Major Project Report

Load Distribution Factors for Composite Multi-Cell Box-Girder Bridges

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Structural Engineering

Submitted by

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Candidates Declaration Certificate

This is to declare that the major project titled "**Load Distribution Factors for Composite Multi-Cell Box-Girder Bridges"** is the bonafide record of work done by me (Bhaskar Sengar) for the partial fulfillment of the requirements for the degree of Master of Engineering in Civil Engineering (Structural Engineering), Delhi College of Engineering. This project has been carried out under the supervision Dr.Prof.D.Goldar I have not submitted the matter embodied in this report for the award any other degree or diploma.

July 28, 2005

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Abstract

Composite steel-concrete multi-cell box-girder bridges combine excellent torsional stiffness with elegance. While the current design practices recommended few analytical methods for the design of multicell box girder bridges, practical requirements in the design process necessitate a need for a simpler design method. This thesis presents an extensive parametric study- using the finite element method- in which 20 bridges of various geometries were analyzed. The parameters considered here are: number of cells, number of lanes and span length. Based on the parametric study, moment and shear distribution factors are deduced for such bridges subjected to IRC loading.

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1.0 INTRODUCTION

The use of multicell box girders in bridge deck construction can lead to considerable economy. This type of construction leads to an efficient transverse load distribution, due to the excellent torsional stiffness of the section. Further, utilities and services can be readily provided within the cells.

Various analytical and numerical methods have been developed for analysis of cellular bridges. The American association of State Highway and Transportation Officials (AASHTO 1996) and the forthcoming Canadian Highway Bridge Design Code (CHBDC 1997) recommend the use of the grillage-analogy method, folded-plate method, finite-strip method, and finite-element method for the analysis of composite multicell box girder bridges. Published research on this subjects,[e.g., Scordelis et al.(1985) among others] dealt with analytical and numerical formulations, while other researchers (Scordelis 1975; Siddique and Ng 1988, etc.) conducted experimental studies to investigate the accuracy of the existing method of analysis. Several investigators (e.g., NCHRP 1991; Nutt et al. 1998) studied the load distribution in multicell box girder girder bridges. Since in India not much research had been done in composite cellular bridges therefore Central Road Research Institute (CRRI) has taken step in this field of research however, the aforementioned investigations were confined to reinforced or prestressed concrete construction and did not include composite concrete deck-steel construction. Therefore, load distribution factors for moment and shear are required for required for composite cellular bridges to fill the gap found in previous studies as well as in bridge codes.

1.1 The Aim of Parametric Study

The objective of this study is to conduct a parametric study to examine the key parameters that may influence the load distribution characteristics of composite concrete deck-steel multicell box girder bridges under IRC loading. It should be noted that the dead load of the bridge was not considered in this study. The parameters considered herein are number of cells, number of lanes, and span length. The results from experimental study on a bridge model was first considered for validating the model adopted in the parametric study, but due to shortage of time it was not done. The data generated from the study is used to deduce expressions for moment as well as shear distribution factors for different loading conditions to aid in the design of such bridges.

1.2 Structure of the Thesis

An overview of general structure of this thesis is presented below.

In Chapter 2, general information about the types of composite construction is given, their behaviour and how bonding between concrete is done by the help of different shear connectors.
In Chapter 3, information about the general behaviour of composite beams is given such interface behaviour and buckling behaviour.
In Chapter 4, a selection of common theories are discussed. The subjects are: theory of thin-walled structures, bending of thin plates, buckling of thin plates, box girder analysis and torsion of thin-walled open sections. This chapter also discusses about the different types global analysis of composite bridges.
In Chapter 5, the behaviour of steel box girder on the application of external load had been discussed along with bracing system.
In Chapter 6, the description about bridge prototype and its modeling in STAAD.Pro, the STAAD.Pro analysis results and use of these results to deduce distribution factors of 20 Prototype bridges along with graphical variation of distribution factors with respect to parameters considered.
In Chapter 7, the distribution formulae for composite bridge according to IRC loading were deduced by the help of distribution data’s generated in chapter 6
In Chapter 8, conclusions are stated and some recommendations for further research are suggested.
In Appendix A, some basic commands in STAAD.Pro are presented along with the editor language used in STAAD.Pro used to model and analyse the composite box girder bridge.
In Appendix B, the manually solved case of composite box girder bridge, in which moment of resistance of the section is calculated by elastic analysis.

1.3 Composite Bridges

Steel beams supporting concrete slabs have been used to form he basic super- structure of large numbers of deck bridges for many years. Since 1945 the number of composite bridges being built has significantly increased. The pressure of steel shortage in Germany after the Second World War forced engineers to adopt the most economical design method

available to be able to cope with the large amount of reconstruction of bridges and buildings destroyed. New codes of practice in other countries, the publications of papers describing the results of experimental work and eventually the publication of text books have all helped to make engineers familiar with the composite construction [1].

Composite bridges are structures that combine materials like steel, concrete, timber or masonry in some combination. The behavior of the composite structure is heavily influenced by the properties of its component materials. For example, the use of a concrete slab on a steel girder uses the strength of concrete in compression and the high tensile strength of steel. Looking at the basic behavior of a composite structure there are two fundamental effects that need to be considered: the differences between the materials and the connection of the two materials. Stronger, stiffer materials like steel attract proportionally more load than materials such as concrete. If there is no connection then the materials will behave independently, omitting the positive effects, but if adequately connected the materials act as one whole structure.

Most common composite structures are either precast, prestressed concrete beams with a cast concrete slab or steel girders with a concrete slab. Composite structures can be used for a wide range of structures such as foundations, substructures, superstructures and for a diverse range of bridge structures like tunnels, viaducts, footbridges and cable stayed bridges.

Steel-concrete composite box girders may advantageously be used for bridges with long spans, for bridges with significant horizontal curvature or simply for aesthetic reasons. The boxes may be complete steel boxes with an overlay slab or an open box where the concrete slab closes the top of the box.

The open top form of box girders, consisting of steel webs and a bottom flange, has only small top flanges sufficient for stability during concreting. The advantages of this form are that access to all parts of the section is available, which, e.g., facilitates welding, and that

the web can be inclined which allows a larger span in the transverse direction of the bridge. A disadvantage of the open box is that the high torsional stiffness of a closed section is not present during construction until the concrete slab has gained strength, which makes it more sensitive to lateral instability during construction.

The stresses induced by the loads will depend upon the magnitude of the load and its eccentricity, the box geometry and the number and stiffness of diaphragms. The use of a box form will aid the distribution of eccentric loads. Vertical loads that act eccentrically with respect to the centre line in a box girder results in twisting of the box section. Twisting moment is resisted by pure shear stresses in the walls of a box. Longitudinal normal stresses arising from the relative warping of the section under torsion are not considered in theory of pure torsion. However, these stresses can attain very large values when the closed cross-sections are flexible. For example, considering a general loading on a box section, as shown in Figure 1, in which a single vertical eccentric load is replaced by sets of forces representing vertical, torsional and distortional loading. The general loading in Figure 1 can be represented as two different components of loading, one causing bending and the other causing torsion as shown in Figures 1(b) and 1(c), respectively. The torsional loading component can be subdivided further into a pure torsional component and a distortional component as shown in Figures 1(d) and 1(e), respectively. Although the pure torsional component will normally result in negligible longitudinal stresses, the distortional component will always tend to deform the cross-section, thus creating distortional stresses in the transverse direction and warping stresses in the longitudinal direction. The distortion of the cross-section will be resisted by cross frames and diaphragms and hence an accurate analysis involves evaluating the distortional warping and shear stresses and the associated distortional bending stresses in the transverse frames [1].

Here D = depth of cell;
B = width of cell;
Q = vertical load.

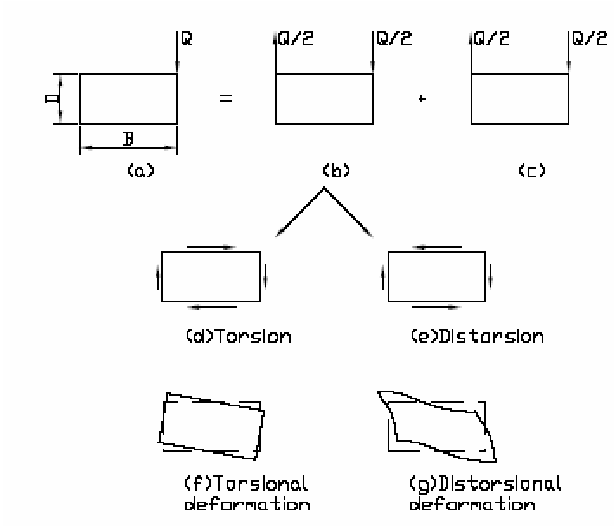


Fig.1: Idealisation of eccentric loading in box girder (from [1]).

Where composite action can be achieved, certain structural advantages appear. In comparison with the non-composite case the advantages may be summarised as: a reduction in steel area required, an increase in the overload capacity and a reduction in construction depth. The first two factors will lead to a reduction in the steel weight required to support a given load and the composite structures will show economy over their non-composite counterparts [2].

1.4 Comparison between Composite Box Girder Bridge and I-Girder Bridge.

1) Economies in box girder design are possible in comparison with similar I-beam and plate girder structures because the completed composite girder section has a higher torsional stiffness, and consequently a greater lateral distribution of live load, than I-beam structures having a similar flexural strength.

2) Additional economies are also possible in the fabrication and erection of box girders, as compared to similar I-beam structures, by virtue of the elimination of much wind and transverse bracing.

1.5 Load Distribution

For the computation of the bending moment due to live load, the distribution of the loads between longitudinals has to be determined. the economics and even the practicability of bridges often depend on maximum utilisation of the structure. The designer is generally trying minimise the dead weight of structure to carry a given load and the problem becomes intensified as spans increase. For this reason account needs to be taken of the load-distributing properties of the deck system particularly where wheeled loads of the IRC vehicle type are concerned. Considering for example a point load placed directly over one girder, the distribution problem is that of determining the proportion of the point load transferred to the other girders by cross girders, deck slab or other transversely spanning elements. Much attention has been given to the problem and solution techniques of varying complexity and accuracy exists [3]. Suffice it to say that for the initial design of composite decks a useful method is of the type in which cross girders and transverse deck are replaced by an equivalent elastic medium, while the longitudinal beam and slab remain unaltered. Such a method using a basic function analysis has been developed by Hendry and Jaeger. Where there access to a computer, standard grillage programs can be used but

unless a large amount of work is involved the 'hand' method will be end to be quick and simple. Hendry-Jaegar method is directly applicable to composite bridge made of concrete slab resting on steel I-girders. So, to analyze composite box girder bridge either we need to idealize steel box in to I-sections and solve it by the help of Henry-Jaegar method or solve it by finite element method by taking help of some computer software. A simple two cell box girder (1l-2c-20) has been solved manually to show the application of Hendry-Jaeger's method is given in Appendix A

To solve finite element problem manually is too cumbersome so, STAAD.Pro is used as a computational tool to simplify the process.

1.6 Finite Element Analysis

The finite element method as we know it today seems to have originated with Courant in

1943 [4]. Courant determined the torsional rigidity of a hollow shaft by dividing the cross-section into triangles and interpolating a stress function φ linearly over each triangle from the values of φ at nodes.

The name finite element was coined by Clough in 1960. Many new elements for stress analysis were soon developed. In 1963, finite element analysis acquired respectability in academia when it was recognised as a form of the Rayleigh-Ritz method. Thus finite element analysis was seen not just as a special trick for stress analysis but as a widely applicable method

having a sound mathematical basis. The first textbook about finite element analysis appeared in 1967 and today there exists an enormous quantity of literature about finite element analysis [5].

General-purpose computer programs for finite element analysis emerged in the late 1960's and early 1970's. Since the late 1970's, computer graphics of increasing power have been attached to finite element software, making finite element analysis attractive enough to be used in actual design. Previously it was so tedious that is was used mainly to verify a design already completed or to study a structure that had failed. Computational demands of practical finite element analysis are so extensive that computer implementation is mandatory. Analyses that involve more than 100 000 degrees of freedom are not uncommon [4]. Finite element analysis, also called the finite element method, is a method for numerical solution of field problems.

A field problem requires determination of the spatial distribution of one or more dependent variables. Mathematically, a field problem is described by differential equations or by an integral expression. Either description may be used to formulate finite elements.

Individual finite elements can be visualised as small pieces of a structure. In each finite element a field quantity is allowed to have only a simple spatial variation, e.g. described by polynomial terms up to x^2 , xy and y^2. The actual variation in the region spanned by an element is almost certainly more complicated; hence a finite element analysis provides an approximate solution.

In more and more engineering situations today, we find that it is necessary to obtain approximate numerical solutions to problems, rather than exact closed-form solutions.

Elements are connected at points called nodes and the assemblage of elements is called a finite element structure. The particular arrangement of elements is called a mesh. How the finite element method works can be summarised in the following general terms [6]:

1. ***Discretise the continuum***. The first step is to divide the continuum or solution into elements. A variety of element shapes may be used and different element shapes may be employed in the same solution region.

2. ***Select interpolation functions***. The next step is to assign nodes to each element and then choose the type of interpolation function to represent the variation of field variable over the element.

3. ***Find the element properties***. Once the finite element model has been established the matrix equation expressing the properties of the individual elements is ready to be determined.

4. ***Assemble the element properties to obtain the system equations***. The matrix equations expressing the behaviour of the elements must be combined to form the matrix equations expressing the behaviour of the entire solution region or system.

5. ***Solve the system equations***. The assembly process of the preceding step gives a set of simultaneous equations that can be solved to obtain the unknown nodal values of the field variable.

Finite element analysis has advantages over most other numerical analysis methods

1.7 STAAD.Pro

STAAD.Pro is a Structure analysis design program providing powerful analysis features in an interactive and visual environment. The geometry definition of each part is parametric and feature based, which allows quick modifications. Parts can be created in numerous ways and they are then assembled together to create the analysis model. Section and material properties can be defined and assigned to regions of the parts. The program offers a range of analysis procedures, such as: static stress analysis, eigenvalue buckling analysis and collapse and postbuckling analysis.

There are also possibilities to take nonlinear behaviour into consideration, including geometric, material and contact nonlinearity. The loading can be created combining concentrated, distributed and pressure loads and body forces. STAAD.Pro contains advanced algorithms for automatic meshing regions and the density of the mesh can be controlled by applying mesh seeds globally and locally. The program also includes an extensive element library, including element families such as:

- Solid elements
- Shell elements
- Membrane elements
- Beam elements
- Truss elements
- Spring elements
- Rigid elements

Finally STAAD.Pro provides a suite of post processing features in order to enable efficient interpretation of results [7].

2.0 Composite Construction

2.1 Types of Composite Construction

2.1.1 UNIQUENESS OF STEEL AND COMPOSITE CONSTRUCTION

Composite steel and concrete structural members are formed by bonding a steel component, such as an I-section, to a concrete component, such as a reinforced concrete slab, so that the two components now act as one. In order to standardize the terminology used in this thesis the steel component of a composite member will be referred to as the steel element, while the concrete component of the composite member will be referred to as the concrete element.

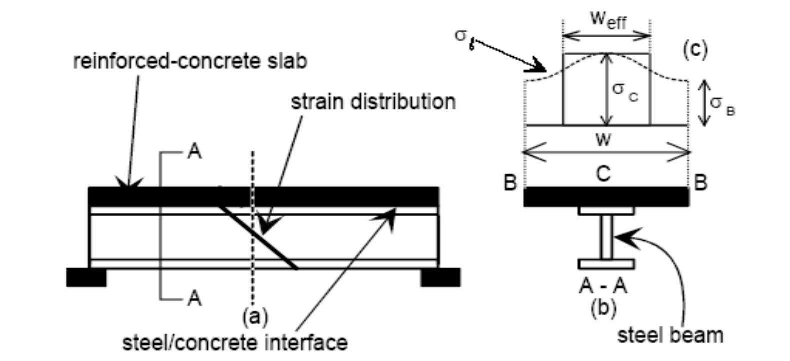
Engineers are all too familiar with the problems involved in constructing in either steel or concrete, as each of these materials has its own peculiarity. For example, steel structural members are generally fabricated as components consisting of thin plate elements, so they are prone to local and lateral buckling, as well as to fatigue. Therefore, steel standards are concerned predominantly with the prevention of failure by instability or buckling. Conversely, concrete structural members are generally thick and unlikely to buckle. However, concrete is very weak in tension, and is inclined to creep and shrink with time. In order to overcome the problem of weak tensile strength, a major effort in design is in placing steel reinforcing bars as a substitute for the weak concrete.

Steel and concrete composite structural members are also subjected to the possibility of buckling of the steel element and tensile cracking of the concrete. However, they are also prone to failure of the bond between the steel element and the concrete element, which is often referred to as debonding. It is the behaviour of this bond between the concrete and steel elements that gives composite construction its unique peculiarity. Hence, in the design of composite steel and concrete structures, the engineer not only has to understand the behaviour of the individual components of the steel and concrete, but also the bond between these components.

Composite structures also have other unique characteristics. For example, a prismatic steel beam can be considered to have a flexural strength that is constant at any section of the beam, and so in design it is sufficient to ensure that the flexural strength is larger than the maximum applied moment. The same can be said of a reinforced concrete beam, except at positions that are very close to the ends of the reinforcing bars. However, unlike steel beams or reinforced concrete beams, the strength of a composite beam varies along its length, and so the strength at all sections of a composite beam must be compared with the applied load along the beam. Steel plates in steel members can buckle locally in either direction transverse to the plane of the plate. However in composite members, a steel plate adjacent to a concrete element can only buckle away from the concrete element and hence these plates are less prone to buckling than those in steel members. Such an example is the steel skin in a concrete-filled tubular composite column, which can only buckle away from the enclosed concrete core.

2.1.2 USE OF COMPOSITE FORMS OF CONSTRUCTION

There are several reasons for combining steel and concrete elements to form composite members. Most structural slabs are made from reinforced concrete, as this has good sound and fire insulation properties, and a common form of construction is to use steel beams to support the slab as shown in Fig. 2.1. The applied flexural forces in this unbonded system are therefore resisted solely by the steel beam, with the top half being subjected to compression with the possibility of buckling, and with the bottom half being in tension. When the steel/concrete interface is bonded together so that the two elements act as one, then the steel element will be subjected to tension and the concrete element will resist the compressive forces. Best use is therefore being made of the two materials, as concrete is effective in compression and steel is effective in tension. The depth of the beam resisting flexure has now increased from that of the steel beam acting by itself to that of the composite beam, and this can double both the flexural strength and stiffness of the beam, and consequently lead to reduced span to depth ratios. Reducing span to depth ratios may only give a small saving in beam materials, and thus appear only to achieve a small overall saving in costs. However, it is worth noting that this reduction in depth affects the whole building and will lead to reduced floor heights, with consequent savings in column heights, glazing and cladding, and will eventually lead to reduced foundation loads.



W = width of concrete slab
W_{eff} = effective width
σ = bending stress

Fig.2.1 Steel and concrete elements

Steel sections used as both beams and columns are often encased in concrete for fire protection, as in Fig. 2.2. The concrete encasement also helps resist local and overall buckling of the steel section, as well as being capable of resisting compressive forces. Hence a smaller steel section can be used when the steel section is encased in concrete, and this will offset in part the extra cost of encasement.

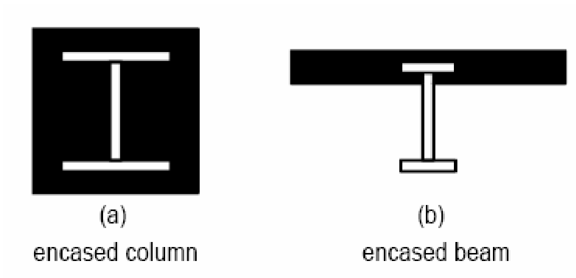


Fig.2.2 Fire protection

Composite structures are also formed when steel sections are used as permanent formwork for the concrete portion, and are designed to act integrally with the concrete so that the two components act as one. An interesting long-standing and efficient form of construction is to use composite columns made from circular steel tubes as in Fig. 2.3(b). Using the steel tube externally to the concrete element may appear to defeat the object of using composite construction for fire prevention, but this technique has many other advantages which may well offset the additional cost of fire protecting the exposed steel. For example, the concrete element is now fully encased by the steel element so that the concrete is less prone to shrinkage, has a higher compressive strength as it is now triaxially restrained, and acts integrally with the steel element. Furthermore, the steel element is restricted to buckling away from the concrete element, so that it is less likely to buckle locally. A further advantage of this type of construction is obtained when the steel column is designed to withstand several stories of construction load, so that concreting of the steel tube does not delay the steel construction sequence and occurs well below the steel construction zone. Concrete-filled hollow steel elements are now used extensively in high-rise office building construction.

