COMPOSITE BOX GIRDER BRIDGES

1.0 INTRODUCTION

Composite steel - concrete box girder bridges have been widely used for medium span (45 m to 100 m) bridge structures. Composite box girders offer an attractive form of construction. They are aesthetically pleasing and are composites of steel box girders and concrete deck slab (Fig. 1). Apart from their pleasing forms, they have other economic advantages over the familiar I-beam and plate girder structures. A composite box girder section has high torsional stiffness, which results in a greater lateral distribution of the live load. Another advantage is the elimination of most of the wind and transverse bracings. The advantages of using composite box girders are given below.

- Economic span length longer
- Girder section can be erected by mobile crane
- Tortional performance reduce bearing requirement
- Due to tortionally stiff section, intermediate bracing can be avoided
- Improved resistance to aerodynamic excitation
- Smooth lines on the sides and below
- Clean surfaces
- Use of sections curved in plan
- Sloping webs

Painting surface is much less. Clear surfaces of boxes lead to fewer corrosion traps. Access for internal inspection and maintenance is an important consideration in design.

A variety of cross-sectional geometries for the completed box structure is possible. The number of girders may vary from two to six or more depending upon the plan geometry of the structure. Typical cross sectional geometries are shown in Fig. 2. Support bearing arrangements can be both single and double. The end support diaphragms can be deep or shallow (see Fig. 3)

2.0 DESIGN PREMISE

A composite box girder highway bridge is generally made up of reinforced concrete deck slab on top of one or more fabricated steel open box girders. Normally two boxes are used for carrying minor roads. For wider roads, multiple boxes, four or more, may be needed. Alternatively, for wider roads, twin box sections with cross-girders may be used. Here the slab spans longitudinally rather than transversely between the lines of the box webs. This configuration is used for spans greater than 100 m. For narrow roads, single box section with the slab haunched over the web lines could serve the purpose. For longer spans, wide single boxes with cross beams and cantilevers may be appropriate.

In practice, two different classes of composite box girders are considered. In one class steel boxes are completely closed, and the other is in the form of an open 'U' section. For both the classes, the box section could be either rectangular or trapezoidal. The bottom flange width is narrower compared to the top flange. In elevation box section may have a constant depth or they may be haunched. The soffit portion is usually curved for better appearance. In plan, the box girders may be curved in order to suit the highway layout. Due to their high torsional properties, they are well suited where truly curved girders are required.

3.0 BEHAVIOUR OF BOX GIRDER BRIDGES

The most important feature of the box girder bridge is its large torsional stiffness by virtue of the closed section. This has considerable effect on the global bending behaviour of the system by way of sharing of the vertical shear in a more equitable manner between the web planes. This further results in the even sharing of bending stress by the bottom flange plate. Thus, box girders behave more efficiently compared to I - beam structures. Due to the even sharing of the load, there is less need to design for peak load effects, which might occur at any time on only one plate girder.

The choice of a box girder has its own problems. The use of wide thin panels for web and flange sections is clearly less efficient than the more stocky I or plate girder section. When more webs are introduced, the thinner web panels would need stiffening arrangement. In spite of this, their limiting shear stress would have a lower value and they may be less effective in bending. Compression flanges of the box sections may also be not fully effective due to local buckling considerations. It may be noted here that plate girder flanges are usually fully effective for the reason that compact sections are chosen for plate girders. Panel slenderness effect which causes reduction in effective section must also be considered in choosing a configuration.

Other notable effects that must be considered in the use of box girders are due to distortion and warping. These effects can be minimised by proper internal stiffening and proportioning of the cross-section. Another important effect is the shear lag in the case of wide flanges.

3.1 Bending, torsion and distortion

In general, an eccentric load applied to a box girder is a combination of three components-bending, torsion and distortion. As shown in Fig. 4, an externally applied eccentric force can be considered as a combination of bending and torsion components.

The externally applied forces would be resisted by supports or bearings. The torsion is resisted by the box section by a shear flow around the perimeter as shown in Fig. 5. The torsional couple on the box is separated into two parts: pure torsion and distortion. The distortion component consists of a set of internal forces, which are in static equilibrium. The effects depend on the behaviour of the structure between the point of application of the load and the nearest point of support where the section is restrained against rotation.

