benefit of practicing engineers, a set of simple modification factors are proposed in the Table 6.2 to overcome the conservative designs produced by the simple theory of bending. The proposed modification factors, account for the influence of the side cantilevers on the longitudinal bending stress. These modification factors can be applied directly to the stresses obtained from the simplified methods.

Peak top flange stress  =  Modification Factor x Stress from simple beam approach
Table 6.2 Modification Factors for maximum stress at mid span at the top flange-web junction under symmetric loading

<table>
<thead>
<tr>
<th>(x/L) (\downarrow)</th>
<th>(b_c/b_l) (\rightarrow)</th>
<th>(b_c/x)</th>
<th>Modification Factor</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>0.1</td>
<td>0.2</td>
<td>0.3</td>
</tr>
<tr>
<td>0.5</td>
<td>0.0233</td>
<td>0.0465</td>
<td>0.0814</td>
</tr>
<tr>
<td>0.4</td>
<td>0.0291</td>
<td>0.0581</td>
<td>0.1017</td>
</tr>
<tr>
<td>0.35</td>
<td>0.0333</td>
<td>0.0667</td>
<td>0.1167</td>
</tr>
<tr>
<td>0.33</td>
<td>0.0349</td>
<td>0.0698</td>
<td>0.1221</td>
</tr>
<tr>
<td>0.32</td>
<td>0.0357</td>
<td>0.0714</td>
<td>0.125</td>
</tr>
<tr>
<td>0.3</td>
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<td>0.26</td>
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</tr>
<tr>
<td>0.25</td>
<td>0.0465</td>
<td>0.0930</td>
<td>0.1628</td>
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<tr>
<td>0.15</td>
<td>0.0833</td>
<td>0.1667</td>
<td>0.29167</td>
</tr>
</tbody>
</table>
Example 1

The problem presented by Jirousek, Bouberguiq and Saygun (1979), is considered but slightly modified as shown in Figure 6.40. The bridge has prestressing cables having parabolic profile and each web carrying one cable. Using symmetry, only one half of the cross section and half of span are analyzed. The bridge is analyzed for a cable tension of 28 kN, with out any other load.

![Figure 6.40 Cross section](image)

For the cross section of the box girder shown in the figure, the stress obtained by the simple beam theory,

\[
\begin{align*}
\text{The distance of the section from the end, } x & = 12.50 \text{ m} \\
\text{Total span, } L & = 37.50 \text{ m} \\
x/L & = 0.33 \\
\text{Peak bending stress in the top flange(Jirousek,1979)} & = 68.1 \times 10^{-2} \text{ N/mm}^2 \\
\text{Cantilever width} & = 1850 \text{ mm} \\
\text{Width of side cantilever/ Width of top flange between the two webs, } b_c/b_t & = 0.487 \\
\text{Modification Factor (from table 6.2)} & = 1.13
\end{align*}
\]
Peak top flange stress  =  Stress based on simple beam approach x Modification factor  =  $64.2 \times 10^{-2}$ N/mm$^2$

% variation between the stress obtained from finite element analysis and the stress obtained using modification factors is 5.7%.

**Example 2**

Considering BG2,

The distance of the section from the end, $x$  =  1.65 m

Total span, $L$  =  4.3 m

$x/L$  =  0.5

Peak bending stress in the top flange (FEM)  =  4.57 N/mm$^2$ Cantilever width  =  100 mm

Width of side cantilever/ Width of top flange between the two webs, $bc/btw$  =  0.19

Modification Factor  =  1.26

Peak top flange stress  =  Stress based on simple beam approach x Modification factor

=  3.43 x 1.26

=  4.32 N/mm$^2$

% variation between the stress obtained from finite element analysis and the stress obtained using modification factors is 5.47%.