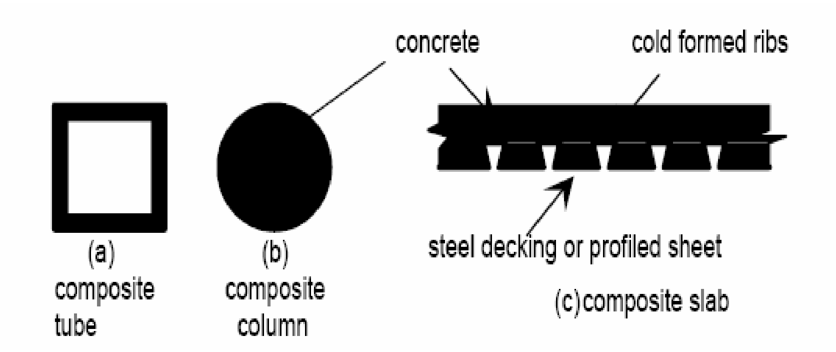


Fig. 2.3 Permanent and integral shuttering

A further example of permanent and integral shuttering is the use of steel decking in the construction of composite slabs in Fig. 2.3(c). The steel decking, which is made by cold rolling ribs into galvanised sheets, is designed to support the construction loads during concrete casting, and is also designed to act compositely with the hardened concrete, so reducing the amount of reinforcing bars required. A very common form of construction is to substitute a composite profiled slab for the solid slab in Fig.2.1, and then to bond the two elements together with mechanical shear connectors forming a double composite action, so that the composite slab now acts as part of the composite beam. Steel decking can also be used as permanent and integral shuttering to the sides of reinforced concrete beams, and also in the construction of reinforced concrete walls.

Steel plates can be bonded to the surface of existing reinforced concrete beams as in Fig. 2.4. The composite action between the steel plate and the reinforced concrete flexural member increases both the shear and flexural strength, and hence this form of construction can be used to upgrade existing reinforced concrete structures.

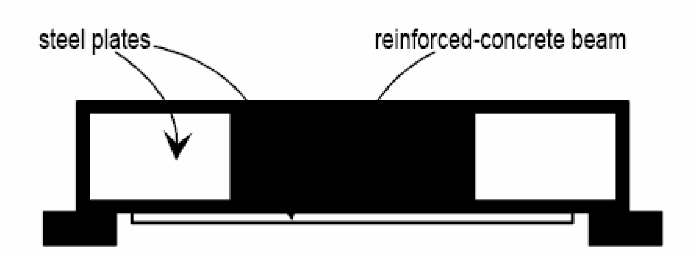


Fig.2.4 Strengthening and stiffening

2.1.3 BOND BETWEEN STEEL AND CONCRETE ELEMENTS

The bond which must be achieved between the steel element and the concrete element in a composite member is crucial to the composite action. When the two elements only touch at an interface such as shown in Fig. 2.1, then they are often tied together using mechanical forms of shear connection, examples of which are given in Fig. 2.5. When one element encases the other element as in the columns and tube in Figs.2.2(a) and 2.3(a) and (b), then the two elements are tied together by interface forces induced by the geometry of the encasement and any bond strength. In the case of composite plated beam construction in Fig. 2.4, the plate can be bonded to the existing reinforced concrete beam by gluing or bolting, or by a combination of gluing and bolting. In all cases, the bond must be designed to resist the longitudinal shear forces at the steel/concrete interface. However, the bond must also be designed to prevent separation between the steel and concrete elements in order to ensure that the curvature in the steel and concrete elements is the same. Hence the interface bond must be able to resist both tensile forces normal to the steel/concrete interface, and shear forces parallel to the steel/concrete interface.

Stud shear connectors, as shown in Fig. 2.5(a), are probably the most common type of mechanical shear connector used, and consist of a bolt that is electrically welded to the steel member using an automatic welding procedure. The shank and the weld-collar adjacent to the steel flange are designed to resist the longitudinal shear load, whereas the head is designed to resist the tensile loads that are normal to the steel/concrete interface. Studs of 19 mm diameter are used frequently, and have a shear strength of around 120 kN. Bolts can also be attached directly to the flange, prior to casting the concrete, through friction welding by spinning the bolt whilst in contact with the flange, or by bolting as shown in Fig. 2.5(b). Alternatively, the steel and concrete elements can be bolted together after casting as in (g) and (h). A further form of attaching shear connectors is to use explosively driven pins as in (d). In hand welded channels (c), the longitudinal shear load is resisted mainly by the bottom flange of the channel whilst the top flange resists the tensile loads normal to the steel/concrete interface. Angle sections (f) behave in a similar fashion, except that the normal tensile loads are resisted by reinforcing bars that are threaded through holes in the leg. Block connectors (e) form a very stiff and strong shear connection, and the hooped bars resist the normal tensile loads.

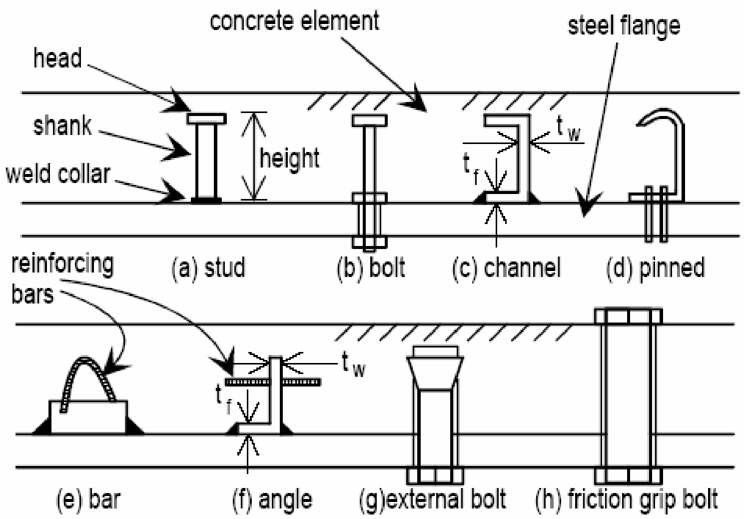


Fig. 2.5 Mechanical shear connectors

There is an enormous variety of mechanical shear connectors varying in shape, size and methods of attachment. However, they all have the following important similarities. They are steel dowels embedded in a concrete medium, they have a component that is designed to transmit longitudinal shear forces, they have a component that is designed to resist normal tensile forces and hence prevent separation at the steel/concrete interface, and they all impart highly concentrated loads onto the concrete element.

When mechanical shear connectors are not used, such as occurs when one element is encased by another as in Figs. 2.2 and 2.3, then the two elements are tied together by interface forces that are induced by the geometry of the encasement. Full encasement, as in the columns in Fig. 2.2(a) and 2.3(b), ensures that there is no separation, and the longitudinal shear is now transmitted by interface friction and chemical bond, and it is often assumed that the strains in the steel and concrete at the interface are the same. A similar action occurs in composite slabs, as in Fig. 2.3(c), but in this case it is the encasement of the rib that prevents interface separation.

There are numerous types of profiled sheeting, some of which are shown in Fig. 1.6. Their shapes are generally chosen as a compromise between enhancing the bond at the steel/concrete interface, and enhancing the performance of the permanent shuttering to resist the construction load and any instability effects due to the wet concrete. In all cases, the longitudinal shear is transmitted by the encased ribs, so this form of shear connection will be referred to as rib shear connectors. The bond performance of these ribs is improved by rolling indentations and protrusions into the rib, so that the longitudinal shear is also transmitted by mechanical action that is analogous to the transfer of shear in cracked reinforced concrete sections by aggregate interlock.

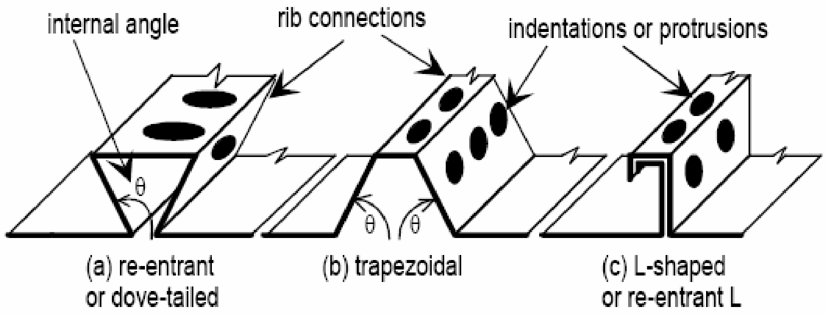


Fig. 2.6 Steel decking from profiled sheets

2.1.4 SHAPES OF COMPOSITE SECTIONS

The shapes of composite sections are as varied as the imagination of the designer. Composite sections can be made from hot-rolled steel sections by encasing the steel sections in concrete as in Fig. 2.7(a), or by encasing the concrete element with steel (b), (c) and (d), or by encasing the concrete within two skins of steel to form a composite tube as in (e).

Mechanically bonded sections have a variety of forms as shown in Fig. 2.8. T-beams, that are made from standard hot-rolled sections as shown in (a), are a common form of construction. Because the top flange of the steel element of the T-beam contributes very little to the overall strength and is mainly present to hold the shear connectors in place, the composite section can be made more efficient by welding a plate to the bottom flange and reducing the size of the top flange as shown in the plate beam in (b), thereby making the steel section monosymmetric. In fact, the top flange can be removed altogether and the connectors welded to the web, as in the hybrid beam in (c). The depth of the composite section can be increased with the addition of haunches in (d), and when the steel element is at the edge of the slab or adjacent to a service duct a composite L-beam is formed, as in (e). Composite beams in bridges are often formed from steel box sections (g) instead of I-sections, and can also be formed from open box girder sections (f) where the connectors are concentrated over the webs.

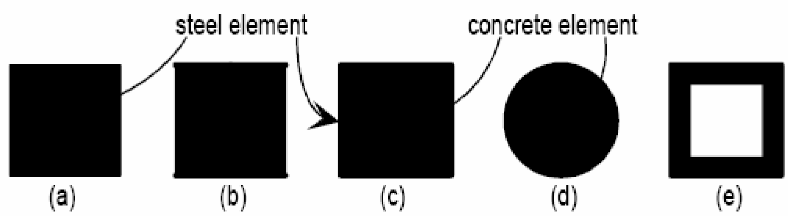


Fig. 2.7 Hot rolled steel elements

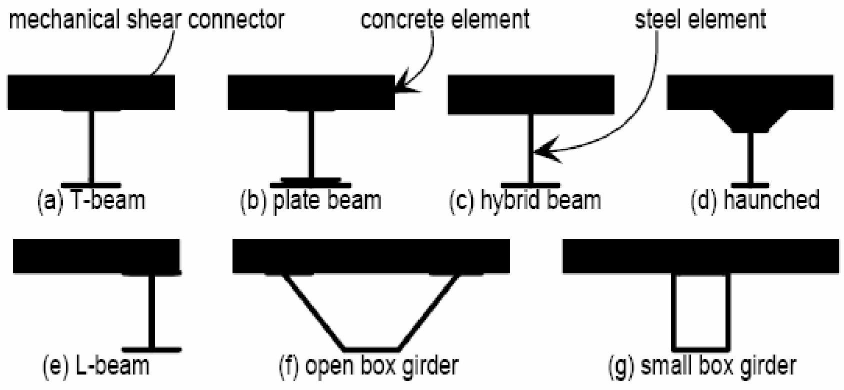


Fig. 2.8 Mechanically bonded sections

Composite beams are often constructed with openings in the webs to allow for the passage of services, as in Fig. 2.9(a). Openings are also formed in stub girder construction (b) to allow for the passage of services, to accommodate secondary transverse composite beams, and to increase the depth of the main section and hence allow for greater spans. Composite truss girders (c) and (d) are another form of open web construction.

As mentioned earlier, composite sections are also formed when steel decking, which is made from cold rolled profiled sheets, is used as permanent and integral shuttering for reinforced concrete members. Examples of this form of construction are shown in Fig. 2.10. Composite profiled slabs (a) are a very common form of construction in steel framed structures, and are now often used in concrete framed structures as well. Profiled sheets can also be used as permanent and integral shuttering in the construction of reinforced concrete walls (c) and beams (b). In advanced forms of construction, fully braced box girders (d) can be constructed from profiled sheeting, and then infilled with concrete.

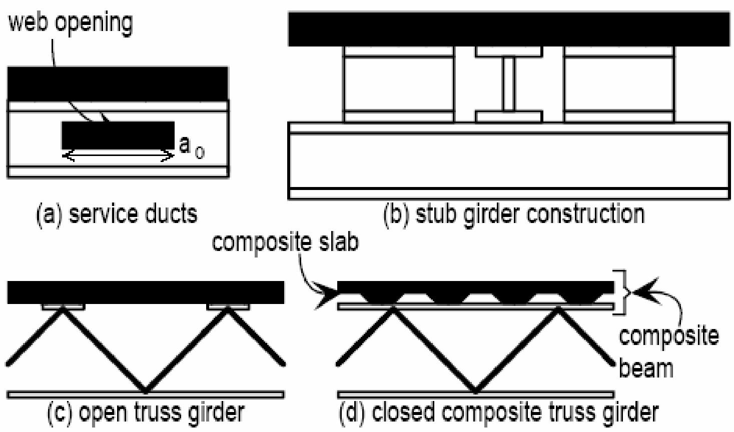


Fig. 2.9 Open web girders

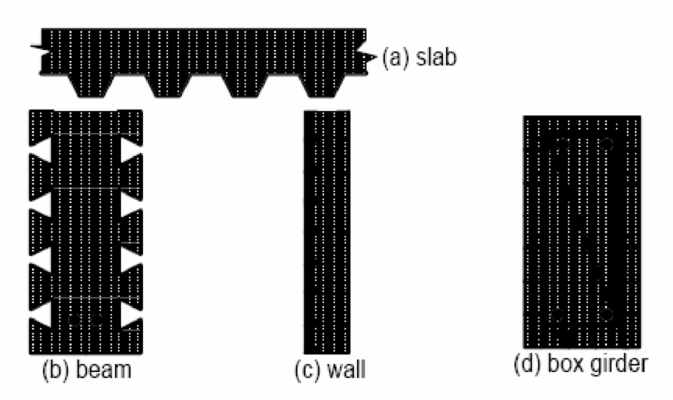


Fig. 2.10 Profiled sections

Examples of double composite action are shown in Fig. 2.11. The composite slab can be made to act integrally with the steel section by welding connectors through the troughs of the profiled section. The ribs of the profiled section can be parallel to the steel element (b), or transverse to the steel element (c), or even diagonal to the steel element. When the ribs of the profiled section are parallel to the steel element as in (b), then a haunched composite beam is formed. This type of construction, in which there is a double composite action, can also be used where there are web openings, as in Fig. 2.9(a), and also in composite stub girders (b) and in trusses (c) and (d). In the closed box girder section in Fig. 2.11(a), the shear connectors tie the steel girder to the concrete slab along the length of the beam. However, the shear connectors also tie the top flange of the steel girder to the concrete slab, so that this plate is acting compositely with the slab in the transverse direction. Hence composite actions in two directions are occurring within the composite beam. The flange plate acting compositely with the slab in Fig. 2.11(a) is an example of composite plate girder construction. This form of construction also occurs in composite plated beams, as in Fig. 2.4, where the shear connection between the plate element and the reinforced concrete element can be provided by gluing or bolting.

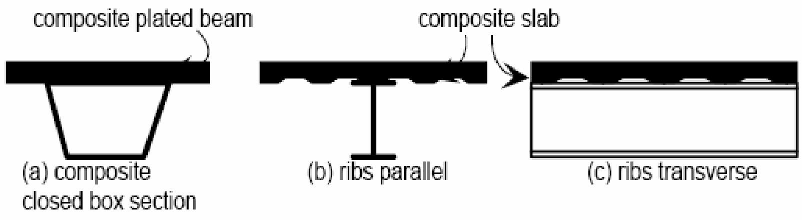


Fig. 2.11 Double composite action

Finally, in order to illustrate the versatility of composite construction, it is worth noting that the composite truss in Fig. 2.9(d) is an example of triple composite action, as the composite truss girder incorporates composite beams between the joints as well as transverse composite slab action. This form of construction is especially popular in North America.

3.0 General Behavior of Composite Beams

3.1 Introduction

The deformation of a continuous composite beam under gravity loads is sketched in Fig. 3.0(a). The distribution of forces in the positive or sagging region between the points of contra flexure is completely different from the behaviour in the negative or hogging region between a point of contra flexure and the adjacent support.

Between points of contra flexure, the composite beam can be visualized as being simply supported, as shown in Fig. 3.0 (b). The resultant of the flexural forces in the concrete element of the composite beam is compressive, and the resultant forces in the steel element is tensile, so that the cross-section of the beam can be considered to consist of both a steel and concrete element as in (c). The concrete fiber adjacent to the steel/concrete interface is trying to expand under the flexural forces as shown in (b), whereas the steel fiber adjacent to the interface is trying to contract under the flexural loads. This relative deformation distorts the connectors, causing them to bear onto the concrete in the zones marked with an asterisk. The connectors are therefore applying a thrust onto the concrete that is directed towards the mid span of the beam, and are themselves subjected to horizontal shear forces. The flexural distortion of the composite beam also tries to induce vertical separation between the steel and concrete elements, and the tensile component of these forces is resisted by the shear connectors.

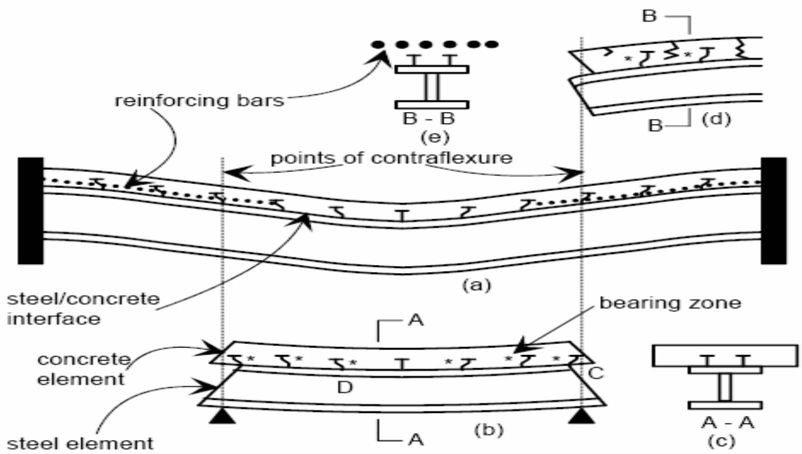


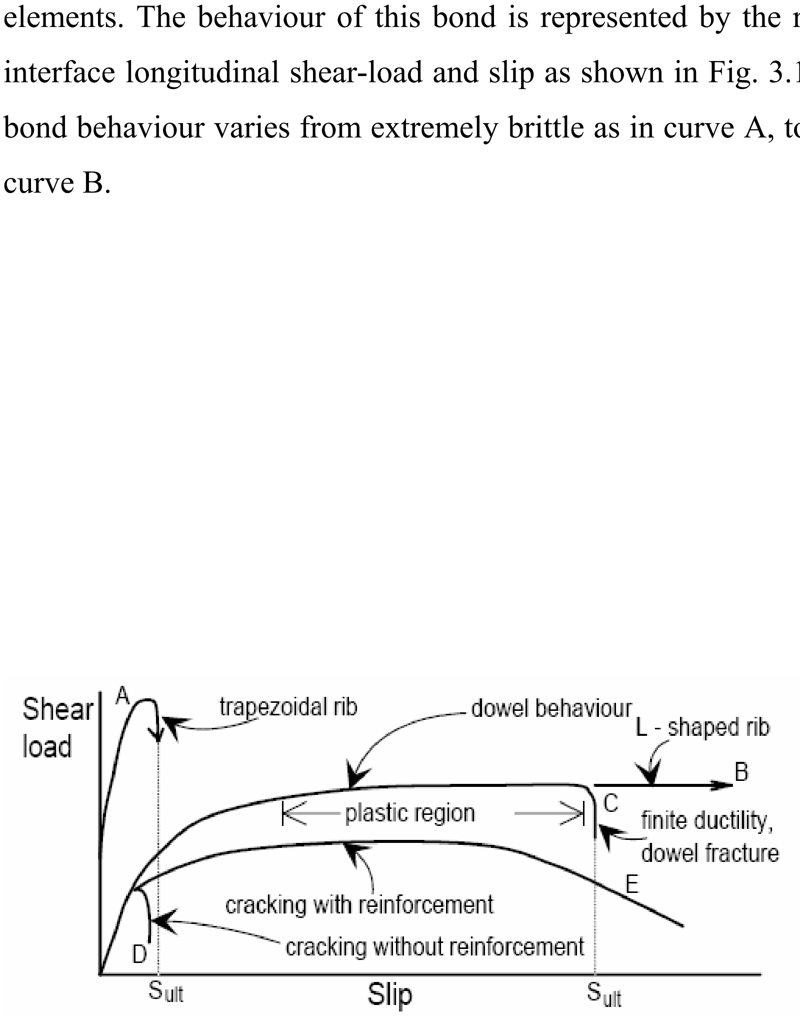
Fig. 3.0 Deformation of composite beam

In most simply supported composite beams in buildings, the neutral axis lies in the concrete element, so that all of the steel is in tension and hence buckling does not occur once the concrete has hardened. When the neutral axis lies in the steel element, the top flange may be totally or partially in compression. However, the steel top flange is less likely to buckle than a compression flange in a steel beam because it can only buckle away from the concrete element, and even then this buckle may be restrained by the ability of the shear connectors to resist the normal tensile forces across the interface.