The box girders are provided with bearings at supports. The number of bearings could be one or two. If two bearings are provided, they are provided either directly under each web or inside the line of the webs. In order to resist the reaction forces on the bearings, the webs will have to be provided with bearing support stiffeners. Further, to resist the distortional effects caused due to transmitting torsion from the box to the bearing supports, diaphragms will also be required. If only a single bearing is provided, a stiffened diaphragm will be required to resist the reaction as well as to distribute the force to the webs. Between the points of support, intermediate transverse web stiffeners may be provided to develop sufficient shear resistance in a thin web. Provision of intermediate diaphragms would minimize distortional effects caused due to eccentrically applied loads.

3.2 Torsion and torsional warping

As mentioned previously, box girders are subjected to torsion. For a single box, the torque is resisted by shear flow (force/unit length) which acts around the walls of the box. This shear flow is constant around the box and is given by $q=T/2A$, where T is the torque and A is the area enclosed by the box. The shear flow induces shear stresses and strains in the walls of the box and causes a twist per unit length, θ, given by

$$\theta = \frac{T}{GJ}$$

Where, J = torsion constant

However, pure torsion of a thin walled section will also produce warping of the cross section if the section does not have sufficient symmetry.

For example, consider a rectangular box section of width and depth B and D respectively and flange and web thickness, t_f and t_w, subjected to torsion. The shear flow under a torque T is given as q = T/2BD.

The shear stress in the flange (Fig.6) is given by $\tau_f = q/t_f = T/2BDt_f$.

If viewed from top, it can be seen that each flange is sheared into a parallelogram, with a shear angle, $\phi = \tau_f/G$.

Assuming the end section to remain plane, the relative horizontal displacement between top and bottom corners would be ϕL at each end (Fig.6) and there would be a twist between the two ends of the box , the twisting angle over the length L due to tension is given by,

$$2\phi L/D = \frac{2\tau_f L}{DG} = \frac{TL}{BD^2Gt_f}$$

Similarly, considering the shear displacements of the webs, if the end sections are held to remain plane, the twist of the section would be TL / B^2DGt_w. From this, it is clear that if the end sections are to remain plane, $TL / BD^2 GT_f = TL / B^2DGt_w$ i.e., $Dt_f = Bt_w$. This is the condition for the end sections to remain plane. If the condition is not met, there will be a slight counter rotation in the planes of the two flanges and the webs and a resulting warping of the section (Fig.7)

For a box supported and torsionally restrained at both ends and subjected to applied torque in the middle, warping is fully restrained in the middle by virtue of symmetry and torsional warping stresses are generated. For continuous box sections, intermediate supports are torsionally restrained. The warping restraint introduces longitudinal warping stresses and associated shear stresses, just like the effects of bending in each wall of the box. The shear stresses modify the uniformity of shear stress calculated by pure torsion theory, reducing the stress near the corners and increasing in mid-panel. Since the combined effects occur at the corners, the warping shear stresses are ignored and a uniform distribution is used. The longitudinal stresses are maximum at the corners. They should be accounted for when considering the occurrence of yield stresses in service and the stress range under fatigue loading. However, there is no reduction in torsional resistance due to yielding as the longitudinal stresses do not contribute in carrying torsion. In other words, a little yielding is acceptable at the Ultimate Limit State (ULS) and torsional warping stresses need not be considered in ULS checks.

3.3 Distortion

When torsion is applied directly around the perimeter of a box section by forces equal to the shear flow in each side, the cross section does not change its shape. In case torsion is not applied in this manner, provision of a diaphragm or stiff frame would ensure that the section remains square and the torque is applied as a shear flow around the perimeter. The diaphragm or frame is then subjected to distortional forces (Fig. 5). In practice, such diaphragms and frames can be provided only at discrete locations such as supports and points of heavy load application. In the case of a beam where point load can occur anywhere along the length, the distortional effects must be carried by other means. The occurrence of distortion and the manner in which it is carried between effective restraint can be explained using a simply supported box with diaphragm only at the supports and subjected to a point load over one web at mid span. In the absence of any transverse moment continuity at the corners, the cross section will distort (Fig.8). Each side of the box will bend in its own plane and since the sides remain connected along the common edges, the cross section changes shape as shown in the figure.