In the negative moment region adjacent to the supports as in Fig. 3.0 (d), the composite beam acts as a cantilever. The concrete is in tension and is generally fully cracked, so it does not contribute directly to the strength of the structure. The beam can therefore be considered to consist of two steel components, these being the steel element and the reinforcing bar elements as in (e). The section is therefore no longer composite but a steel structure, although it is still referred to as a composite beam as the concrete is required to transfer the load to the reinforcing bars, in the same way as the load is transferred through cracked concrete to the reinforcing bars in concrete structures. The flexural deformation of the steel and concrete elements causes the same distortion in the shear connectors as occurs in the positive region, so that these connectors are also applying a thrust onto the concrete directed towards the centre of the beam as shown in (d). The bottom flange and much of the web of the steel element is subjected to compression, so that the negative moment region is prone to buckling. This behaviour is described in Section 3.1.2.

3.1.1 INTERFACE BEHAVIOR

The deformations, stress distributions and modes of failure of composite beams depend on the behaviour of the shear connection between the steel and concrete elements. The behaviour of this bond is represented by the relationship between the interface longitudinal shear-load and slip as shown in Fig. 3.1 It can be seen that this bond behaviour varies from extremely brittle as in curve A, to extremely ductile as in curve B.



X-axis=Slip (mm)

Y-axis=Shear load (KN)

Fig. 3.1 Interface shear behaviour

The rib in a profiled sheet as shown in Fig. 2.6, in composite profiled sheet construction transfers most of the longitudinal shear and prevents interface separation as it is encased by the concrete. This form of shear connection will therefore be referred to as rib shear connectors to distinguish them from mechanical shear connectors as in Fig. 2.5. The bond characteristics of rib shear connectors depends on the shape of the profiled rib. Composite beams made with L-shaped ribs, as in Fig. 2.6(c), exhibit extremely good ductile bond characteristics, as in curve B in Fig. 3.1. This is because the rib is fully encased by the concrete, and so can slide through the concrete without detaching. L-shaped rib connectors can therefore sustain almost unlimited slip before failure, and hence are well suited for composite construction. However, beams made with the trapezoidal ribs as in Fig. 2.6(b) tend to exhibit very brittle bond characteristics, as in curve A of Fig. 3.1, because the rib is not fully encased. After the chemical bond at the interface is broken, the trapezoidal rib tends to detach from the concrete, although embossments do help to make the rib more ductile. The bond in composite beams with trapezoidal ribs tends to fail at small finite slips S_{ult}, as in curve A in Fig. 2.1, which may prevent the composite beam from achieving its flexural capacity.

Mechanical shear connectors have a similar range of bond characteristics as rib shear connectors. The stud, bolt and angle connectors in Fig. 2.5 exhibit substantial plastic regions, but will fracture at a finite slip as in curve C in Fig. 3.1. This is because the slip capacity is now controlled by the deformation capacity of the connector, as compared to profiled L-shaped ribs which simply slide through the concrete. Block connectors, (e) in Fig. 1.5, have very limited plastic regions as in curve A in Fig. 3.1 and can therefore be considered as non-ductile. All mechanical shear connectors have finite slip capacities, and fracture of the connector at this finite slip S_{ult} can cause premature failure of the composite beam if this limited slip capacity is not designed against. Unlike profiled-rib connectors, mechanical connectors tend to impose very high concentrations of load onto the concrete element. This concentrated load is transferred from the steel element to the concrete element through the dowel action of the connectors. All mechanical connectors are simply steel dowels embedded in a concrete medium as illustrated in Fig. 3.2(a). The resistance of a connector to this dowel action is referred to as the dowel strength, and this strength is often quoted in national standards. The concentrated load is dispersed into the concrete element and the action of this dispersal can induce tensile cracking, as shown in the plan view of the concrete element in Fig. 3.2 (b). These tensile cracks are induced by ripping, shear and splitting actions. Tensile cracking can also be induced by the dowel action, particularly when the connector is also resisting separation at the steel/concrete interface of the composite beam, and these cracks are referred to as embedment cracks as shown in (a). These four forms of tensile failure of the slab can affect both the dowel strength and ductility of the shear connection. When there are no reinforcing bars crossing the planes of cracking, then the strength of the dowel reduces immediately cracking occurs and the slip capacity is also reduced, as in D in Fig. 1.13. The presence of reinforcement across a crack plane makes failure much more ductile, as in curve E, and can even allow increases in load after cracking.

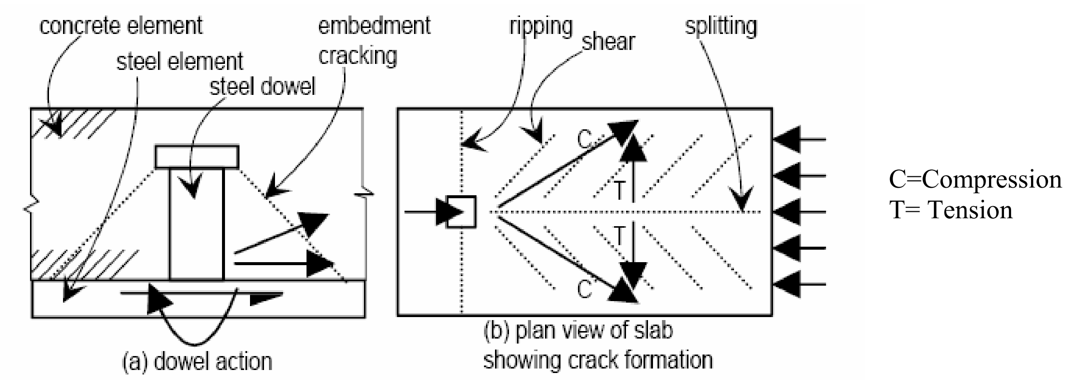


Fig. 3.2 Transfer of connector force

3.1.2 BUCKLING BEHAVIOUR

Buckling in General

Buckling means loss of the stability of an equilibrium configuration, without fracture or separation of the material or at least prior to it [4]. Generally there are two types of buckling: *Bifurcation buckling* and *Snap-trough buckling.* Bifurcation buckling is the kind of buckling familiar from elementary column theory. For an axial compressive load of magnitude P_{cr}, called the critical load, the straight prebuckling configuration ceases to be a stable state of equilibrium and an alternative buckled configuration is also possible. Buckling may also appear without bifurcation, as a limit point, where there is no alternative and infinitesimally close equilibrium configuration.

A primary path is the original load-displacement line or curve and its extension.

The secondary path is the alternative path that originates when the critical load is reached. The two paths intersect at the bifurcation point. Past the bifurcation point, the primary path is unstable. Although it is possible mathematically that the structure follows the primary path, a real structure will follow the secondary path instead. If the secondary path has a positive derivative (rises), the structure has post-buckling strength. A limit point is a maximum on a load-displacement curve.

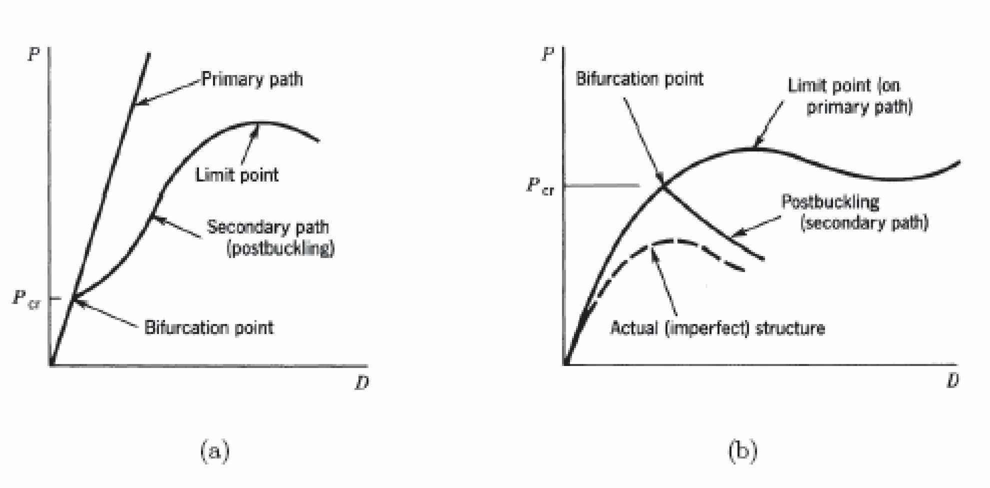
It is not a bifurcation point because there is no immediate adjacent equilibrium configuration. When a limit point load is reached under increasing load, snap through buckling occurs, as the structure assumes a new configuration by suddenly moving. A collapse load is the maximum load a structure can sustain without gross deformation. It may be greater or less than the computed bifurcation buckling load as shown in Figure 3.3.

Figure 3.3 shows possible behaviours when loads are applied to a structure. Here P is the load magnitude and D is the displacement. The material is assumed to remain linearly elastic and loads are gradually applied.

In Figure 3.3 (a) the prebuckling path is linear. At bifurcation, two very closely spaced equilibrium positions are possible. Thereafter, for $P > P$cr, a real (imperfect) structure follows the secondary path. Since the postbuckling path rises, the structure can be said to have postbuckling strength. In this case, Pcr characterizes a local buckling action that has little to do with overall strength. The structure will collapse at the limit point, which is considerably greater than the critical load.

Figure 3.3 (b) describes a different type of behaviour. The structure has a nonlinear prebuckling path and the postbuckling path falls, hence the structure has no postbuckling strength. Closely spaced primary and falling secondary paths implies that the structure is imperfection sensitive. Imperfection sensitivity means that small changes in load directions, geometry and/or boundary conditions strongly affect the collapse load.

If knowledge of the structural behaviour of the model is little or none, one mustanticipate that a computed bifurcation buckling load may be far above or far below the actual collapse load, imperfections may be influential and that prebuckling nonlinearities may be important [8].



X-axis= Displacement (mm)

Y-axis=Load (KN)

Fig 3.3: Possible load versus displacement behaviour of thin-walled structures (From [8]).

3.1.2.2 Overall Buckling

Both in the construction phase and when the concrete has hardened, overall buckling of the steel element must be designed against. Under the action of wet concrete during construction, the top flange of the steel is subjected to compressive stresses in the positive bending region, and instability may take place by the usual lateral-torsional buckling that is treated in national codes of practice for structural steelwork. Lateral-torsional buckling, as shown in Fig. 3.4(a), occurs when each cross-section of the steel element displaces and twists as a rigid body. Fortunately, this mode of failure is relatively easy to predict, and cross-bracing such as that shown in Fig. 3.4(b) or propping is often used.

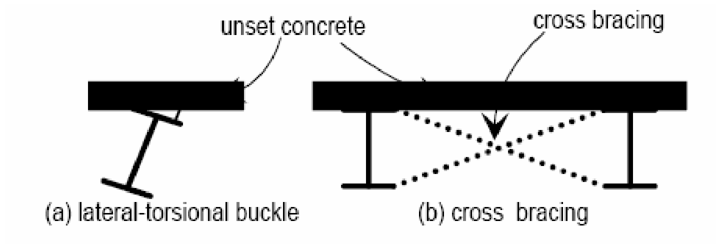


Fig. 3.4 Lateral - torsional buckling

On the other hand, the steel in the negative moment region of a hardened composite beam may buckle in what is called a lateral-distortional mode (Bradford 1992), as shown in Fig. 3.6. This is because the shear connection between the top flange of the steel element and the concrete slab prevents the twist of this flange during buckling, rendering the stiffness of the web to provide resistance to buckling of the bottom compressive flange of the steel element. The cross-section thus distorts in its plane during buckling. Clearly in this case, lateral-torsional buckling such as that shown in Fig. 3.6(a) is not possible, although many national composite standards consider this as the limit state to be designed for, and accordingly cross-bracing is often specified. Lateral-distortional buckling strengths are greater in composite beams than lateral-torsional buckling resistances, and there is both theoretical and experimental evidence that the cross-bracing shown in Fig. 3.5(b) is unnecessary to prevent lateral-distortional buckling in the majority of cases when a universal section steel element is used.

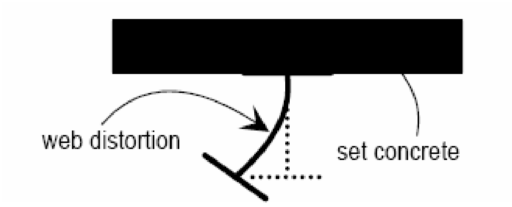


Fig. 1.6 Lateral - distortional buckle

3.1.2.3 Local buckling

In many cases, local buckling of the compression flanges and web is an instability possibility that must be prevented. Local buckling of a steel cross-section, as shown in Fig. 3.7, takes place when the component plates of the section distort out-of-plane, but with the straight line junctions at the intersections of the component plates remaining straight. The steel element in a composite beam may buckle locally in both the positive and negative bending regions.

Under positive bending and when the neutral axis lies in the steel web, the top flange is subjected to compressive stresses. This flange may buckle locally, but the restraint offered by the shear connectors, and the presence of the rigid concrete slab, contribute towards suppressing this mode of buckling. Such a buckling mode is usually unlikely to occur in many situations prior to the attainment of the full plastic moment.

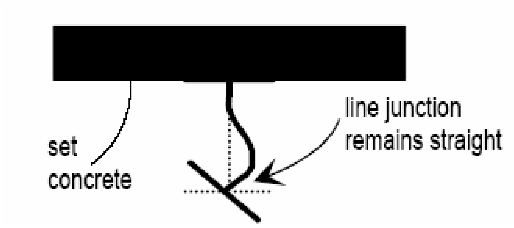


Fig. 3.7 Local buckling

In the negative moment region, local buckling can and often occurs. It is characterised by distortion of the web and rotation of the usually stocky flange, as shown in Fig. 3.7. To quantify local buckling, national composite codes of practice place limits on the depth to thickness ratios of the web which depend on the position of the neutral axis. Limits are also placed on flange width to thickness ratios. Compact sections are defined as having web depth to thickness ratios d_w/t_w such that a plastic hinge will form prior to local buckling, as well as requiring limits on the flange breadth to thickness ratio b_f/t_f. Non-compact sections will yield before local buckling, but will buckle locally before full plasticity of the beam is achieved. On the other hand, slender sections will buckle locally before first yield of the bottom flange. Some composite bridge girders are made from slender steel components, and reduced bending moments must be used. Recent research (Azhari and Bradford 1993) has shown that the provision of a longitudinal stiffener attached to the web near the bottom flange may increase the local buckling capacity. Whilst not greatly deployed yet in practice, a longitudinal stiffener should serve to obtain a more favourable section classification, so that the flexural capacity of the composite beam prior to local buckling may be increased.

4.0 THIN-WALLED STRUCTURES

In this chapter a brief literature review of thin-walled structures and an overview of bending and buckling of thin plates are presented together with the theories of torsion of thin-walled open sections. A thin-walled structure is defined as a structure that is made from thin plates joined along their edges. The plate thickness is small compared to other cross-sectional dimensions, which are in turn often small compared to the overall length [10]. There are several reasons why thin-walled structures must be given special consideration in their analysis and design. In a thin-walled beam the shear stresses and strains are much larger relative to those in a solid rectangular beam. When certain thin-walled structures are twisted there is a so-called warping of the cross-section and the Bernoulli hypothesis[1] is violated. The term warping is defined as the out-of-plane distortion of the cross-section of a beam in the direction of its longitudinal axis. Thin-walled structures are also susceptible to local buckling if the in-plane stresses reach their critical values. If this happens, the geometry of the cross-section changes, in contrast to overall buckling where the cross-sectional form is retained, as in the case of a pin-ended column. However, if a thin-walled column is made sufficiently long it may suffer overall buckling before it buckles locally. This means that thin walled structures must be designed against both local and overall buckling. Theory and experiments show that these two phenomena can interact and when this happens the buckling load can decrease below the values of the individual loads.

4.1 Brief Literature Review

There is an extensive amount of literature dealing with the theory of thin-walled structures and it is beyond the scope of this thesis to review this literature to any larger extent. However, a brief review of some literature on the elastic buckling of thin-walled structures is given below.

The membrane theory of plates was first studied by Euler (1766) and the flexural theory by Bernoulli (1789) and Navier (1823). The theory for combined membrane and flexural effects was developed by Kirchhoff (1877) and Saint-Venant (1883). At this state, the governing equation for thin isotropic plates loaded laterally with q per unit area and in-plane forces N_x, N_z and N_{xz} per unit length was

$$D\left[\frac{\partial^4 w}{\partial x^4}+2\frac{\partial^4 w}{\partial x^2\partial z^2}+\frac{\partial^4 w}{\partial z^4}\right]=q+N_x\frac{\partial^2 w}{\partial x^2}+2N_{xz}\frac{\partial^2 w}{\partial x\partial z}+N_z\frac{\partial^2 w}{\partial z^2} \tag{4.1}$$

where w is the lateral deflection [12].

At the turn of the twentieth century the equation governing the buckling of flat plates was available and it was known that it forms the basis of an eigenvalue problem. At that time it was not recognized that as the plate buckles the values of N_x, N_z and N_{xz} at a given point would vary because of the stretching of the plate.

The next development which overcame this deficiency was due to Foppl (1907) who introduced the stress function Φ and paved the way for von Karman (1910) to derive the governing equations for perfectly flat plates [11]:

$$D\left[\frac{\partial^4 w}{\partial x^4}+2\frac{\partial^4 w}{\partial x^2\partial z^2}+\frac{\partial^4 w}{\partial z^4}\right]=\frac{\partial^2\Phi}{\partial z^2}\frac{\partial^2 w}{\partial x^2}+\frac{\partial^2\Phi}{\partial x^2}\frac{\partial^2 w}{\partial z^2}-2\frac{\partial^2\Phi}{\partial x\partial z}\frac{\partial^2 w}{\partial x\partial z}+q \tag{4.2}$$

$$\frac{\partial^4\Phi}{\partial x^4}+2+\frac{\partial^4\Phi}{\partial x^2\partial z^2}+\frac{\partial^4\Phi}{\partial z^4}=Et\left[\left(\frac{\partial^2 w}{\partial x\partial z}\right)^2-\frac{\partial^2 w}{\partial x^2}\frac{\partial^2 w}{\partial z^2}\right] \tag{4.3}$$

These equations enabled the post-buckling behaviour of perfectly flat plates to be studied.The von Karman large-deflection equations for flat isotropic plates with in-planeloading were modified to account for anisotropy by Rostovtsev (1940) and later the effect of initial imperfections were included resulting in the following simultaneous equations:

$$\begin{aligned}D_x\frac{\partial^4 w}{\partial x^4}+2H\frac{\partial^4 w}{\partial x^2\partial z^2}+D_z\frac{\partial^4 w}{\partial z^4}=\frac{\partial^2\Phi}{\partial z^2}\frac{\partial^2(y+w)}{\partial x^2}+\\ \frac{\partial^2\Phi}{\partial x^2}\frac{\partial^2(y+w)}{\partial z^2}-2\frac{\partial^2\Phi}{\partial x\partial z}\frac{\partial^2(y+w)}{\partial x\partial z}+q\end{aligned} \tag{4.4}$$

$$\begin{aligned}\frac{1}{t_z E_z}\frac{\partial^4\Phi}{\partial x^4}+2\left(\frac{1}{K_{xz}}-\frac{\nu_x}{t_x E_x}-\frac{\nu_z}{t_z E_z}\right)\frac{\partial^4\Phi}{\partial x^2\partial z^2}+\frac{1}{t_x E_x}\frac{\partial^4\Phi}{\partial z^4}=\\ \frac{\partial^2 y}{\partial z^2}\frac{\partial^2 w}{\partial x^2}+2\frac{\partial^2 y}{\partial x\partial z}\frac{\partial^2 w}{\partial x\partial z}-\frac{\partial^2 y}{\partial x^2}\frac{\partial^2 w}{\partial z^2}-\frac{\partial^2 w}{\partial z^2}\frac{\partial^2 w}{\partial x^2}+\left(\frac{\partial^2 w}{\partial x\partial z}\right)^2\end{aligned} \tag{4.5}$$

These appear to be the most general equations currently available for solving plate buckling problems [11].