The in-plane bending on each side produces longitudinal stresses and strains; because they are in the opposite sense in the opposing faces of the box, they give rise to warping of the cross-section. The longitudinal stresses are known as *distortional warping stresses* and the associated shear stresses are called *distortional shear stresses*. If a flexible intermediate cross-frame is provided in the above example at the point of application of the load, it resists the distortion by sway bending (Fig. 9). A stiffer frame would definitely undergo less distortion

If there is no intermediate cross-frame and there is transverse moment continuity at the corners, the box walls are subject to the same sway deflection (Fig. 9), but the bending now takes place in the walls of the box. The bending of the cross-frames and the walls of the box, due to distortional forces produce transverse distortional bending stresses in the box section. The distortional behaviour depends on the interaction between two sorts of behaviour: warping and transverse distortional bending. This behaviour has been found to be analogous to that of *Beam on an Elastic Foundation* (BEF). Here, the beam stiffness represent the warping resistance and the elastic foundation represents the transverse distortional bending resistance. The BEF model is the basis for the rules in Appendix B of BS 5400 part 3.

When a concentrated load is applied eccentrically to a box section, the distortional effects are maximum at the vicinity of the point of load application. The decay of the distortional effects away from the point of application can be studied by considering the BEF analogy for a load in the middle of a simply supported box girder with diaphragms only at the supports. This is depicted in Fig. 10. Warping stresses correspond to bending of the beam and distortional bending stresses to the displacement of the foundation. The rate of decrease depends on the relative magnitudes of longitudinal bending and warping resistances and on the length of the beam.

The provision of intermediate diaphragms in the box girder can be represented in the BEF analogy by the addition of discrete springs. The intermediate restraints are considered as flexible springs and the BEF model can be analyzed. A modified response using flexible restraints is shown in Fig. 11.

3.4 Stiffened compression flanges

The wide flanges used in box girders are vulnerable to buckling under compression and they may need to be stiffened to carry the required load. To increase the load resistance of long compression panels, longitudinal stiffeners are introduced which restrict the width of the individual panels.

These stiffeners carry load in compression and they in turn need to be restrained against buckling out of the plane of panel. The restraint is provided by transverse stiffeners, cross beams or diaphragms. In very wide and long flanges with longitudinal and transverse stiffeners, buckling of the whole stiffened panel needs to be considered. Simple rules have been derived to express the strength of a plate panel in terms of an effective width parameter. In a longitudinally stiffened flange, half the effective width of each plate panel is considered to be attached to the stiffeners to form effective struts between out-of-plane restraints. The strength of the flange is the sum of the strengths of the effective struts.

3.5 Shear lag

In a cantilever box section with web and flange panels subjected to free-end loads, the elementary bending theory will give a tensile bending stress in the flange panel uniform across at any cross section. Actually, however, the panel acquires its tensile stresses from shear stresses on its edges transferred to it by the webs. The distribution of tensile stress across the width will not be uniform (Fig. 12); but higher at the edges than at the middle. This departure from the uniformity assumed by the elementary theory is known as 'shear lag' since it is due to a shear deformation in the panels.

In box girders, wide steel flanges and the deck slab are susceptible to shear lag, particularly at the supports. This can be neglected at ULS, but must be considered for fatigue behaviour. Though exact calculation of shear lag is complex, simple relationships for standard cases are quite adequate for normal purposes.

3.6 Support of box girders

The support arrangements of a bridge should be able to carry the twisting moment caused by any eccentric disposition of traffic loading. Plate girder bridges are tortionaly flexible and weak, hence they must be provided with at least two bearings at the support. For box girders only one bearing needs to be provided at intermediate supports and twin bearings at the end support to carry the tortional forces. Where there is significant plan curvature, single bearing may be sufficient as the curvature itself generates tortional restraints.

4.0 INITIAL DESIGN

4.1 General

In the initial design stage, the selection of structural arrangement and member sizes is covered after the layout of the highway has been finalized. Highway bridges are usually designed to carry a combination of uniformly distributed loading and loading from an abnormal heavy vehicle. These loads, together with other secondary loads, are specified in codes. The step-by step procedure for detailed design of box girder bridges is given in Appendix I. Steps for detailed design of various components of the structure are given Appendix II - XV.