Bryan (1891) was the first to solve the problem of a simply supported rectangular plate with two opposite sides carrying uniform compressive loads. The same problem was later solved by Timoshenko (1907). Timoshenko also analysed many other cases of flat plates with different boundary conditions. Following the advent of the finite difference and the relaxation technique and later with the increasing use of computers and finite elements it has become relatively easy to solve this problem for a wide variety of plate shapes and stress distributions.

$$M_x = -D\left(\frac{\partial^2 w}{\partial x^2} + \nu\frac{\partial^2 w}{\partial y^2}\right) \tag{4.6}$$

$$M_y = -D\left(\frac{\partial^2 w}{\partial y^2} + \nu\frac{\partial^2 w}{\partial x^2}\right) \tag{4.7}$$

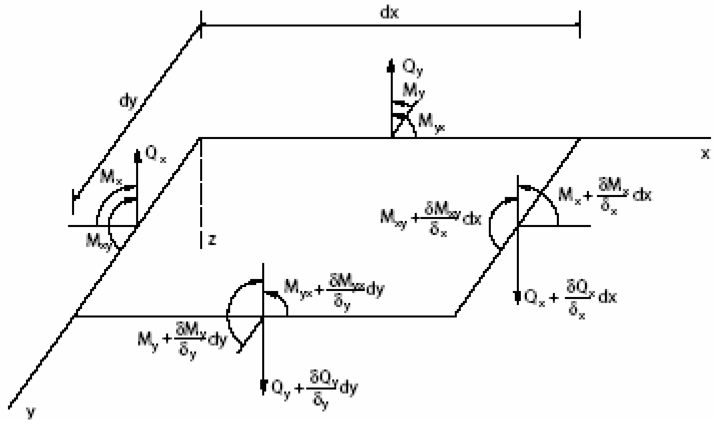


Fig. 4.1: Plate element (from [14])

4.2 Bending of Thin Plates

The structural analysis of a plate is carried out by considering the state of stresses at the middle plane of the plate. All the stress component are expressed in terms of the deflection *w* (*x, y*) of the plane. This deflection function has to satisfy a linear partial differential equation, which, together with its boundary condition, completely defines *w* (*x, y*). Fig. 4.1 shows a plate element cut from a plate whose middle plane coincides with the *xy* plane. The middle plane of the plate is subjected to a lateral load of intensity *q*. It can be shown, by considering the equilibrium of the plate element that the stress resultants are given as:

$$M_{xy} = -M_{yx} = D(1-\nu)\frac{\partial^2 w}{\partial x \partial y} \tag{4.8}$$

$$V_x = \frac{\partial^3 w}{\partial x^3} + (2-\nu)\frac{\partial^3 w}{\partial x \partial y^2} \tag{4.9}$$

$$V_y = \frac{\partial^3 w}{\partial y^3} + (2-\nu)\frac{\partial^3 w}{\partial y \partial x^2} \tag{4.10}$$

$$Q_x = -D\frac{\partial}{\partial x}\left(\frac{\partial^2 w}{\partial x^2} + \frac{\partial^2 w}{\partial y^2}\right) \tag{4.11}$$

$$Q_y = -D\frac{\partial}{\partial y}\left(\frac{\partial^2 w}{\partial x^2} + \frac{\partial^2 w}{\partial y^2}\right) \tag{4.12}$$

$$R = 2D(1-\nu)\frac{\partial^2 w}{\partial x \partial y} \tag{4.13}$$

where M_x and M_y are the bending moments per unit length in the *x* and *y* directions,respectively. M_{xy} and M_{yx} are the twisting moments per unit length. Q_x and Q_y are the shearing forces per unit length in the *x* and *y* directions, respectively. V_x and

V_y are supplementary shear forces in the x and y directions, respectively and R is the corner force. $D = Eh^3/12(1 - v^2)$ which is flexural rigidity of the plate per unit length, E is the modulus of elasticity, h is the thickness of the plate and v is Poisson's ratio. The governing equation for the plate is obtained as:

$$\frac{\partial^4 w}{\partial x^4} + 2\frac{\partial^4 w}{\partial x^2 \partial y^2} + \frac{\partial^4 w}{\partial y^4} = \frac{q}{D} \tag{4.14}$$

Any plate problem should satisfy the governing equation (4.14) and boundary conditions of the plate [3].

4.3 Buckling of Thin Plates

Buckling of a plate involves out-of-plane movement of the plate and results in bending in two planes. A significant difference between axially compressed columns and plates is apparent if their buckling characteristics are compared. For a column, buckling terminates the ability of the member to resist axial load, and the critical load is thus the failure load of the member. However, the same is not true for plates due to the membrane action of the plate. Subsequent to the critical load, plates under compression will continue to resist increasing axial force, and will not fail until a load considerably in excess of the critical load is reached. The critical load of a plate is therefore not its failure load. Instead, the load-carrying capacity of a plate must be determined by considering its post buckling behavior.

To determine the critical in-plane loading of a plate by the concept of neutral equilibrium a governing equation in terms of biaxial compressive forces N_x and N_y and constant shear force N_{xy} as shown in Fig. 4.2 can be derived as:

$$D\left(\frac{\partial^4 w}{\partial x^4} + 2\frac{\partial^4 w}{\partial x^2 \partial y^2} + \frac{\partial^4 w}{\partial y^4}\right) + N_x\frac{\partial^2 w}{\partial x^2} + N_y\frac{\partial^2 w}{\partial y^2} + 2N_{xy}\frac{\partial^2 w}{\partial x \partial y} = 0 \tag{4.15}$$

Numerical methods of analysis of plates, which include both geometric and material non-linearities are available today and these analyses are capable of assessing the ultimate strength and post-critical stiffness of plates with fabrication imperfections.

A square element, as shown in Fig. 4.3, whose edges are oriented at 45° to the edges of a plate subjected to pure shear, experiences tensile stresses on two opposing edges and compressive ones on the other two. These compressive stresses induce a form of local buckling with elongated bulges oriented at about 45° to the plate edges. As with the compressive loading, a thin plate loaded in shear can support an applied stress well in excess of the elastic critical one. This is due again to the resistance to in-plane deformation. As the applied shear stress is increased beyond τ_{cr} the plate buckles elastically and retains little stiffness in the direction in which the compressive component acts. However, the inclined tensile component is still resisted fully by the plate. The inclined buckles become progressively narrower and the plate acts like a series of bars in the tension direction, developing a so-called tension field. Further increase of applied stress causes plastic deformation in the part of the tension field, which rotates to line up more closely with the plate diagonal.

Tension field action is particularly important in plate and box girders, in which the function of the web plates is primarily to resist shear [14].

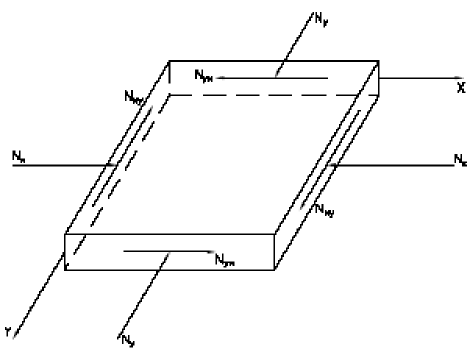


Fig. 4.2: Plate subjected to in-plane forces.

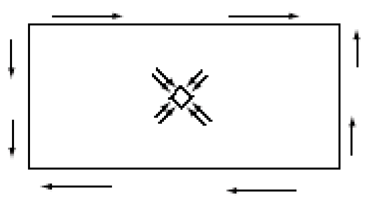


Fig. 4.3: Plate subjected to pure shear; stresses on square element at 45^{o}.

4.4 Plate and Box Girder Analysis

The high bending moments and shearing forces associated with the carrying of large loads over long spans as in the case of bridges frequently necessitates the use of fabricated plate and box girders. In their simplest form, plate and box girders can be considered as an assemblage of webs and flanges. In order to reduce the self-weight of these girders and thus achieve economy, slender plate sections are employed. Hence local buckling and postbuckling reserve strength of plates are important design criteria. Flanges in a box girder and webs in plate and box girders are often reinforced with stiffeners to allow for efficient use of thin plates. The designer has to find a combination of plate thickness and stiffener spacing that will result in the most optimal section with reduced weight and fabrication cost. There are some difficulties that are usually encountered by designers of plated structures [3]:

- The engineer's simple 'plane sections remain plane' theory of bending is no longer adequate, even for linear elastic analysis.

- Non-linear elastic behavior caused by the buckling of plates can be of great importance and must be allowed for.

- Because of this complex non-linear elastic behavior, and also because of stress concentration problems, some yielding may occur at loads which are quite low in relation to ultimate collapse loads. While such yielding may not be of great significance as regards rigidity and strength, it means that simple maximum stress criteria are no longer sufficient.

- Because of the buckling problem in plates and stiffened panels, complete plasticization is far from being realized at collapse. Hence simple plastic criteria are also not sufficient.

- Complex interactions occur between flanges, webs and diaphragms and the pattern of this interaction can change as the level of load increases.

4.5 Torsion of Thin-Walled Open Sections

Thin-walled open cross-sections composed of slender plates are particularly susceptible to lateral torsional buckling, because the torsional rigidity of such cross-sections is low and so their resistance to torsional instability is limited.

The analysis of lateral torsional buckling behaviour of beams is more complex than that of in-plane buckling behaviour of columns because the lateral buckling problem is intrinsically three-dimensional. The problem is further complicated because the lateral (out-of-plane) deflection and twisting are coupled, so this coupling effect must be considered in the analysis

4.5.1 UNIFORM TORSION OF THIN-WALLED OPEN SECTIONS

When an equal and opposite torque T is applied to the ends of a simply supported beam with a thin-walled open section, the twisting moment along the length of the members is constant and the beam is said to be under a *uniform torsion.* Under the action of the torque, warping of the cross-section will occur and plane sections of the cross-section no longer remain plane as a result of the uneven axial deformation that takes place over the entire cross-section.

For the simply supported beam, in which warping of all the cross-sections is unrestrained, the applied torque is resisted only by shear stresses developed in the cross-section. These stresses act parallel to the edge of the component plates of the cross-section, as shown in Fig. 4.4. The distribution of these shear stresses is the same for all thin-walled, open cross-sections. The magnitude of these shear stresses will be proportional to the distance from the midline of the component plate. These shear stresses are called *Saint-Venant shear stresses* and the associated torsion is referred to as *Saint-Venant torsion,* T_{sv}. The angle of twist γ over the length L caused by the Saint-Venant torsion is given by

$$\frac{\gamma}{L} = \frac{T_{sv}}{GJ} \tag{4.16}$$

where γ/L is the angle of twist per unit length, G is the shear modulus and J is the torsional constant of the cross-section.

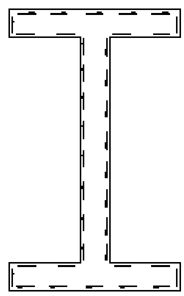


Fig. 4.4: Saint-Venant shear stress distribution in an I-section (from [7]).

The rate of twist is expressed as

$$\frac{\mathrm{d}\gamma}{\mathrm{d}z} = \frac{T_{sv}}{GJ} \tag{4.17}$$

where z is the coordinate axis along the length of the beam. Equation (4.17) can be written as

$$T_{sv} = GJ\frac{\mathrm{d}\gamma}{\mathrm{d}z} \tag{4.18}$$

The Saint-Venant torsion expressed in Equation (4.18) is also referred to as uniform or pure torsion.

4.5.2 NON-UNIFORM TORSION OF THIN-WALLED, OPEN SECTIONS

Consider a cantilever beam subjected to a torque applied at the free end. At the free end the cross-section is free to warp, so the applied torque is resisted only by Saint-Venant torsion. At the fixed end, however, warping is prevented. As a result, in addition to Saint-Venant torsion, there exists another type of torsion known as *warping restraint torsion* in the cross-section. If the cross-section is prevented from warping, axial strain and axial stresses must be induced in the cross-section, in addition to the shear stresses. These induced axial stresses are in self-balance because no external axial force is applied to the beam.

The resultant of these axial stresses in the two flanges constitutes a pair of equal moments called the *bi-moment,* M_f, acting oppositely in each of the two planes of the flanges.

The development of these bending moments, or bi-moments, in the flanges in the cross-section is shown in Fig. 4.5. Since warping is prevented at the fixed end, the two flanges of the beam must bend in opposite directions as the cross-section rotates under the action of the applied torque. The bending of the flanges will thus induce bending moments M_f at the fixed end. The bending moment can be expressed in terms of the lateral displacement u_f as

$$M_f = EI_f \frac{\mathrm{d}^2 u_f}{\mathrm{d}z^2} \tag{4.19}$$

where E is the modulus of elasticity, I_f the moment of inertia of one flange about the y axis of the cross-section, and u_f the lateral displacement of the flange. Associated with the bending moment in one flange is the shear force V_f given by

$$V_f = -\frac{\mathrm{d}M_f}{\mathrm{d}z} = -EI_f \frac{\mathrm{d}^3 u_f}{\mathrm{d}z^3} \tag{4.20}$$

The shear forces are present in both flanges of the I-section. They are equal in magnitude but act in opposite directions, as shown in Figure 4.6. This pair of shear

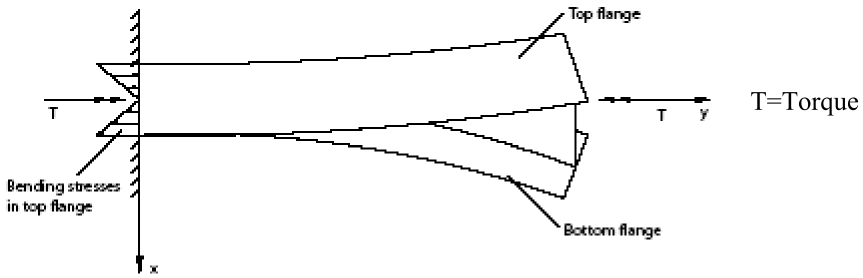


Fig. 4.5: Bending of flanges due to warping restraint at the fixed end (from [12]).

forces constitute a couple acting on the cross-section. The resulting torsion, which is referred to as the *warping restraint torsion* or *non-uniform torsion,* is given by

$$T_w = V_f h \tag{4.21}$$

where h is the distance between the lines of action of the shear forces. Equation (4.21) can be expressed as

$$T_w = -EI_f \frac{h^2}{2} \frac{\mathrm{d}^3\gamma}{\mathrm{d}z^3} = -EC_w \frac{\mathrm{d}^3\gamma}{\mathrm{d}z^3} \qquad (4.22)$$

where

$$C_w = \frac{I_f h^2}{2} \qquad (4.23)$$

is called the *warping constant* of the I-section. The warping constant is different for different cross-sections.

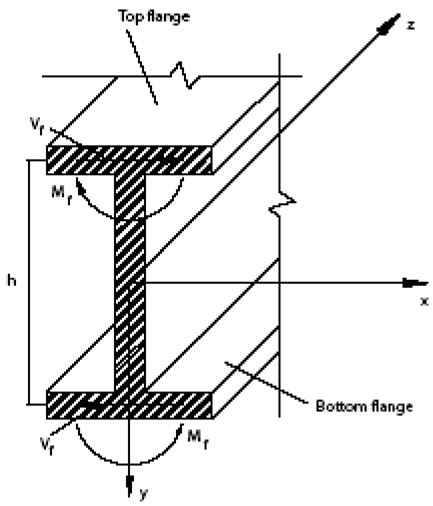


Fig. 4.6: Moment and shear developed at the fixed-end cross-section of an I-section (from [12]).

If warping is restrained, the applied twisting moment will be resisted by both Saint-Venant torsion and warping restraint torsion.

$$T = T_{sv} + T_w \qquad (4.24)$$

or

$$T = GJ \frac{\mathrm{d}\gamma}{\mathrm{d}z} - EC_w \frac{\mathrm{d}^3\gamma}{\mathrm{d}z^3} \qquad (4.25)$$

Equation (4.25) represents the internal twisting moment that will develop in the cross-section when the member is twisted. The first term represents the resistance of the cross-section to twisting and the second term represents the resistance of the cross-section to warping.

Saint-Venant torsion is always present when a member is subjected to twisting and rotates. Warping restraint torsion will develop if a cross-section is prevented from warping when it is being twisted [12].

4.6 Global Analysis

Global analysis is the calculation of stress resultants in critical regions throughout the structures for the various loading cases, for both the serviceability and the ultimate limit states. The results are often needed in the form of envelopes of maximum and minimum value and have to be summed fore calculation of cumulative damage. This work becomes impracticable unless the principle of superposition can be used. This implies the use of linear-elastic analysis at the serviceability limit state.

At the ultimate limit state, in elastic behaviour is acceptable. Appropriate methods of analysis are available for cross sections and for certain complete members, but for global analysis of the structure elastic theory is almost used for want of any thing better. The chief exception is the simple beam and slab deck, for which a combination of yield-line theory for the slab and plastic theory for the beams has been practicable

Most of the work on concrete decks and on steel bridge structures can be applied to composite bridge structures, provided that care is taken to consider their special features, given below.

1) ***Cracking of concrete***. For concrete box girder bridges, it is generally accepted that fir design purposes the distribution of internal forces, moments and displacements can be based on an elastic analysis of an uncracked homogeneous concrete system. This is also true for decks of composite bridges, but for composite members the differences between the reduction of flexural stiffness in positive moment and in negative moment regions due to

cracking is greater than in all-concrete structures, so ‘uncracked’ analysis of continuous members is less accurate.

The extent of cracking is strongly influenced by locked-in stresses due to shrinkage, and also makes uncertain the prediction of the effective breadth of a concrete flange. The coincidence in composite structures of these uncertainties with the need for accurate evaluation of stresses in steelwork, to avoid buckling and fatigue failures, is the principal difficulty that is peculiar to composite structures.

2) ***Shrinkage of concrete***. In comparison with an all-concrete member, axial shortening due to shrinkage is less, but changes of curvature are greater, as the shrinkage occurs in one flange of the member only. In continuous members, the secondary effects of shrinkage may influence the design of he steel work or planning of the sequence of construction.
3) ***Temperature effects***. These may be more severe than in composite structures, because bare steel both conducts and radiates heat more rapidly than concrete.
4) ***Slip at shear connectors***. This occurs only in composite structures and requires special consideration when partial-interaction design is used.
5) ***Method of construction***. Changes in the effective cross-sections of members during construction are likely to occur in long-span bridges in any material, but the widespread use of unproped construction forces designer to consier construction stresses in almost all composite bridges.

4.7ANALYSIS OF COMPOSITE SECTION

When a reinforced concrete slab is just supported over a steel I-beam, the two components have equal deflection; but their deformation and hence the stress pattern are different as shown in Fig.4.7 The bottom of the slab will be in tension, while the top of the steel beam will be in compression. The two components act in a non-composite manner.

In the case of composite section, where the total longitudinal shear is fully transferred at the junction of the steel beam and the in-situ concrete slab by means of shear connectors, the deformations of the slab and the steel beam at the junction are the same, an the stress pattern will be as shown in Fig.4.7 The tensile stress at the bottom of the steel beam is now smaller than for the non-composite steel beam, because the section modulus for the composite section is larger than for the non-composite section. The stress at the bottom of the slab is usually compressive. The deflection of the composite section is much less than that for the corresponding non-composite section due to increased moment of inertia.

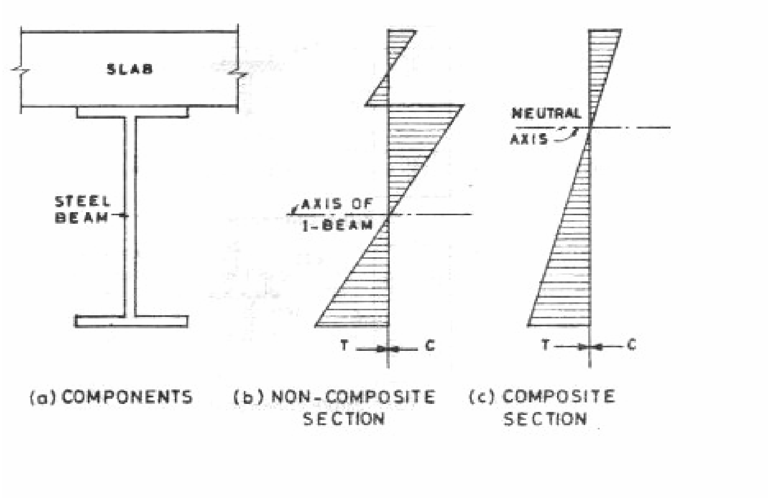


Fig 4.7.Stress Block of Composite Section (from [23])

4.7.1 ELASTIC ANALYSIS

Elastic design methods require that stresses in the beam must nowhere exceed specified working stress values, theses values being based on steel yield or concrete crushing stresses reduced by a suitable factor of safety.

Transformed Area Method

For an elastic analysis the following assumption are generally made:

a) The shear connection between beam and slab is sufficient to ensure that slip does not significantly affect the assumption of full interaction.
b) Both steel and concrete are linearly elastic materials.
c) Concrete undergoing tensile strain is in effective in resisting load

The transformed area method can usefully be employed so that the composite beam can be treated as if composed of a single material. This effect is achieved by dividing the actual concrete area (the product of effective width and depth in compression) by the relevant modular ratio m = Es/Ec. In cases where the steel beam is relatively small in depth in relation to slab, the composite neutral axis may fall within the slab. The concrete below the neutral axis is then cracked in tension and thus ineffective. When this occurs the effective area of concrete is a function of the neutral axis position:

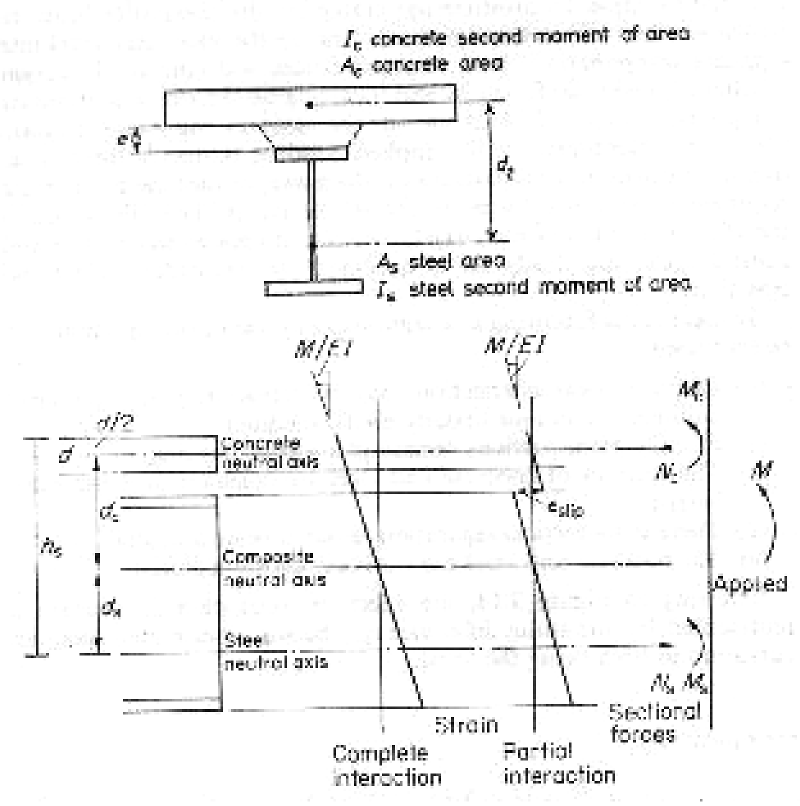


Fig. 4.8. Strain diagram for complete and partial interaction [25]

Case 1 - Composite neutral axis below slab (Fig. 4.8)

Actual area of concrete $A_c = bd$ (4.26)

Transformed area $= A/m$ (4.27)

Total composite area $A_t = A_s + A_c/m$ (4.28)

$A_s\, d_t = A_t\, d_c$ (moments of area about neutral axis of slab) (4.29)

$d_c, \quad = d_t\,(A_s / A_t)$ (4.30)

Case 2 - Composite neutral axis in slab

Refer to reference [25]

Having determined the depth of the neutral axis the remaining section properties can be calculated.