4.2 Choice of a box girder form

For straight bridges, box girders may be expensive; but has advantages of pleasing appearance and reduced maintenance. For bridges with significant plan curvature, box girders should always be considered. Generally, a box girder bridge would have approximately same weight of steel as an I-beam bridge; deck slab thickness will be similar. With tortionally stiff beams in box girders, number of bearings and support position can be minimized. Curvature is more easily achieved with box girders and the tortional effects can be accommodated easily. However, box girders require a more complex analysis and design than simple I-beams.

4.3 Cross-section arrangements

The basic variables in choosing cross-sections with box girders are:

- The shape of the box - trapezoidal or rectangular
- Closed or open steel section
- With or without cross girders

Three examples of typical cross-sections generally used in practice are given in Figs 13-15. In Fig. 13, the section provides a line support and the slab span is effectively that of an ordinary plate girder. In Fig. 14, a wider box is used with wider spacing between boxes. Thick slab (300 mm), tortional restraint of the box and the stiffened steel top flange, all go to increase the spacing between the boxes.

In Fig. 15, the open steel box is widened to create approximately equal spans for the slab. With trapezoidal section, the inclined webs reduce the width of the bottom flange and, for a given area increase its thickness. Thus the flange becomes more fully effective. For longer spans, narrow rectangular box girders can be used. Rectangular sections are suited to wide decks. Haunched boxes are easily arranged with rectangular sections. For wide boxes, crossbeams may also be provided.

4.4 Section depth

Typically, the depth of the box girder may be assumed between 1/20 and 1/25 of the major span. Shallower section would result in greater weight. Variable depth sections can be easily achieved for rectangular section. If a curved soffit is used, internal transverse flange stiffeners would be required to resist the radial component of force.

4.5 Initial selection of flange and web sizes

Flange and web sizes depend on the configuration of the cross-section and the moment to be carried. At first, a coarse estimate is used to determine properties for a simple grillage model and that model is used to give a better indication of the distribution of bending moment. Dead load effects constitute a small proportion of the total design load effects. A concrete deck slab of 250 mm thickness and surfacing of 120 mm can be assumed.

Steel dead load can be assumed between 150 Kg/m^2 and 500 Kg/m^2. Live load depends on the number of lanes of traffic, which are carried. Footways should also be loaded. For approximately constant sections and roughly equal spans, dead load bending moments can be based on moments in a fixed-ended beam. Total live load bending moment at mid span and support regions can be calculated similarly as proportions of moments in a fixed-ended beam. For haunched beam, more moment is attracted to intermediate support regions; less is carried at mid span. In the case of box girders, loads are shared equally among the number of boxes. The live load moments are increased by 20% to cater for the approximation. The total dead and live load moments can then be used to make a first estimate of flange sizes; webs can be ignored while fixing flange sizes. The girders will be made up in several sections, suitable for transportation. Splice positions are decided and plate thicknesses are changed appropriately. Information about availability of steel plate and sections may be obtained during the initial design stage. For economy, it is best to standardize on as few plate sizes as possible.

4.6 Economic and practical consideration

The initial design - the configuration in section and elevation - takes proper account of the particular features of box girders, their construction, performance and maintenance. A good knowledge of how the box is constructed is essential. The flanges and webs will be fitted with stiffeners before they are assembled. Cross-frames or diaphragms will be needed at this stage to ensure that the cross section is held in shape during welding. Closed trapezoidal boxes are usually assembled in inverted position and the bottom flanges added last. Internal welding after closure is usually necessary - support diaphragm at least must be welded all round. Joints between flanges and webs are easier to make as fillet weld. The assembled box should be of appropriate size to fit transportation on public road. Open boxes will require plan bracing on the open side, to provide tortional stiffness during construction. Fitting and welding of stiffeners is expensive and it is often cheaper to use a thicker plate with less stiffening. Stiffened diaphragm can be very expensive to fabricate. Thick unstiffened diaphragms can even be considered for small boxes. Bolted splices are quicker to make on site. The diaphragm details, stiffeners and man hole sizes / positions may affect the box section size. Access for internal inspection and maintenance is an important consideration in design. Provision of access holes in the web or bottom flange at intermediate positions may be necessary. Internal drainage should also be considered. The main advantage of composite box girder over prestressed concrete box girder is the speed and ease of construction.