Second moment of area of composite section

$I_t = I_s + I_c/m + A_c\, d_c^2 /m + A_s\, d_s^2$

Put $S_t = A_c\, d_c /m = A_s\, d_s$. (Moment of steel or equivalent concrete area about composite neutral axis)

$I_t = I_s + I_c /m + S_t\, d_c + S_t\, d_s$

$$= I_s + I_c /m + S_t\, d_t \qquad (4.31)$$

The stress at any level can then be computed from $\sigma = My/ I_t$ bearing in mind that concrete stresses are obtained from $My/ I_t m$.

5.0 CONSTRUCTION LOADING

As part of design, the design engineer will select a cross-sectional geometry for a bridge structure which complies with the functional requirements of the highway. The position of the deck relative to the centerline of individual box girders and the completed box system will generally be established at an early stage in the design process.

Two examples of typical sectional geometry are given, together with various load cases in schematic form, as Fig. 5.1; both concentric and eccentric loading of the girders by the concrete deck are illustrated. For the idealized concentric loading case, the weight of the deck will be equally divided between the four inclined webs. Since the box girders are loaded along the centerline, twist about the shear centre will not occur and the flexural and torsional loadings are easily identified. Bracing of the compression flanges is required to avoid lateral movement or buckling due to the action of a combination of vertical load and horizontal force.

Geometric design constraints may result in the deck concrete being eccentric with respect to the longitudinal centerline of individual girders (Fig. 5.1). The case of an eccentric deck concrete results in a uniformly distributed load, w_c located at an eccentricity, e, with respect to the centerline acting on the individual box girders. Accordingly, flexural and torsional loading develops, giving rise to vertical deflection and horizontal deflection as a consequence of rotation of the girder about the shear centre. For both concentric and eccentric loading, end reactions provide a restraint for shear due to the vertical loading.

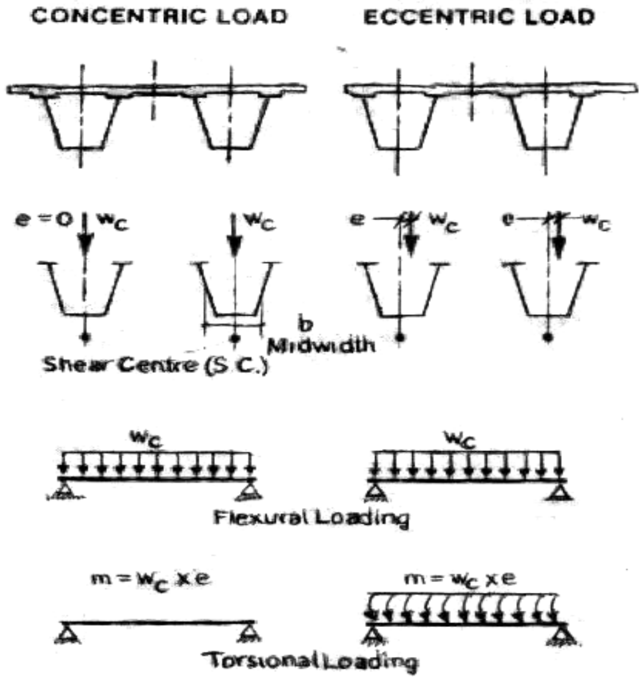


Fig.5.1. Basic Loading (from [24])

Other loading can give rise to predictable torsional effects during construction; these include wind loading of the exposed girders, the finishing machine, and form work (Fig 5.2). in each case adequate torsional restraints must be provided, not only to ensure the overall stability of the girders but also to limit the lateral deflections of the individual girders to an acceptable level. Calculation of the torsional loads and moment due to wind, equipment and formwork is possible at the design stage, with the use of simplifying assumptions.

P=Machine load
W_w=Wind load
W_f=Formwork load
e_w, e_p, w_f= Eccentricities

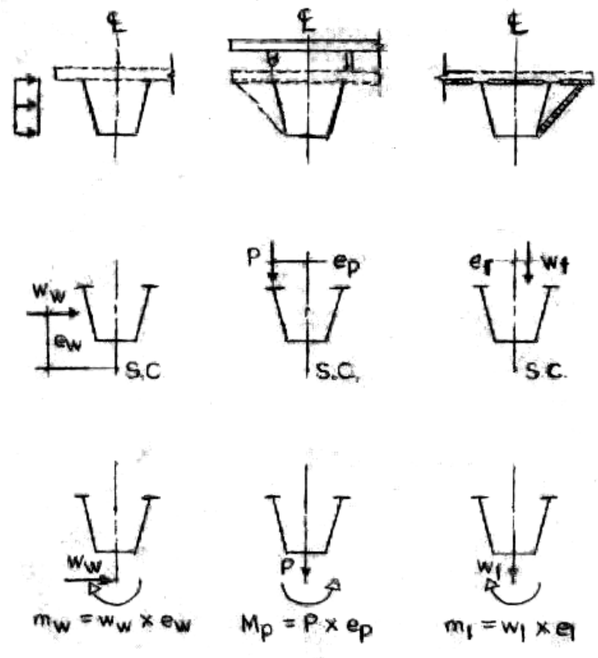


Fig 5.2. Construction Loading (from [24])

The flexural loading due to the concrete and formwork leads to shear and bending moment diagrams having a form familiar to designers. The concrete, formwork and wind loads combine to give torque and warping moment diagrams similar to those for bending. (Fig. 5.3)

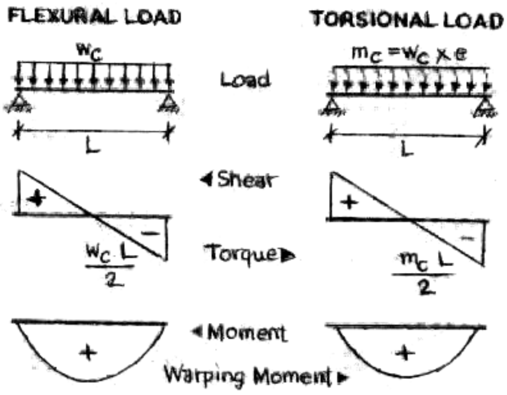


Fig 5.3. Flexural and Torsional Loads (from [24])

5.1 Effect of Loading on Box Girder

An arbitrary line loading on a simple-span box girder (Fig. 5.4 a) has and torsional components. Under the gravity load, the section effects rigidly (longitudinal bending) and deforms (bending distortion) (Fig 5.4 b) Under the torsional loading, the section rotates rigidly (mixed torsion) and deforms (torsional distortion) (Fig 5.4 c) (Branco, 1981; Branco and Green, 1984a)

P=Eccentric Loading
P_C=Compression Force
P_T=Tort ional Force
e= Eccentricity

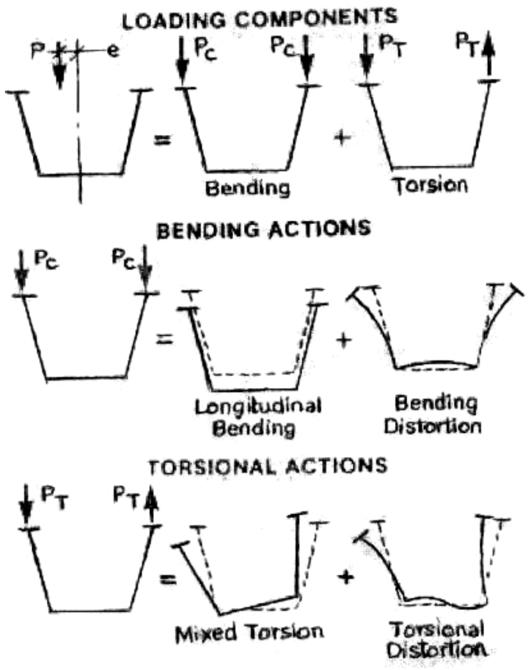


Fig. 5.4. General behaviour of open section. (from [24])

5.1.1 LONGITUDINAL BENDING

Pen a girder is subjected to transverse loads acting through the shear center bending normal stresses arise. Associated shear stresses also occur due to the variation of the bending moment along the beam. These can be calculated from the equilibrium considerations. Such analyses are based on the Navier hypothesis and neglect shear strains due to bending which induce warping displacements in the cross-section. These warping displacements are almost linear in the webs and vary parabolically in the flanges, giving a redistribution of the bending normal stresses, the shear lag effect (Fig. 5.5). This effect is usually small in box girders, except at sections where large concentrated loads or thin, wide girder flanges are present (Trahair, 1977).

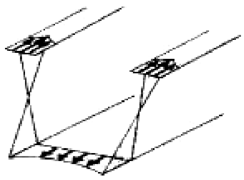


Fig 5.5. Shear Lag. (from [24])

5.1.2 MIXED TORSION

When gravity loads-do not act through the shear centre a girder will twist about the longitudinal axis. Uniform torsion and the associated St Venant shear stresses will occur if the rate of change of the angle of twist is constant along the girder and warping is unrestrained. It' there 'is a variation of girder, non uniform torque or if warping is prevented along the girder, non-uniform torsion (mixed torsion) occurs and longitudinal tortional warping stresses develop (Fig.5.6). The variation of these stresses induces warping shear stresses in the cross-section which add to the St Venant shear stresses so as to resist the total torque in the section. In the open box girders, the torque is mainly resisted by warping torsion because St Venant stiffness is comparatively small.

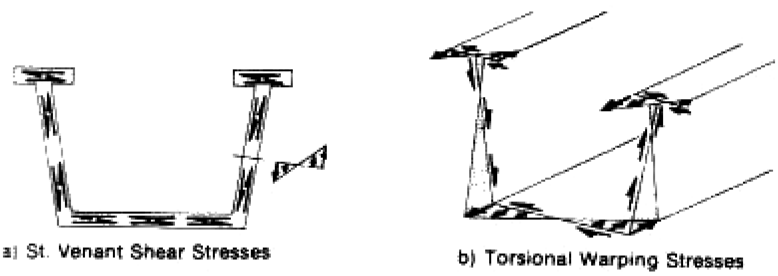


Fig.5.6. Torsional and warping stresses (from [24])

5.1.3 BENDING DISTORTION

Due to the transverse bending of the plates forming the girder, the cross-sectional shape changes in the presence of external loads. Bending distortion (spreading) occurs when transverse loads are applied concentrically through the shear centre. In open box girders, this distortion causes outward bending of the webs upward bending of the bottom flange and in-place bending of the top flanges (Fig. 5.4 b). Transverse bending stresses occur in the bottom flange and in the webs. In the top flanges due to the in-plane, bending, longitudinal spreading stresses arise (Fig.5.6).

5.1.4 TORSIONAL DISTORTION

Torsional load tends to deform the cross-section with bending of the walls (Fig. 5.4c). If the load is uniform along a girder having no diaphragms, uniform distortion is restrained only by the transverse stiff ness of the plate elements. Under non–uniform distortion, the plate elements also bend in the plane of the element, with shear stresses resisting distortional load. The additional vertical deflection of the webs increases the twist of the cross-section and results in additional distortional warping stresses (Fig. 5.7).

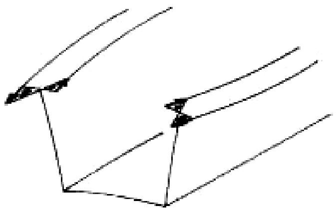


Fig 5.6. Spreading stresses (from [24])

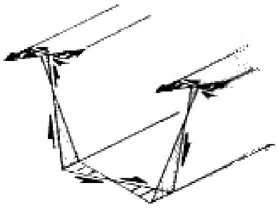


Fig 5.7. Distortional warping (from [24])

5.2 Bracing Systems

If distortional stiffness is lacking, the cross-section of the open deform, or if the torsional stiffness is small, excessive twist can Bracing systems are usually installed within the box girder to distortion within the section or increase the torsional stiffness, or minimizing these effects (Fig. 5.8).

Ties can usually be placed between top flanges to prevent bending distortion, and distortional braces, connecting top and bottom flanges are used if torsional distortion of the cross-section is to be prevented. Neither ties nor distortional braces add to the St Venant torsional stiffness of girder.

Horizontal bracing can be placed slightly below the top flanges to crease the torsional stiffness or to reduce twist. This bracing can be limited to the girder ends (torsional boxes), or be provided throughout the length of the girder (top chord bracing). A torsion box (Fig. 5.8) provides warping restraint at the supports and effectively changes an open section box from a torsional simple span to a torsional fixed span, so reducing the midspan rotation to one-fifth that of a simple span system. A distortional brace must be included as part of a torsion box.

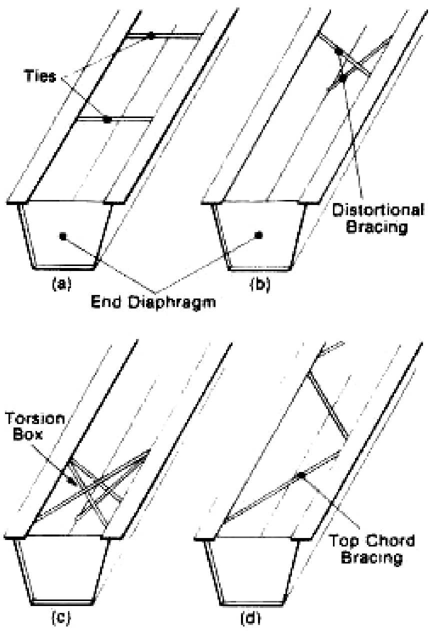


Fig. 5.8. Bracing systems (from [24])

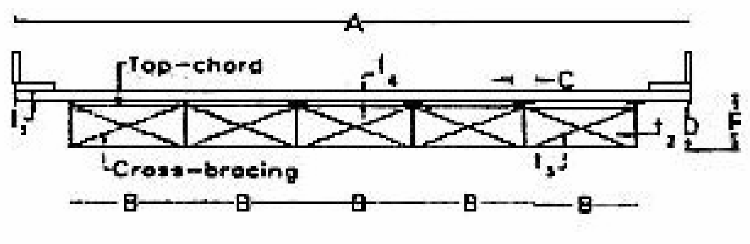
Top chord bracing (Fig 5.8) converts an open box to a quasi-closed box with calculable properties (Kollbruner and Basler, 1969). Depending upon the area and geometry of bracing, the quasi-closed box can be 10 to 20 times stiffer than an open box. Here, stiffness refers to the torque per unit midspan rotation rather than St Venant torsional stiffness.

Stay-in-place metal or wood forms should not be used to close the cross-section instead of top chord bracing, until connection details capable of transferring the shear force between components are developed. Interconnecting bracing systems between adjacent girders, or even shoring, where site conditions are suitable, can also be used to prevent rotation. [24]

6.0 STAAD.Pro ANALYSIS OF BRIDGE PROTOTYPES

6.1 Description of Bridge prototypes

In this present study, 20 simply supported single span bridges of different configuration were used. The basic cross-sectional configurations for the bridges, with span-to-depth ratio of 25 are presented in Table 1. The symbols used in the first column in Table 1 represent designation of the bridge types considered; L stands for lane, c stands for cell, and the number at the end of the designation represents the span length in meters. For example, 2L-3c-80 denotes a simply supported bridge of two-lane, three-cell and 80 m span. The cross sectional symbols used in Table 1 are shown in Fig.6.1.The number of lanes were taken as 1, 2, 3, and 4. Five different span lengths of 20, 40, 60, 80 and 100 m were included, representing the range for medium span bridges. Number of cells ranged from 2 to 3 in case of one-lane, 2 to 7 in case of two-lane, 3 to 7 in case of three-lane, and 5 to 7 in case of four-lane bridges. When changing the number of cells for the same bridge width, the thickness of the top steel flanges web and bottom flanges were altered to maintain the same shear stiffness of the webs and overall flexural stiffness of the cross section. The bridge width was taken 6.8 m in case of one-lane, 9.3 m in case of two-lane, 13.05 m in case of three-lane, 16.8 m in case of four-lane bridges. The ranges of the parameters considered in this study were based on an extensive survey of actual designed composite box girder bridges (Heins 1978). The moduli of elasticity of concrete and steel were taken as 25 and 205 GPa, respectively. Poisson’s ratio was assumed as 0.17 for concrete and 0.25 for steel. Solid plate diaphragms were provided at the supports. The material and thickness for the end diaphragms were taken to be the same as those for the webs.



A=Concrete deck width

B=Steel cell width

C=Top flange width
D=Cell depth
F=Over all depth of Bridge

t_1=thickness of top flange

t_2=thickness of web

t_3=thickness of bottom flange t_4=thickness of concrete deck slab

Fig.6.1 Cross-section symbols for five-cell bridge

<u>Table 1</u>

<u>Geometries of prototype bridges in Parametric Study</u>

Bridge Type	Span, L (m)	Number of lanes	Number of cells	Cross Section Dimensions (mm)								
				A	B	C	D	F	t_1	t_2	t_3	t_4
1L-2c-20	20	1	2	6800	2270	300	800	1025	16	10	10	225
2L-3c-20	20	2	3	9300	2325	300	800	1025	16	10	10	225
3L-5c-20	20	3	5	13050	2175	300	800	1025	16	10	10	225
4L-7c-20	20	4	7	16800	2100	300	800	1025	16	10	10	225
1L-2c-40	40	1	2	6800	2270	375	1600	1825	28	14	12	225
2L-3c-40	40	2	3	9300	2325	375	1600	1825	28	14	12	225
3L-5c-40	40	3	5	13050	2175	375	1600	1825	28	14	12	225
4L-7c-40	40	4	7	16800	2100	375	1600	1825	28	14	12	225
1L-2c-60	60	1	2	6800	2270	450	2400	2625	40	18	15	225
2L-3c-60	60	2	3	9300	2325	450	2400	2625	40	18	15	225
3L-5c-60	60	3	5	13050	2175	450	2400	2625	40	18	15	225
4L-7c-60	60	4	7	16800	2100	450	2400	2625	40	18	15	225
1L-2c-80	80	1	2	6800	2270	530	3200	3425	52	22	17	225
2L-3c-80	80	2	3	9300	2325	530	3200	3425	52	22	17	225
3L-5c-80	80	3	5	13050	2175	530	3200	3425	52	22	17	225
4L-7c-80	80	4	7	16800	2100	530	3200	3425	52	22	17	225
1L-2c-100	100	1	2	6800	2270	600	4000	4225	64	26	20	225
2L-3c-100	100	2	3	9300	2325	600	4000	4225	64	26	20	225
3L-5c-100	100	3	5	13050	2175	600	4000	4225	64	26	20	225
4L-7c-100	100	4	7	16800	2100	600	4000	4225	64	26	20	225

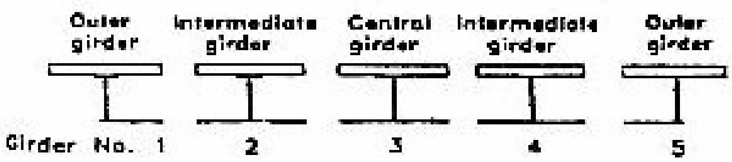


Fig.6.2 Idealized cellular bridge for moment distribution

6.2 Bridge Modeling

The modeling of each prototype bridge was carried out using finite-element method, with STAAD. Pro software. A four-node shell element with six degrees of freedom at each node was used to model the concrete deck slab, steel webs, steel bottom flange, and end diaphragms(For Figure Refer Section 6.8). A three-dimensional two-node beam element was adopted to model the steel top flanges. (For Figure Refer Section 6.8). Two different constraints were used in the modeling, namely

1) The roller support at one end of the bridge, constraining both vertical and lateral displacements at the lower end nodes of each end webs.

2) The hinge support at the other end of the bridge, restricting all possible transnational at the lower end nodes of each end webs.