5.0 CONCLUSION

In this chapter, a brief introduction on box girder bridges is presented. The advantages of box girder bridges over the common plate girder type bridges are explained. The behaviour of box girder bridges is explained in detail. Some guidelines for the preliminary design of box girder bridges have also been included.

6.0 REFERENCES

BRITISH STANDARDS INSTITUTION

BS 5400: Steel and composite bridges

1. Part 1: 1988 General statement
2. Part 2: 1978 Specification for loads
3. Part 3: 1982 Code of practice for design of steel bridges
4. Part 4: 1990 Code of practice for design of concrete bridges
5. Part 5: 1979 Code of practice for design of composite bridges
6. 2.Iles D.C., 'Design guide for composite box girder bridges' SCI Publication 140, The Steel Construction Institute, 1994

7. Timoshenko S.P. and Gere J.M., Theory of elastic stability (2nd Edition), McGraw-Hill, London, 1961
8. Timoshenko S.P. and Goodier J.N., ' Theory of-Elasticity', 2nd Edition, McGraw-Hill, London,

Fig. 1 Completed structure

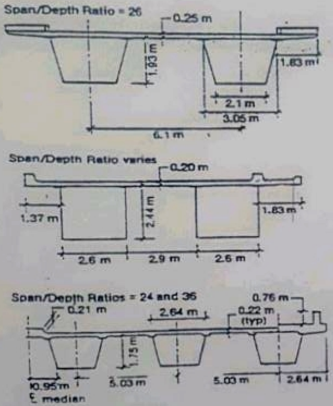


Fig. 2 Typical cross sections

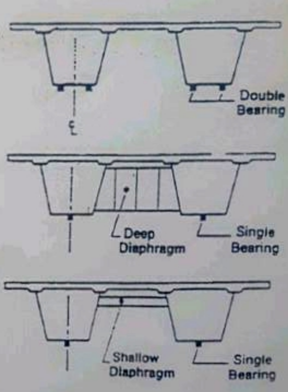


Fig. 3 Various support geometries

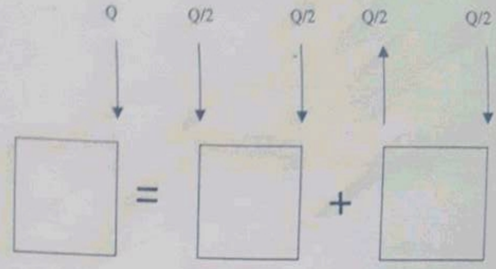


Fig. 4 Bending and torsion components of an eccentric load

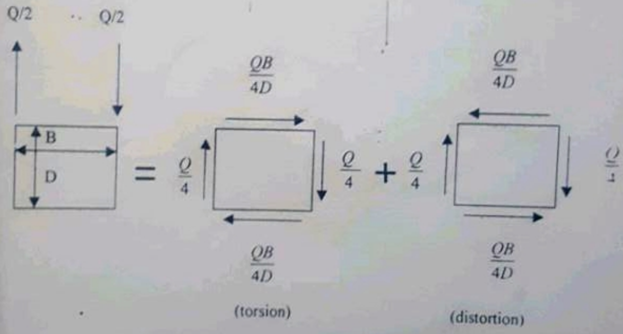


Fig. 5 Pure torsion and distortion components

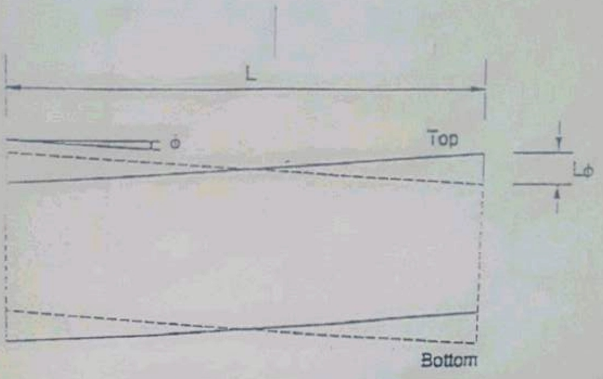


Fig. 6 Shear displacement of top and bottom flanges (ends kept plane)