The shell nodes of the concrete deck slab and beam element nodes of the steel top flanges ensuring the full interaction between the two as shear connectors were considered.

The parametric study was based on the following assumptions;

(1) The reinforced concrete slab deck is composite with the top steel flange of the cells

(2) All materials are elastic and homogeneous

(3) Outer curbs and railing are ignored and

(4) The concrete deck slab is considered uncracked.

6.3 Loading Conditions

In present study only live load is considered which is wheel load. The dead load of the bridge was not considered in this study. Live load included IRC wheel load of Class AA, Class A, and Class (70R + Bogie load). These loads were moved along the span length of the bridge at different critical span point, which varies along the width of the bridge also. Among these loads two types of conditions were considered one is fully loaded and other is partially loaded. In partially loaded cases the wheel loads close to the curbs were applied at a distance of 1.2 m (carriageway widths are over 5.5 m as per IRC code) from the inside edge of the curb. The impact factors of the moving load were also considered which varies according to the span length. The loads were applied on a simply supported girder, with a span equal to that of the bridge prototype, to determine which case produced the maximum moment and maximum shear force. It was seen after moving the loads along the span that Class 70R gives critical cases of maximum moment and maximum shear. Fig 6.2(a,b)

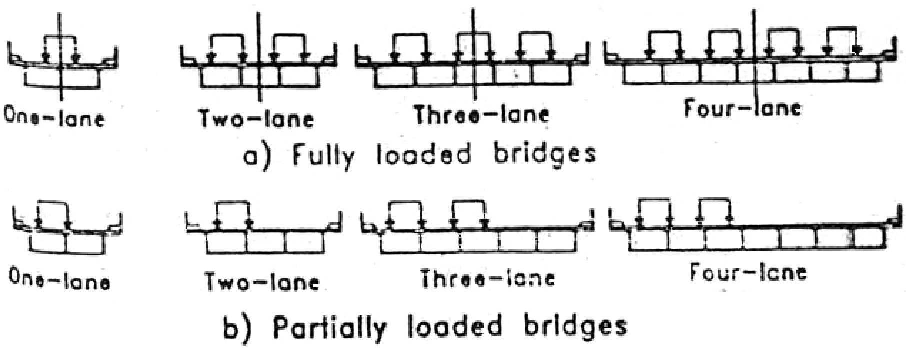


Fig.6.3 **IRC Wheeled Loading Cases Considered in Parametric Study**

Table 2

IRC CLASS A LOADING									
18500	**1100**	**3200**	**1100**	**4300**	**3000**	**3000**	**3000**	**Spacing b/w wheels (mm)**	
27	**27**	**114**	**114**	**68**	**68**	**68**	**68**	**←←←←←←Axel load (KN)**	
								Span length (L)	**Impact factor (I)**
31.67	31.67	133.7	133.7	79.8	79.8	79.8	79.8	20	0.173
29.62	29.62	125.1	125.1	74.6	74.6	74.6	74.6	40	0.097
29.4	29.4	124	124	74	74	74	74	60	0.088
29.4	29.4	124	124	74	74	74	74	80	0.088
29.4	29.4	124	124	74	74	74	74	100	0.088

Table 3

IRC CLASS 70R LOADING								
910	**3960**	**1520**	**2130**	**1370**	**3050**	**1370**	**910**	**Spacing b/w wheels (mm)**
80	**120**	**120**	**170**	**170**	**170**	**170**	**←←←←←←Axel load (KN)**	
							Span length (L)	**Impact factor (I)**
93.2	139.8	139.8	198.1	198.1	198.1	198.1	20	0.165
87.2	130.8	130.8	185.3	185.3	185.3	185.3	40	0.09
87	130.6	130.6	185	185	185	185	60	0.088
87	130.6	130.6	185	185	185	185	80	0.088
87	130.6	130.6	185	185	185	185	100	0.088

Bogie Loading for Class 70R			
1830	**1220**	**1830**	**Spacing b/w wheels (mm)**
200	**200**	**←←←←←←Axel load (KN)**	
		Span length (L)	**Impact factor (I)**
233	233	20	0.165
218	218	40	0.09
217.6	217.6	60	0.088
217.6	217.6	80	0.088
217.6	217.6	100	0.088

6.4 Load Distribution Factors

The cellular cross section was divided into I-beam shape girders as shown in Fig.6.2. Each girder consisted of the web, steel top flange, concrete deck slab, and steel bottom flange. In order to determine the moment distribution factor, $D_{MS,}$ carried by each girder of the bridge, the maximum moment M was calculated in a simply supported girder

subjected to line of wheel loads of an IRC Class 70R. The longitudinal moment carried by each girder of the prototype bridge, $M_{max,}$,determined from finite-element analysis of the loaded bridge prototype by the help of STAAD.Pro. The moment distribution factor, D_{MS}, was then calculated from the following relationship:

$$D_{MS} = \frac{M_{\max}}{M} \tag{6.1}$$

To calculate the shear distribution factors, D_{SS}, carried by each web the maximum reaction force, V, in a simply supported girder, subjected to a line of wheel loads of an IRC Class 70R. The maximum reaction under each web, V_{max},was obtained for each bridge prototype from finite-element analysis bridge prototype by the help of STAAD.Pro. the shear distribution factor, D_{SS}, was then determined as follows:

$$D_{SS} = \frac{V_{\max}}{V} \tag{6.2}$$

6.5 Moment & Shear of Prototype Cellular Bridge (M_{max}) &(V_{max})

These moments and shear data's are generated from STAAD.Pro analysis of 20 prototype composite multi-cell box girder bridge data's of varying parameters.

Table 4

Fully Loaded

1L-2c-20	Girder 1	Girder 2	Girder 3
Max M(KNm)	2628.82	2670.05	2628.82
Shear(KN)	766.64	790.37	790.37

1 L -2c-40	Girder 1	Girder 2	Girder 3
Max M(KNm)	7941.899	8070.436	7941.899
Shear(KN)	1182.7	1219.1	1219.1

1 L -2c-60	Girder 1	Girder 2	Girder 3
Max M(KNm)	12687.45	12934.46	12687.45
Shear(KN)	1376.72	1419.35	1419.35

1 L -2c-80	Girder 1	Girder 2	Girder 3
Max M(KNm)	17302.757	17573.94	17302.757
Shear(KN)	1392.799	1437.542	1437.542

1 L -2c-100	Girder 1	Girder 2	Girder 3
Max M(KNm)	21859.499	22202.837	21859.499
Shear(KN)	1422.604	1466.399	1466.399

1 L -3c-20	Girder 1	Girder 2	Girder 3	Girder 4
Max M(KNm)	873.425	1803.563	1803.563	873.425
Shear(KN)	252.78	538.941	538.941	252.78

1 L -3c-40	Girder 1	Girder 2	Girder 3	Girder 4
Max M(KNm)	2712.17	5385.23	5385.23	2712.17
Shear(KN)	439.5611	872.63	872.63	439.5611

1 L -3c-60	Girder 1	Girder 2	Girder 3	Girder 4
Max M(KNm)	4327.715	8592.38	8592.38	4327.715
Shear(KN)	481.356	955.68	955.68	481.356

1 L -3c-80	Girder 1	Girder 2	Girder 3	Girder 4
Max M(KNm)	5895.78	11706.67	11706.67	5895.78
Shear(KN)	481.49	956.05	956.05	481.49

1 L -3c-100	Girder 1	Girder 2	Girder 3	Girder 4
Max M(KNm)	7441.72	14775.68	14775.68	7441.72
Shear(KN)	491.66	975.471	975.471	491.66

2 L -2c-20	Girder 1	Girder 2	Girder 3
Max M(KNm)	2752.41	3306.2	2752.41
Shear(KN)	815.53	979.69	815.53

2 L -2c-40	Girder 1	Girder 2	Girder 3
Max M(KNm)	8297.29	9975.91	8297.29
Shear(KN)	1257.87	1511.08	1257.87

2 L -2c-60	Girder 1	Girder 2	Girder 3
Max M(KNm)	13283.33	15988.36	13283.33
Shear(KN)	1464.53	1759.34	1464.53

2 L -2c-80	Girder 1	Girder 2	Girder 3
Max M(KNm)	18115.43	21714.828	18115.43
Shear(KN)	1481.853	1781.92	1481.853

2 L -2c-100	Girder 1	Girder 2	Girder 3
Max M(KNm)	22887.86	27384.373	22887.86
Shear(KN)	1513.077	1817.659	1513.077

2 L -3c-20	Girder 1	Girder 2	Girder 3	Girder 4
Max M(KNm)	1745.25	2945.815	2945.815	1745.25
Shear(KN)	510.775	852.79	852.79	510.775

2 L -3c-40	Girder 1	Girder 2	Girder 3	Girder 4
Max M(KNm)	5421.12	9368.79	9368.79	5421.12
Shear(KN)	730.83	1518.16	1518.16	730.83

2 L -3c-60	Girder 1	Girder 2	Girder 3	Girder 4
Max M(KNm)	8357.8	14613.526	14613.526	8357.8
Shear(KN)	780.71	1601.92	1601.92	780.71

2 L -3c-80	Girder 1	Girder 2	Girder 3	Girder 4
Max M(KNm)	11727.17	20366.47	20366.47	11727.17
Shear(KN)	821.96	1663.28	1663.28	821.96

2 L -3c-100	Girder 1	Girder 2	Girder 3	Girder 4
Max M(KNm)	14776.56	25706.7	25706.7	14776.56
Shear(KN)	838	1698.175	1698.175	838

2 L -5c-20	Girder 1	Girder 2	Girder 3	Girder 4	Girder 5	Girder 6
MaxM(KNm)	1180.94	1816.07	2095.3	2095.3	1816.07	1180.94
Shear(KN)	422.88	650.26	750.34	750.34	650.26	422.88

2 L -5c-40	Girder 1	Girder 2	Girder 3	Girder 4	Girder 5	Girder 6
MaxM(KNm)	3715.08	5713.22	6578.25	6578.25	5713.22	3715.08
Shear(KN)	585.86	900.94	1039.44	1039.44	900.94	585.86

2 L -5c-60	Girder 1	Girder 2	Girder 3	Girder 4	Girder 5	Girder 6
MaxM(KNm)	5928.078	9116.765	10519.84	10519.84	9116.765	5928.078
Shear(KN)	644.03	990.27	1142.72	1142.72	990.27	644.03

2 L -5c-80	Girder 1	Girder 2	Girder 3	Girder 4	Girder 5	Girder 6
Max +ve M(KNm)	8090.165	12440.1	14182.143	14182.143	12440.1	8090.165

Shear(KN)	667.68	1026.77	1184.82	1184.82	1026.77	667.68

2 L -5c-100	Girder 1	Girder 2	Girder 3	Girder 4	Girder 5	Girder 6
Max +ve M(KNm)	10233.45	15739.02	18161.59	18161.59	15739.02	10233.45
Shear(KN)	680.79	1046.88	1208.1	1208.1	1046.88	680.79

2 L -7c-20	Girder 1	Girder 2	Girder 3	Girder 4	Girder 5	Girder 6	Girder 7	Girder 8
MaxM(KNm)	933.22	1029.9	1830.76	1988.5	1988.5	1830.76	1029.9	933.22
Shear(KN)	281.81	443.29	646.77	702.47	702.47	646.77	443.29	281.81

2 L -7c-40	Girder 1	Girder 2	Girder 3	Girder 4	Girder 5	Girder 6	Girder 7	Girder 8
MaxM(KNm)	2916.46	3917.48	5715.62	6207.41	6207.41	5715.62	3917.48	2916.46
Shear(KN)	442.354	609.41	889.1	965.6	965.6	889.1	609.41	442.354

2 L -7c-60	Girder 1	Girder 2	Girder 3	Girder 4	Girder 5	Girder 6	Girder 7	Girder 8
MaxM(KNm)	4738.87	6274.68	9155.28	9942.18	9942.18	9155.28	6274.68	4738.87
Shear(KN)	524.41	678.13	989.28	1074.53	1074.53	989.28	678.13	524.41

2 L -7c-80	Girder 1	Girder 2	Girder 3	Girder 4	Girder 5	Girder 6	Girder 7	Girder 8
Max +ve M(KNm)	6520.51	8638.89	12603.83	13689.14	13689.14	12603.83	8638.89	6520.51
Shear(KN)	540.233	708.52	1033.7	1122.61	1122.61	1033.7	708.52	540.233

2 L -7c-100	Girder 1	Girder 2	Girder 3	Girder 4	Girder 5	Girder 6	Girder 7	Girder 8
MaxM(KNm)	8263.97	10939.77	15957.47	17331.494	17331.494	15957.47	10939.77	8263.97
Shear(KN)	556.47	735.4	1072.86	1165.34	1165.34	1072.86	735.4	556.47

3 L -5c-20	Girder 1	Girder 2	Girder 3	Girder 4	Girder 5	Girder 6
MaxM(KNm)	1596.83	2797.92	2797.92	2797.92	2797.92	1596.83
Shear(KN)	443.289	1001.85	1001.85	1001.85	1001.85	443.289

3 L -5c-40	Girder 1	Girder 2	Girder 3	Girder 4	Girder 5	Girder 6
MaxM(KNm)	4922.29	8800.51	8800.51	8800.51	8800.51	4922.29
Shear(KN)	760.97	1387.96	1387.96	1387.96	1387.96	760.97

3 L -5c-60	Girder 1	Girder 2	Girder 3	Girder 4	Girder 5	Girder 6
MaxM(KNm)	7844.67	14044.85	14044.85	14044.85	14044.85	7844.67
Shear(KN)	860.27	1525.76	1525.76	1525.76	1525.76	860.27

3 L -5c-80	Girder 1	Girder 2	Girder 3	Girder 4	Girder 5	Girder 6
MaxM(KNm)	10672.6	19143.54	19143.54	19143.54	19143.54	10672.6
Shear(KN)	883.165	1586.1	1586.1	1586.1	1586.1	883.165

3 L -5c-100	Girder 1	Girder 2	Girder 3	Girder 4	Girder 5	Girder 6
MaxM(KNm)	13441.76	24247.49	24247.49	24247.49	24247.49	13441.76
Shear(KN)	899.215	1613.04	1613.04	1613.04	1613.04	899.215

3 L -7c-20	Girder 1	Girder 2	Girder 3	Girder 4	Girder 5	Girder 6	Girder 7	Girder 8
MaxM(KNm)	842.73	1363.86	2715.97	2715.97	2715.97	2715.97	1363.86	842.73
Shear(KN)	297.72	481.898	959.44	959.44	959.44	959.44	481.898	297.72

3 L -7c-40	Girder 1	Girder 2	Girder 3	Girder 4	Girder 5	Girder 6	Girder 7	Girder 8
MaxM(KNm)	2630.38	4258.685	8478.92	8478.92	8478.92	8478.92	4258.685	2630.38
Shear(KN)	409.285	662.37	1319	1319	1319	1319	662.37	409.285

3 L -7c-60	Girder 1	Girder 2	Girder 3	Girder 4	Girder 5	Girder 6	Girder 7	Girder 8
MaxM(KNm)	4212.89	6820.24	13580.155	13580.155	13580.155	13580.155	6820.24	4212.89
Shear(KN)	455.356	737.1	1467.65	1467.65	1467.65	1467.65	737.1	455.356

3 L -7c-80	Girder 1	Girder 2	Girder 3	Girder 4	Girder 5	Girder 6	Girder 7	Girder 8
MaxM(KNm)	5800.1	9387.98	18697.57	18697.57	18697.57	18697.57	9387.98	5800.1
Shear(KN)	475.72	785.89	1564.98	1564.98	1564.98	1564.98	785.89	475.72

3 L -7c-100	Girder 1	Girder 2	Girder 3	Girder 4	Girder 5	Girder 6	Girder 7	Girder 8
MaxM(KNm)	7343.57	11885.63	23672.98	23672.98	23672.98	23672.98	11885.63	7343.57
Shear(KN)	493.94	798.67	2696.47	2696.47	2696.47	2696.47	798.67	493.94

4 L -7c-20	Girder 1	Girder 2	Girder 3	Girder 4	Girder 5	Girder 6	Girder 7	Girder 8
MaxM(KNm)	763.71	2734.122	2721.84	2734.122	2734.122	2721.84	2734.122	763.71
Shear(KN)	269.87	965.85	961.58	965.85	965.85	961.58	965.85	269.87

4 L -7c-40	Girder 1	Girder 2	Girder 3	Girder 4	Girder 5	Girder 6	Girder 7	Girder 8
MaxM(KNm)	2384.58	8535.79	8497.35	8535.79	8535.79	8497.35	8535.79	2384.58
Shear(KN)	370.85	1327.79	1321.69	1327.79	1327.79	1321.69	1327.79	370.85

4 L -7c-60	Girder 1	Girder 2	Girder 3	Girder 4	Girder 5	Girder 6	Girder 7	Girder 8
MaxM(KNm)	3817.78	13671.29	13609.68	13671.29	13671.29	13609.68	13671.29	3817.78
Shear(KN)	412.73	1477.46	1470.92	1499.62	1499.62	1470.92	1477.46	412.73

4 L -7c-80	Girder 1	Girder 2	Girder 3	Girder 4	Girder 5	Girder 6	Girder 7	Girder 8
MaxM(KNm)	5253.65	18831.71	18739.38	18831.71	18831.71	18739.38	18831.71	5253.65
Shear(KN)	431.26	1543.77	1536.84	1543.77	1543.77	1536.84	1543.77	431.26

4 L -7c-100	Girder 1	Girder 2	Girder 3	Girder 4	Girder 5	Girder 6	Girder 7	Girder 8
MaxM(KNm)	6649.99	23830.26	23723.22	23830.26	23830.26	23723.22	23830.26	6649.99
Shear(KN)	447.55	1602.32	1595.28	1602.32	1602.32	1595.28	1602.32	447.55

<u>Partially Loaded</u>

1 L -2c-20	Girder 1	Girder 2	Girder 3
Max M(KNm)	2587.71	2140.69	611.56
Shear(KN)	766.64	634.19	181.26

1 L -2c-40	Girder 1	Girder 2	Girder 3
Max M(KNm)	7845.19	6470.93	1848.68
Shear(KN)	1182.7	982.21	279.567

1 L -2c-60	Girder 1	Girder 2	Girder 3
Max M(KNm)	12488.6	10332.42	2949.29
Shear(KN)	1377	1139.17	325.49

1 L -2c-80	Girder 1	Girder 2	Girder 3
Max M(KNm)	17033.03	14089.81	4025.07
Shear(KN)	1393.23	1152.63	327.78

1 L -2c-100	Girder 1	Girder 2	Girder 3
Max M(KNm)	21518.17	17801.15	5084.01
Shear(KN)	1422.66	1176.94	336.29

2 L -3c-20	Girder 1	Girder 2	Girder 3	Girder 4
Max M(KNm)	2350.4	2113.46	1962.89	422.56
Shear(KN)	689.66	611.81	568.21	117.33

2 L -3c-40	Girder 1	Girder 2	Girder 3	Girder 4
Max M(KNm)	7259.32	6721.63	6242.65	507.7
Shear(KN)	1133.53	1396.13	1396.13	99.087

2 L -3c-60	Girder 1	Girder 2	Girder 3	Girder 4
Max M(KNm)	11567.51	10725.23	9960.15	836.422
Shear(KN)	1283.66	1192.88	1107.92	97.18

2 L -3c-80	Girder 1	Girder 2	Girder 3	Girder 4
Max M(KNm)	15722.66	14612.43	13572.42	1044.12
Shear(KN)	1295.087	1193.33	1108.322	87.03

2 L -3c-100	Girder 1	Girder 2	Girder 3	Girder 4
Max M(KNm)	19753.7	18443.62	17128.56	1223.56
Shear(KN)	1319.46	1218.48	1131.58	81.76

3 L -5c-20	Girder 1	Girder 2	Girder 3	Girder 4	Girder 5	Girder 6
MaxM(KNm)	4025.71	3776.84	3687.41	1908.17	1858.25	0
Shear(KN)	1481.22	1474.41	1320.37	683.25	665.45	0

3 L -5c-40	Girder 1	Girder 2	Girder 3	Girder 4	Girder 5	Girder 6
MaxM(KNm)	12258.166	11676.34	11601.85	6003.17	5846.18	0
Shear(KN)	2238.37	2114.03	1727.19	893.74	870.43	0

3 L -5c-60	Girder 1	Girder 2	Girder 3	Girder 4	Girder 5	Girder 6
MaxM(KNm)	19522.5	18564.72	18510.81	9498.02	9250.27	0
Shear(KN)	2579.55	2461.34	2010.96	1040.57	1013.434	0

3 L -5c-80	Girder 1	Girder 2	Girder 3	Girder 4	Girder 5	Girder 6
MaxM(KNm)	26515.35	25266.89	25214.63	13072.45	12706.61	0
Shear(KN)	2630	2514.98	2085	1078.88	1050.73	0

3 L -5c-100	Girder 1	Girder 2	Girder 3	Girder 4	Girder 5	Girder 6
MaxM(KNm)	33297.98	31827.56	31958.63	16536.35	16106	0
Shear(KN)	2675.77	2567.89	2125.9	1100	1071.39	0

4 L -7c-20	Girder 1	Girder 2	Girder 3	Girder 4	Girder 5	Girder 6	Girder 7	Girder 8
MaxM(KNm)	4719.22	4500.28	3588.64	1830.76	1417.78	0	0	0

Shear(KN)	1439.44	1437.94	1267.76	646.78	467.34	0	0	0

4 L -7c-40	Girder 1	Girder 2	Girder 3	Girder 4	Girder 5	Girder 6	Girder 7	Girder 8
MaxM(KNm)	14321.2	13725.19	11128.49	5715.62	4599.1	0	0	0
Shear(KN)	2299.88	2109.39	1731.1	889.1	723.26	0	0	0

4 L -7c-60	Girder 1	Girder 2	Girder 3	Girder 4	Girder 5	Girder 6	Girder 7	Girder 8
MaxM(KNm)	22819.44	21990.89	17825.32	9155.28	7364.53	0	0	0
Shear(KN)	2659.82	2410.1	1926.28	1033.6	785.83	0	0	0

4 L -7c-80	Girder 1	Girder 2	Girder 3	Girder 4	Girder 5	Girder 6	Girder 7	Girder 8
MaxM(KNm)	31008.53	30024.47	24542.198	12603.83	10217.21	0	0	0
Shear(KN)	2713.15	2477	2012.56	1033.7	833.37	0	0	0

4 L -7c-100	Girder 1	Girder 2	Girder 3	Girder 4	Girder 5	Girder 6	Girder 7	Girder 8
MaxM(KNm)	38986.14	37802.1	31076.1	15957.47	12999.71	0	0	0
Shear(KN)	2767.81	2540.73	2089.1	1072.86	874.11	0	0	0

6.6 Moment & Shear of Idealized Cellular Bridge (M) &(V)

Table 5

Span	***Maximum Moment (KNm)***		***Maximum Shear (KN)***
	End	Middle	
20	2669.394	2669.52	791.164
40	8058.751	8009.564	1220.288
60	12828.561	12836.899	1420.769
80	17401.948	17420.64	1443.316
100	21810.426	21844.586	1467.867

From above moments and shear data's that are obtain by the help of STAAD.Pro. the following moment and shear distribution factors were calculated , these distribution factors were further used to deduce common expressions for moment and shear distribution factors. The variation of distribution factors with respect to no. of cells, span length, and no. lanes were also shown with help of graphs.

6.7 Moment & Shear Distribution Factors

Table 6

Fully loaded

1 L -2c-20	Girder 1	Girder 2	Girder 3
Dms	0.9848	1.0002	.9848
Dss	.969	.999	.999

1 L -2c-40	Girder 1	Girder 2	Girder 3
Dms	0.9855	1.0076	.9855
Dss	.9692	.999	.999

1 L -2c-60	Girder 1	Girder 2	Girder 3
Dms	0.989	1.0039	.989
Dss	.969	.999	.999

1 L -2c-80	Girder 1	Girder 2	Girder 3
Dms	0.9943	1.0088	.9943
Dss	.9653	.996	.996

1 L -2c-100	Girder 1	Girder 2	Girder 3
Dms	1.00225	1.0164	1.00225
Dss	.9692	.999	.999

1 L -3c-20	Girder 1	Girder 2	Girder 3	Girder 4
Dms	0.3272	0.6756	0.6756	0.3272
Dss	0.3195	0.6812	0.6812`	0.3195

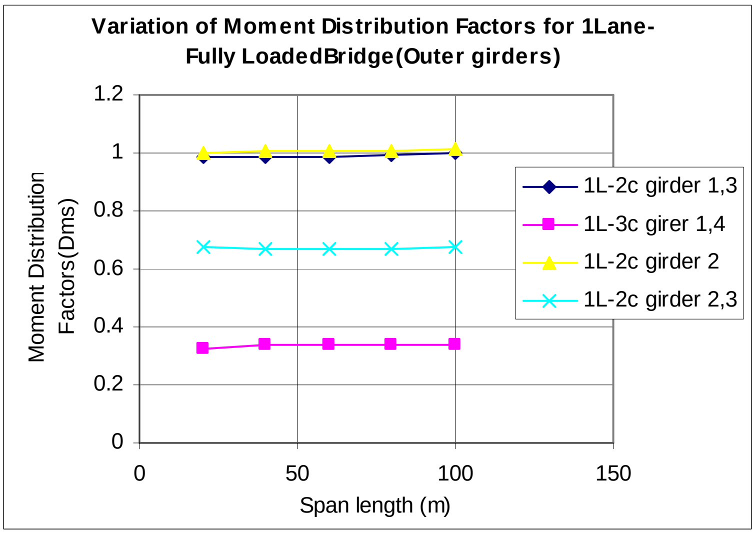
1 L -3c-40	Girder 1	Girder 2	Girder 3	Girder 4
Dms	0.33655	0.67235	0.67235	0.33655

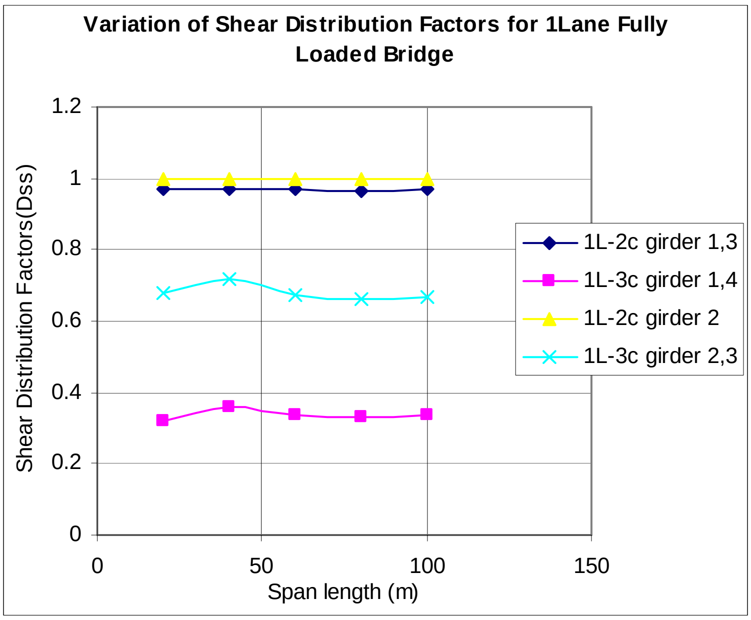
Dss	.36017	0.7151	0.7151	.36017

1 L -3c-60	Girder 1	Girder 2	Girder 3	Girder 4
Dms	0.33735	0.66935	0.66935	0.33735
Dss	.3388	0.67265	0.67265	.3388

1 L -3c-80	Girder 1	Girder 2	Girder 3	Girder 4
Dms	0.3388	0.672	0.672	0.3388
Dss	.3336	0.6624	0.6624	.3336

1 L -3c-100	Girder 1	Girder 2	Girder 3	Girder 4
Dms	0.3412	0.6764	0.6764	0.3412
Dss	.33495	0.66455	0.66455	.33495





2 L -2c-20	Girder 1	Girder 2	Girder 3
Dms	1.0311	1.2385	1.0311
Dss	1.0308	1.2383	1.0308

2 L -2c-40	Girder 1	Girder 2	Girder 3
Dms	1.0296	1.2455	1.0296
Dss	1.0308	1.2383	1.0308

2 L -2c-60	Girder 1	Girder 2	Girder 3
Dms	1.03545	1.2455	1.03545
Dss	1.0308	1.2383	1.0308

2 L -2c-80	Girder 1	Girder 2	Girder 3
Dms	1.041	1.2465	1.041
Dss	1.0267	1.2346	1.0267

2 L -2c-100	Girder 1	Girder 2	Girder 3
Dms	1.0494	1.2536	1.0494
Dss	1.0308	1.2383	1.0308

2 L -3c-20	Girder 1	Girder 2	Girder 3	Girder 4
Dms	0.6538	1.1035	1.1035	0.6538
Dss	.6456	1.0779	1.0779	.6456

2 L -3c-40	Girder 1	Girder 2	Girder 3	Girder 4
Dms	0.6727	1.1697	1.1697	0.6727
Dss	.5989	1.1441	1.1441	.5989

2 L -3c-60	Girder 1	Girder 2	Girder 3	Girder 4
Dms	0.6515	1.1384	1.1384	0.6515

Dss	.5495	1.1275	1.1275	.5495

2 L -3c-80	Girder 1	Girder 2	Girder 3	Girder 4
Dms	0.6739	1.1691	1.1691	0.6739
Dss	.5695	1.1524	1.1524	.5695

2 L -3c-100	Girder 1	Girder 2	Girder 3	Girder 4
Dms	0.6775	1.1768	1.1768	0.6775
Dss	.5709	1.1569	1.1569	.5709

2 L -5c-20	Girder 1	Girder 2	Girder 3	Girder 4	Girder 5	Girder 6
Dms	0.4424	0.6803	0.7849	0.7849	0.6803	0.4424
Dss	.5345	.8219	0.9484	0.9484	.8219	.5345

2 L -5c-40	Girder 1	Girder 2	Girder 3	Girder 4	Girder 5	Girder 6
Dms	0.461	.7133	0.8231	0.8231	.7133	0.461
Dss	.4801	.7383	.8518	.8518	.7383	.4801

2 L -5c-60	Girder 1	Girder 2	Girder 3	Girder 4	Girder 5	Girder 6
Dms	0.4621	.7102	0.8195	0.8195	.7102	0.4621
Dss	.4533	.697	.8043	.8043	.697	.4533

2 L -5c-80	Girder 1	Girder 2	Girder 3	Girder 4	Girder 5	Girder 6
Dms	0.4649	.7141	0.8141	0.8141	.7141	0.4649
Dss	.4626	.7114	.8209	.8209	.7114	.4626

2 L -5c-100	Girder 1	Girder 2	Girder 3	Girder 4	Girder 5	Girder 6
Dms	0.4692	.7205	0.8314	0.8314	.7205	0.4692
Dss	.4638	.7132	.823	.823	.7132	.4638

2 L -7c-20	Girder 1	Girder 2	Girder 3	Girder 4	Girder 5	Girder 6	Girder 7	Girder 8
Dms	.3496	.3858	0.6858	0.7449	.7449	.6858	.3858	.3496
Dss	.3562	.5603	.8175	.8879	.8879	.8175	.5603	. 3562

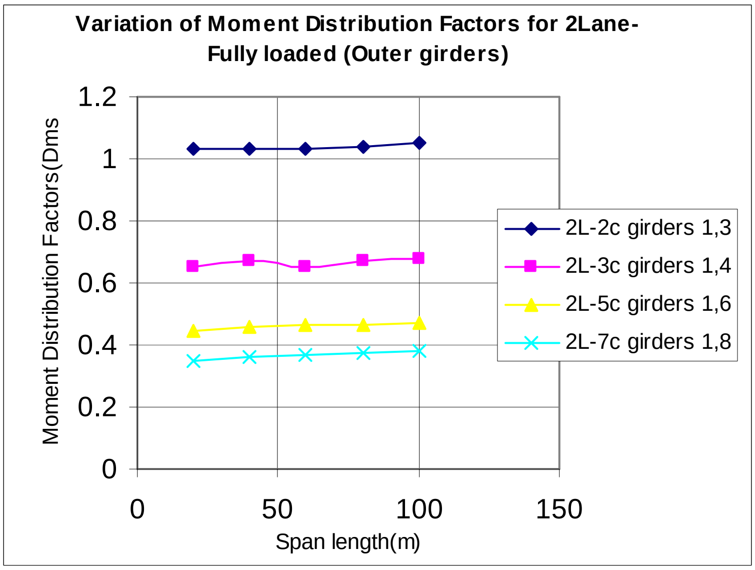
2 L -7c-40	Girder 1	Girder 2	Girder 3	Girder 4	Girder 5	Girder 6	Girder 7	Girder 8
Dms	.3619	.4891	0.7136	0.775	.775	0.7136	.4891	. 3619
Dss	.3625	.4994	.7286	.7913	.7913	.7286	.4994	. 3625

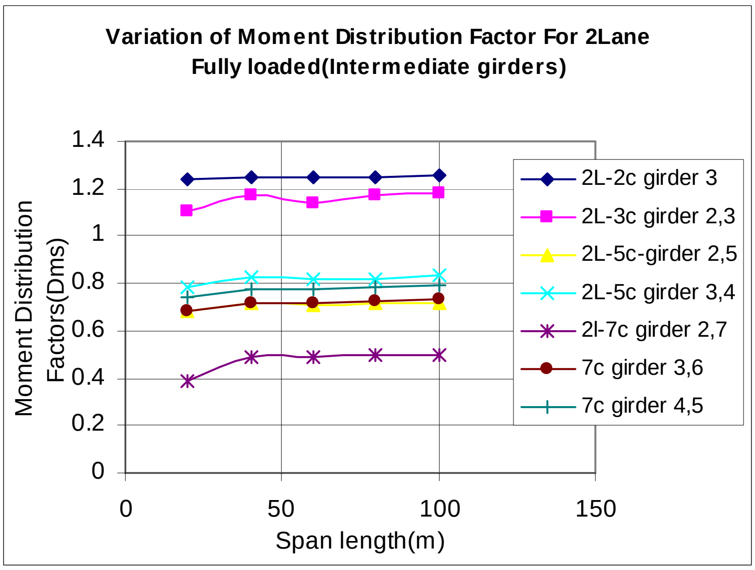
2 L -7c-60	Girder 1	Girder 2	Girder 3	Girder 4	Girder 5	Girder 6	Girder 7	Girder 8

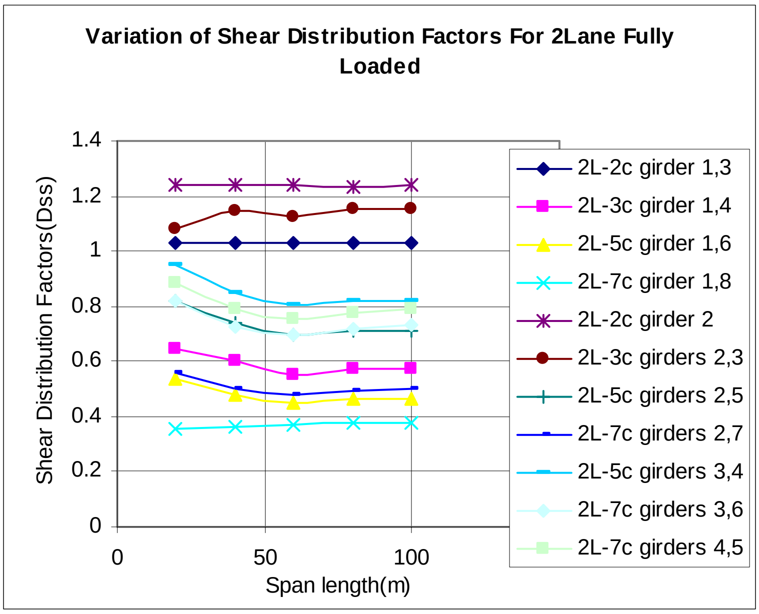
Dms	.3694	.4888	0.7132	0.7745	.7745	0.7132	.4888	. 3694
Dss	.3691	.4773	.6963	.7563	.7563	.6963	.4773	. 3691

2 L -7c-80	Girder 1	Girder 2	Girder 3	Girder 4	Girder 5	Girder 6	Girder 7	Girder 8
Dms	.3747	.4959	0.7235	0.7858	.7858	0.7235	.4959	.3747
Dss	.3743	.4909	.7162	.7778	. 7778	.7162	.4909	.3743

2 L -7c-100	Girder 1	Girder 2	Girder 3	Girder 4	Girder 5	Girder 6	Girder 7	Girder 8
Dms	.3789	.5008	0.7305	0.7934	.7934	0.7305	.5008	.3789
Dss	.3791	.501	.7309	.7939	. 7939	.7309	.501	. 3791







3 L -5c-20	Girder 1	Girder 2	Girder 3	Girder 4	Girder 5	Girder 6
Dms	0.5982	1.0481	1.0481	1.0481	1.0481	0.5982
Dss	.5603	1.2663	1.2663	1.2663	1.2663	.5603

3 L -5c-40	Girder 1	Girder 2	Girder 3	Girder 4	Girder 5	Girder 6
Dms	0.6108	1.09875	1.09875	1.09875	1.09875	0.6108
Dss	.6236	1.1374	1.1374	1.1374	1.1374	.6236

3 L -5c-60	Girder 1	Girder 2	Girder 3	Girder 4	Girder 5	Girder 6
Dms	0.6115	1.0941	1.0941	1.0941	1.0941	0.6115
Dss	.6055	1.0739	1.0739	1.0739	1.0739	.6055

3 L -5c-80	Girder 1	Girder 2	Girder 3	Girder 4	Girder 5	Girder 6
Dms	0.6133	1.0989	1.0989	1.0989	1.0989	0.6133
Dss	.6119	1.09675	1.09675	1.09675	1.09675	.6119

3 L -5c-100	Girder 1	Girder 2	Girder 3	Girder 4	Girder 5	Girder 6
Dms	0.6163	1.11	1.11	1.11	1.11	0.6163
Dss	.6126	1.0989	1.0989	1.0989	1.0989	.6126

3 L -7c-20	Girder 1	Girder 2	Girder 3	Girder 4	Girder 5	Girder 6	Girder 7	Girder 8
Dms	.3157	.5109	1.0174	1.0174	1.0174	1.0174	.5109	.3157
Dss	.3763	.6091	1.2127	1.2127	1.2127	1.2127	.6091	.3763

3 L -7c-40	Girder 1	Girder 2	Girder 3	Girder 4	Girder 5	Girder 6	Girder 7	Girder 8
Dms	.3264	.5317	1.0586	1.0586	1.0586	1.0586	.5317	.3264
Dss	.3354	.5428	1.0809	1.0809	1.0809	1.0809	.5428	.3354

3 L -7c-60	Girder 1	Girder 2	Girder 3	Girder 4	Girder 5	Girder 6	Girder 7	Girder 8
Dms	.3284	.5313	1.0579	1.0579	1.0579	1.0579	.5313	.3284
Dss	.3205	.5188	1.033	1.033	1.033	1.033	.5188	.3205

3 L -7c-80	Girder 1	Girder 2	Girder 3	Girder 4	Girder 5	Girder 6	Girder 7	Girder 8
Dms	.3333	.5389	1.0733	1.0733	1.0733	1.0733	.5389	.3333
Dss	.3296	.5445	1.0843	1.0843	1.0843	1.0843	.5445	.3296

3 L -7c-100	Girder 1	Girder 2	Girder 3	Girder 4	Girder 5	Girder 6	Girder 7	Girder 8
Dms	.3367	.5441	1.0837	1.0837	1.0837	1.0837	.5441	.3367
Dss	.3365	.5446	1.0843	1.0843	1.0843	1.0843	.5446	.3365

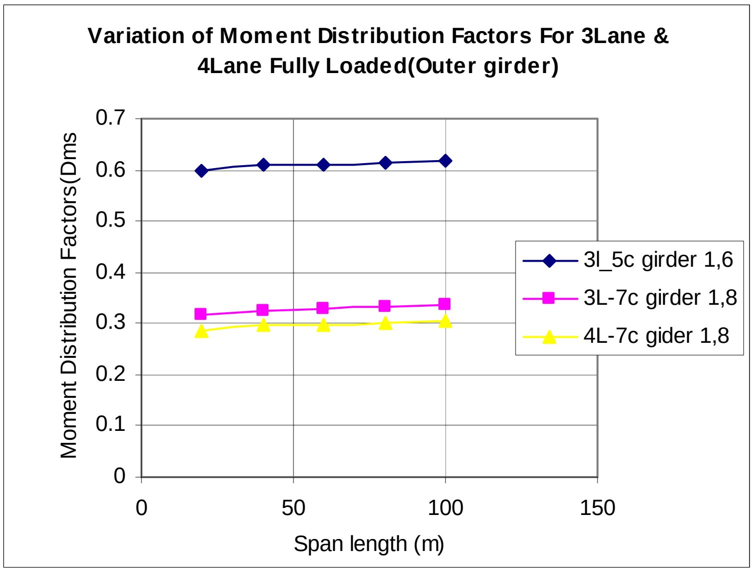
4 L -7c-20	Girder 1	Girder 2	Girder 3	Girder 4	Girder 5	Girder 6	Girder 7	Girder 8
Dms	.2861	1.0242	1.0196	1.0242	1.0242	1.0196	1.0242	.2861
Dss	.3411	1.2208	1.2154	1.2208	1.2208	1.2154	1.2208	.3411

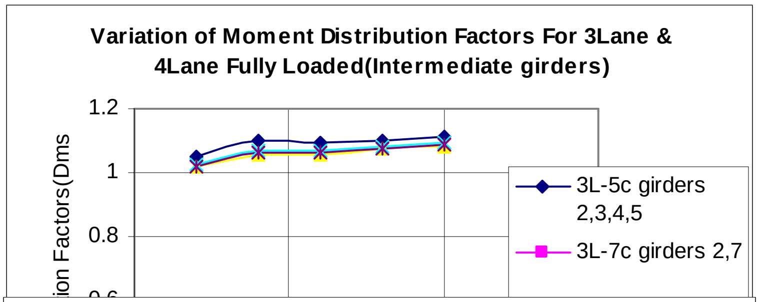
4 L -7c-40	Girder 1	Girder 2	Girder 3	Girder 4	Girder 5	Girder 6	Girder 7	Girder 8
Dms	.2959	1.0657	1.0609	1.0657	1.0657	1.0609	1.0657	.2959
Dss	.3039	1.0881	1.0831	1.0881	1.0881	1.0831	1.0881	.3039