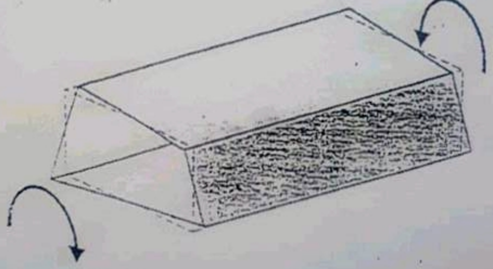


Fig. 7 Warping of a rectangular bob subject to pure torsion

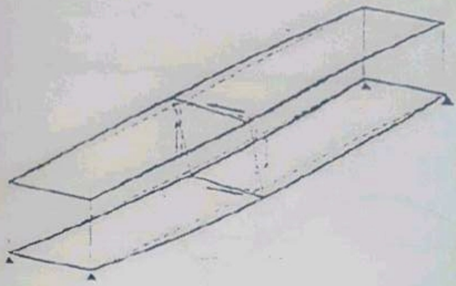


Fig 8 Distortion of unstiffened box (pinned corners)

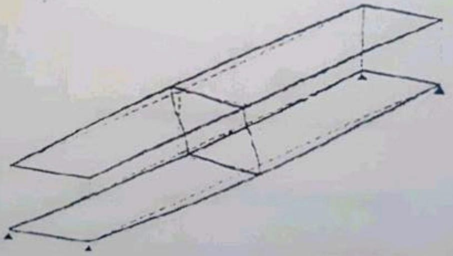
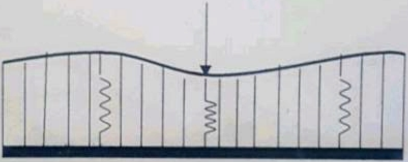


Fig 9 Distortion of box with stiff corners of cross-frames

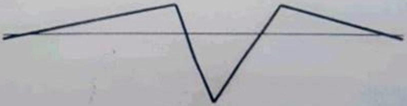
Displacement / distortion

Bending moment / warping

Fig. 10 Beam on elastic foundation analogy



Displacement / distortion



Bending moment / warping

Fig. 11 BEF model with intermediate springs

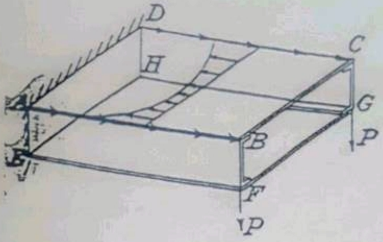


Fig. 12 Shear lag in a box beam

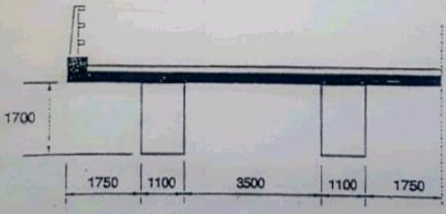


Fig. 13 Section with closed rectangular steel boxes

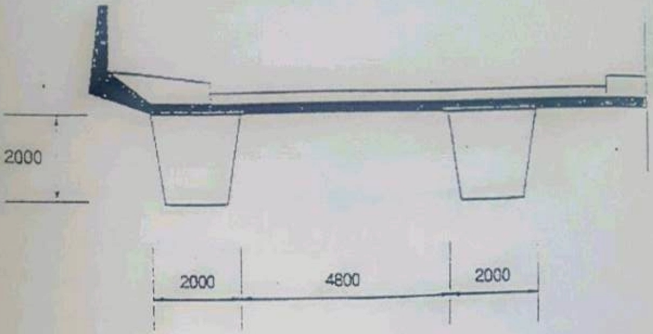


Fig. 14 Section with closed steel boxes

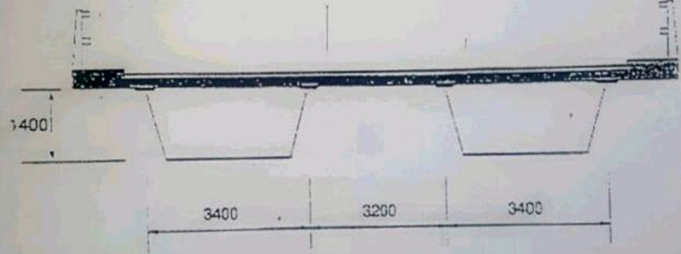


Fig. 15 Section with open steel boxes

Appendix I

Step-by-step procedure for the design of a box girder bridge

1. Assume initial design and develop structural configuration
2. Calculate the gross cross section properties
3. Carry out global analysis of the design and arrive at worst effects at critical locations
4. Check strength of main beams
5. Check the cross frames and distortional behaviour
6. Check the transverse web stiffeners
7. Check the diaphragms
8. Check the deck slab
9. Check the shear connection
10. Check the splices

Appendix II

Ultimate Limit State design of box section beams without longitudinal stiffeners

1. Determine effective section allowing for holes, slender webs and slender compression flanges
2. Calculate stresses through beam section using moments, shears and torsion from global analysis. Warping stresses are obtained from analysis of distortional effects
3. Calculate effective applied moment on box and shear on web
4. Calculate moment capacity of box section, M_D, M_R
5. Calculate shear capacity of one web, V_D, V_R
6. Calculate interaction relationship
7. Check whether resistance ≥ load effects; If "yes", the design is satisfactory. Otherwise, redesign

Appendix III

Ultimate limit state design of box section beams with longitudinal stiffeners

1. Determine effective section allowing for holes, slender webs and slender compression flanges

2. Calculate stresses through beam section using moments, shears and torsion from global analysis. Warping stresses from analysis of distortional effects

3. Check whether the flange stresses $\leq$ limiting stresses. If 'no', redesign; if 'yes', check web panel as in the next step

4. For each web panel calculate equivalent stress in web panel

5. Determine restraint conditions around edges of web panel. Calculate buckling coefficients and ratios (k and m)

6. Check if equivalent stress $\leq \sigma_{yw}$ and interaction ratio < 1.0

7. In step 6, if the inequality is not satisfied, redistribute up to 60% of web load ($t = 0.4\ t_w$) and go to step 2. If the inequality is positive, proceed to next step

8. Check longitudinal stiffeners to each web panel: calculate effective stiffener section and strength, σ_{ls}. Calculate stiffener stress, σ_{se} at 0.4 a from end.

9. Check if $\sigma_{se} \leq \sigma_{yw}$. If 'no', redesign longitudinal stiffeners; if 'yes', design is satisfactory.

Appendix IV

Ultimate limit state design of cross frames and distortional effects

1. Cross frames can be either ring cross frames or braced cross frames

2. In the case of ring cross frames determine effective section of all parts of the cross frame. In the case of braced cross frames, determine effective sections of web and flange stiffeners acting with cross members

3. Determine torsion applied over length equal to half the spacing either side of the cross frame.

4. Check strength of all parts of the cross frame.

5. Check if resistance $\geq$ load effects. If 'no', redesign; if 'yes, proceed to next step

6. Calculate distortional stiffness of box section

7. Calculate distortional stiffness of cross frame

8. If cross frame is effective, use panel length equal to frame spacing for calculation of distortional effects in main box girders

9. If cross frame is not effective, use 'smeared' cross frame stiffness for calculation of distortional effects in main box girders

Appendix V

Ultimate limit state design of transverse web stiffeners

1. Determine effective section of diaphragm/web junction
2. Determine axial loads and out-of-web moments
3. Calculate stresses in web and stiffener
4. If maximum stresses $\leq \sigma_{ys}$, redesign
5. If maximum stresses $\geq \sigma_{ys}$, check restraint provided by horizontal stiffeners. If 'yes', take l_s = distance between effective stiffeners; if 'no', take l_s = length of junction
6. Check if buckling capacity is adequate. If 'yes', the design is satisfactory; if 'no', redesign

Appendix VI

Ultimate limit state design of intermediate stiffeners

1. Determine effective section of stiffener and web
2. Determine loading on stiffener
3. Calculate stresses in stiffener and web
4. Check if $\sigma_e \leq \sigma_{yw}$ and $\sigma_m \leq \sigma_{ys}$
5. If the check in step 4 is 'no', redesign
6. If the check in step 4 is 'yes', check if bearing stress $\leq 1.336\ \sigma_{ys}$. If 'no', redesign; if the check is 'yes', check the buckling capacity.
7. If the buckling capacity is adequate, the design is satisfactory; if 'no', redesign

Appendix VII

Ultimate limit state design of unstiffened support diaphragms

1. Determine effective vertical sections at critical positions across the diaphragm
2. Calculate vertical stresses
3. Calculate horizontal stresses
4. Calculate shear stresses
5. Calculate buckling coefficient
6. Check for yield of diaphragm plate
7. Check for buckling of diaphragm plate
8. If resistance $\geq$ load effects, the design is satisfactory; otherwise, redesign

Appendix VIII

Ultimate limit state design of stiffened support diaphragms - diaphragm stiffeners

1. Determine effective section of diaphragm stiffeners
2. Calculate vertical stresses in bearing stiffeners
3. Determine stresses due to action with plate panels
4. Calculate bending stresses in bearing stiffeners
5. Check for yielding of diaphragm stiffeners
6. Check for buckling of diaphragm stiffeners
7. Check if resistance ≥ load effects. If 'yes', the design is satisfactory; if 'no', redesign

Appendix IX

Ultimate Limit State of plate panels

1. Determine effective vertical sections at critical positions across the diaphragm
2. Calculate vertical stresses including those within the effective section of a bearing stiffener
3. Calculate horizontal stresses in plate panels
4. Calculate shear stresses in plate panels
5. Calculate for yielding of diaphragm plate
6. Calculate for buckling of diaphragm plate
7. Check if resistance ≥ load effects. If 'yes', the design is satisfactory; if 'no', redesign

Appendix X

Ultimate state design of deck slab

1. Determine the maximum stresses in concrete or reinforcement due to global bending
2. *Check if stresses ≤ limiting stresses. If 'no', redesign, if 'yes', proceed to next step*
3. Determine maximum moments in slab due to local bending
4. Calculate ultimate bending resistances in slab

5. Check if resistance ≥ load effects. If 'yes', the design is satisfactory; if 'no', redesign

Appendix XI

Serviceability limit state of deck slab

1. Determine maximum stresses in concrete or reinforcement due to global bending

2. Determine the stresses due to local bending

3. For concrete in compression, check if stresses ≤ limiting stresses. If 'no', redesign

4. For reinforcement in tension: calculate crack width

5. Check if crack width ≤ limiting crack width . If 'no', redesign; if 'yes', the design is satişfactory

Appendix XII

Ultimate limit state for shear connection

1. Calculate shear flow at positions of highest Serviceability Limit State shear flow

2. Select transverse reinforcement to carry shear load

3. The design is satisfactory

Appendix XIII

Serviceability Limit State for shear connection

1. Calculate shear flow at support, 1/4 L and mid span

2. Determine nominal strength of connectors

3. Select shear connector spacing to provide design resistance

Check for fatigue

4. Calculate maximum and minimum load on connectors for fatigue vehicle

5. Determine stress range at weld to connector, σ_v

6. Determine limiting stress range for road category, σ_H

7. Check if $\sigma_v \leq \sigma_H$. If 'no' redesign, if 'yes', satisfactory

Appendix XIV

Ultimate Limit State for bolted splices

1. Determine load in tensile and compressive regions
2. Select bolts and spacing

Check for bolts

1. Calculate force per bolt for ULS stress distribution through beam section
2. Check if force ≤ friction capacity and force ≤ shear and bearing capacity
3. If it is 'no' in step 2, redesign; if 'yes', check tensile areas of beams
4. Determine effective section through bolt holes
5. Check if stress ≤ limiting stress. If 'no', redesign; If 'yes', proceed to step 6

Check for covers

6. Calculate stresses in cover plate
7. Check if cover plate stresses ≤ limiting stress. If 'no', redesign; if 'yes', design is satisfactory

Appendix XV

Serviceability Limit State for bolted splices

1. Check if bolts bear at ULS

Check for bolts

2. Calculate serviceability limit state load on bolts in flanges and webs
3. Calculate friction capacity of High strength friction grip bolts at SLS
4. Check if bolt capacity ≥load. If 'no,, redesign; if 'yes', the design is satisfactory