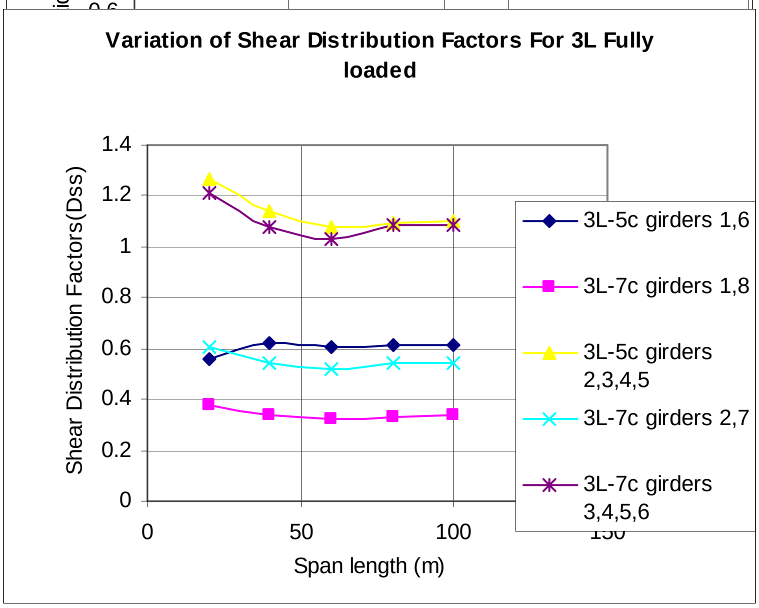
4 L -7c-60	Girder 1	Girder 2	Girder 3	Girder 4	Girder 5	Girder 6	Girder 7	Girder 8
Dms	.2976	1.065	1.0602	1.065	1.065	1.0602	1.065	.2976
Dss	.2905	1.0399	1.0353	1.0555	1.0555	1.0353	1.0399	.2905

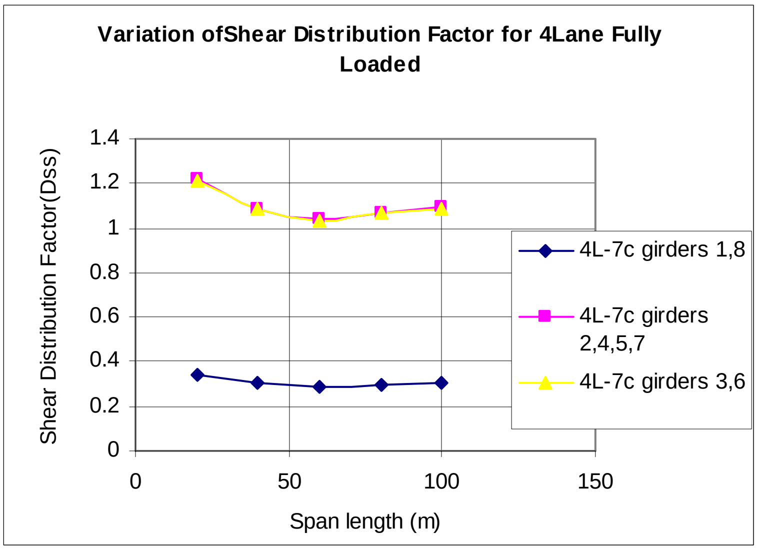
4 L -7c-80	Girder 1	Girder 2	Girder 3	Girder 4	Girder 5	Girder 6	Girder 7	Girder 8
Dms	.3019	1.081	1.0757	1.081	1.081	1.0757	1.081	.3019
Dss	.2988	1.0696	1.0648	1.0696	1.0696	1.0648	1.0696	.2988

4 L -7c-100	Girder 1	Girder 2	Girder 3	Girder 4	Girder 5	Girder 6	Girder 7	Girder 8
Dms	.3049	1.0909	1.086	1.0909	1.0909	1.086	1.0909	.3049
Dss	.3049	1.0916	1.0868	1.0916	1.0916	1.0868	1.0916	.3049









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1 L -2c-20	Girder 1	Girder 2	Girder 3
Dms	0.9694	.8019	.2291
Dss	.969	.8016	.2291

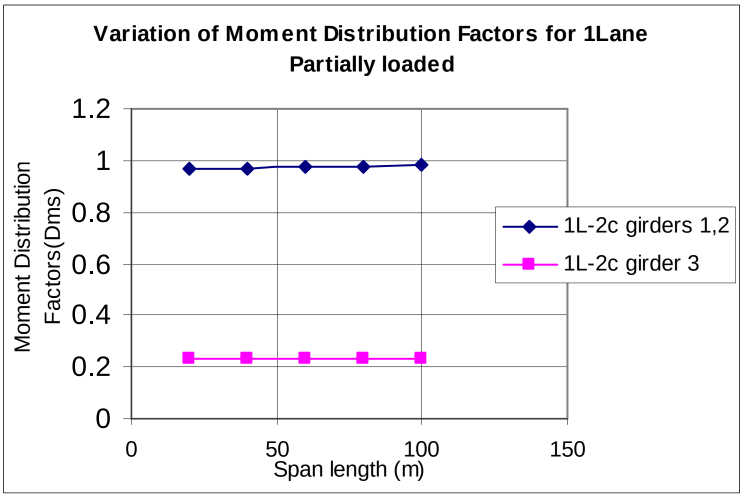
1 L -2c-40	Girder 1	Girder 2	Girder 3
Dms	0.971	.8079	.2294
Dss	.9692	.8018	.2291

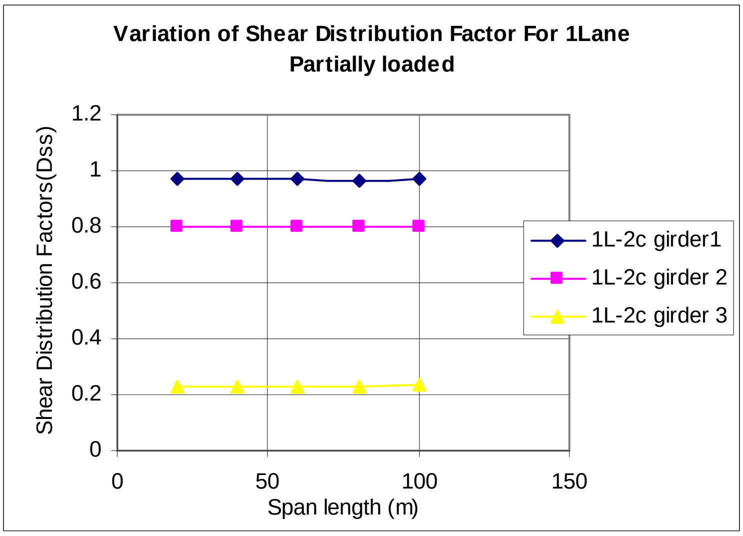
1 L -2c-60	Girder 1	Girder 2	Girder 3
Dms	0.9735	.8049	.2299
Dss	.9692	.8018	.2291

1 L -2c-80	Girder 1	Girder 2	Girder 3
Dms	0.9788	.8088	.2313
Dss	.9653	.7986	.2271

1 L -2c-100	Girder 1	Girder 2	Girder 3

Dms	0.9866	.8149	.2331
Dss	.9692	.8018	.2291





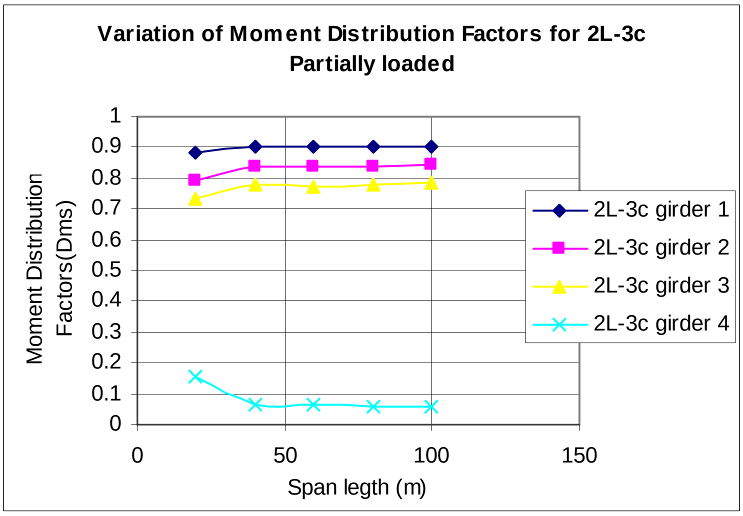
2 L -3c-20	Girder 1	Girder 2	Girder 3	Girder 4
Dms	0.8805	.7917	.7353	0.1583
Dss	.8717	.7733	.7182	0.1483

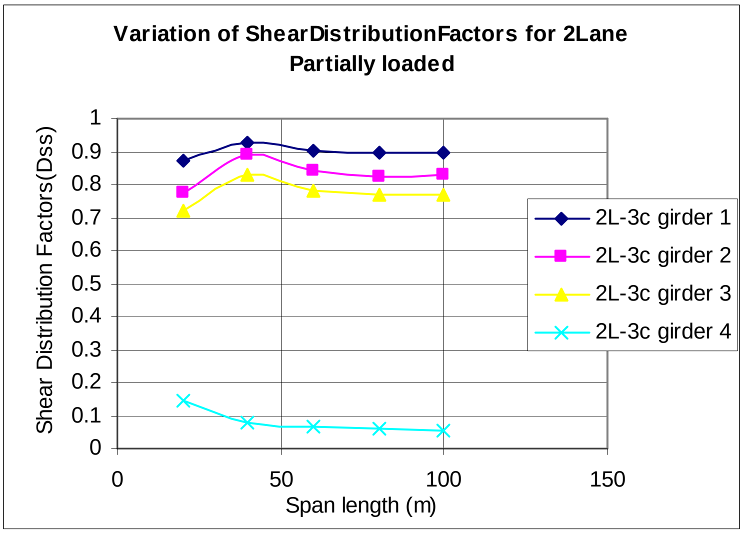
2 L -3c-40	Girder 1	Girder 2	Girder 3	Girder 4
Dms	0.9008	.8392	.7794	0.063
Dss	.9289	.8926	.829	0.0812

2 L -3c-60	Girder 1	Girder 2	Girder 3	Girder 4
Dms	0.9017	.8355	.7759	0.0652
Dss	.9035	.8396	.7798	0.0684

2 L -3c-80	Girder 1	Girder 2	Girder 3	Girder 4
Dms	0.9035	.8388	.7791	0.06
Dss	.8973	.8268	.7679	0.0603

2 L -3c-100	Girder 1	Girder 2	Girder 3	Girder 4
Dms	0.9057	.8443	.7841	0.0561
Dss	.8989	.8301	.7709	0.0557





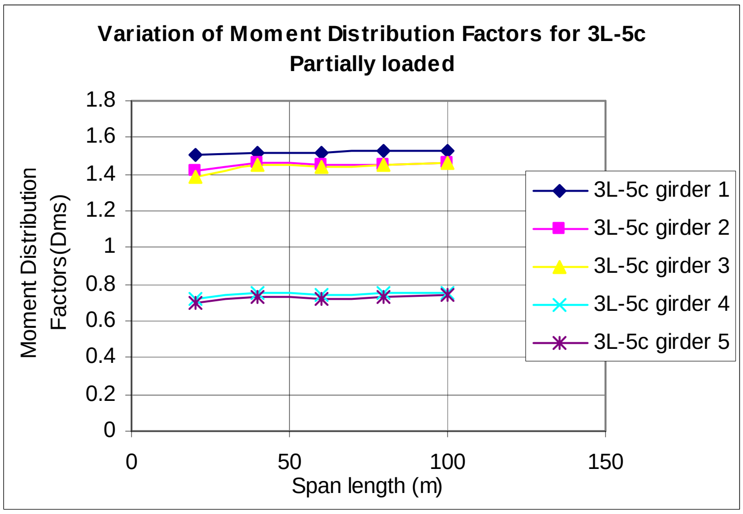
3 L -5c-20	Girder 1	Girder 2	Girder 3	Girder 4	Girder 5	Girder 6
Dms	1.5081	1.4148	1.3813	0.7148	0.6961	0
Dss	1.8722	1.8636	1.6689	.8636	.8411	0

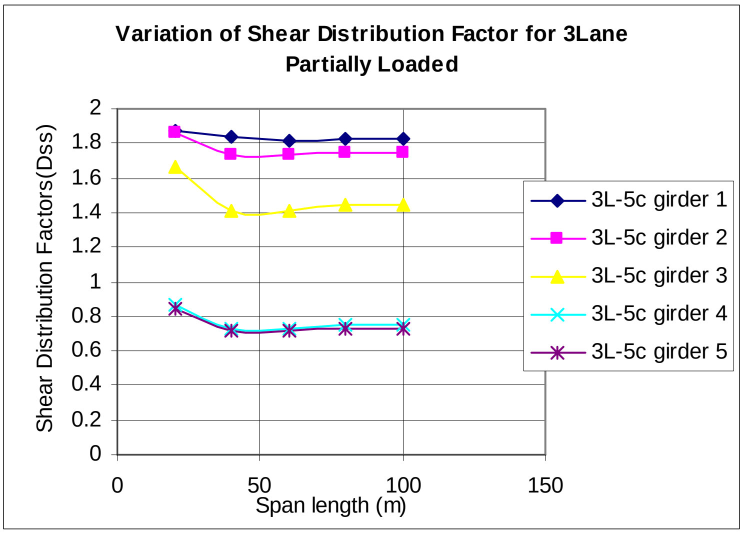
3 L -5c-40	Girder 1	Girder 2	Girder 3	Girder 4	Girder 5	Girder 6
Dms	1.5211	1.4578	1.4485	0.7495	0.7299	0
Dss	1.8343	1.7324	1.4154	.7324	.7133	0

3 L -5c-60	Girder 1	Girder 2	Girder 3	Girder 4	Girder 5	Girder 6
Dms	1.5218	1.4462	1.442	0.7399	0.7206	0
Dss	1.8156	1.7324	1.4154	.7324	.7133	0

3 L -5c-80	Girder 1	Girder 2	Girder 3	Girder 4	Girder 5	Girder 6
Dms	1.5237	1.4504	1.4474	0.7504	0.7294	0
Dss	1.8222	1.7425	1.4446	.7475	.7280	0

3 L -5c-100	Girder 1	Girder 2	Girder 3	Girder 4	Girder 5	Girder 6
Dms	1.5267	1.457	1.463	0.757	0.7373	0
Dss	1.8229	1.7494	1.4483	.7494	.7299	0





4 L -7c-20	Girder 1	Girder 2	Girder 3	Girder 4	Girder 5	Girder 6	Girder 7	Girder 8
Dms	1.7679	1.6858	1.3443	.6858	.5311	0	0	0
Dss	1.8194	1.8175	1.6024	.8175	.5907	0	0	0

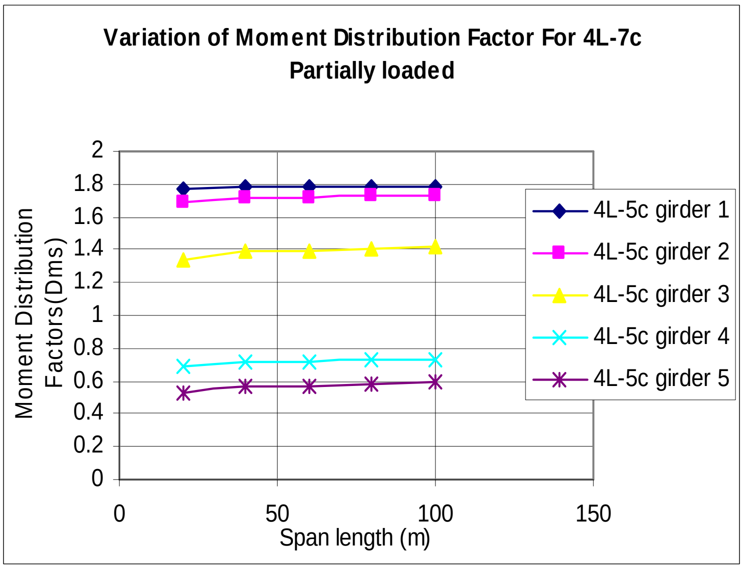
4 L -7c-40	Girder 1	Girder 2	Girder 3	Girder 4	Girder 5	Girder 6	Girder 7	Girder 8
Dms	1.7771	1.7136	1.3894	.7136	.5742	0	0	0

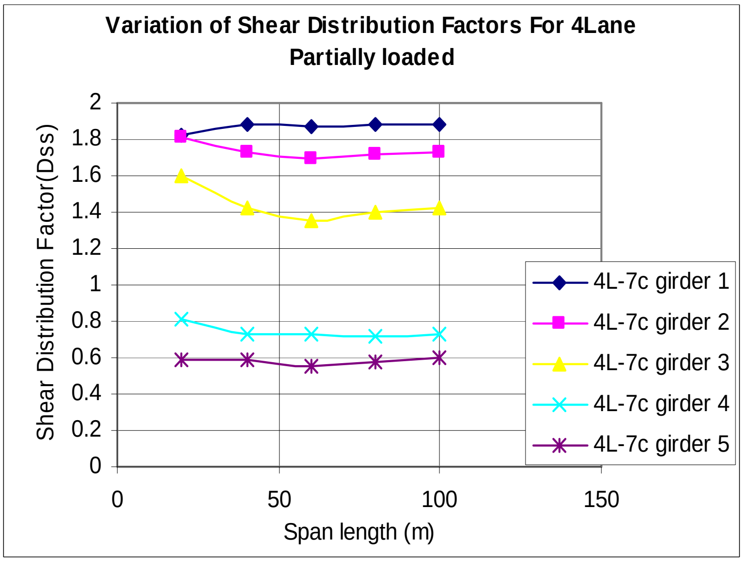
Dss	1.8847	1.7286	1.4186	.7286	.5927	0	0	0

4 L -7c-60	Girder 1	Girder 2	Girder 3	Girder 4	Girder 5	Girder 6	Girder 7	Girder 8
Dms	1.7788	1.7131	1.3886	.7132	.5737	0	0	0
Dss	1.8721	1.6963	1.3558	.7275	.5531	0	0	0

4 L -7c-80	Girder 1	Girder 2	Girder 3	Girder 4	Girder 5	Girder 6	Girder 7	Girder 8
Dms	1.7819	1.7235	1.4088	.7235	.5865	0	0	0
Dss	1.8798	1.7162	1.3944	.7162	.5774	0	0	0

4 L -7c-100	Girder 1	Girder 2	Girder 3	Girder 4	Girder 5	Girder 6	Girder 7	Girder 8
Dms	1.7875	1.7305	1.4226	.7305	.5951	0	0	0
Dss	1.8856	1.7309	1.4232	.7309	.5955	0	0	0





6.8 Drawing Outputs from STAAD.Pro

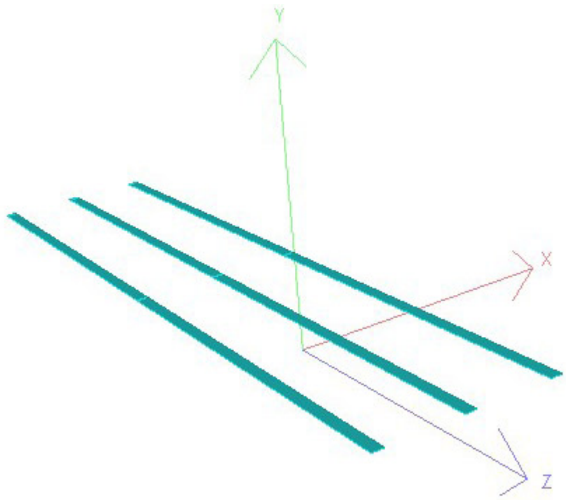


Fig. A. Steel Top Flange (Three Dimensional Two Node Beam Element)

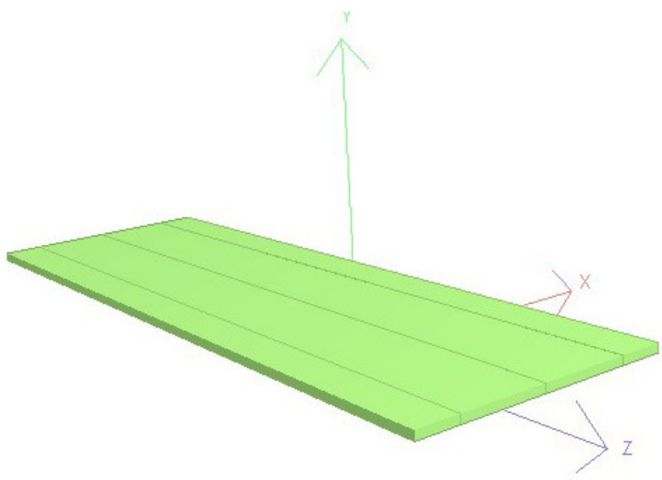


Fig. B. Concrete Deck Slab (Four Node Shell Element with Six Degree of Freedom)

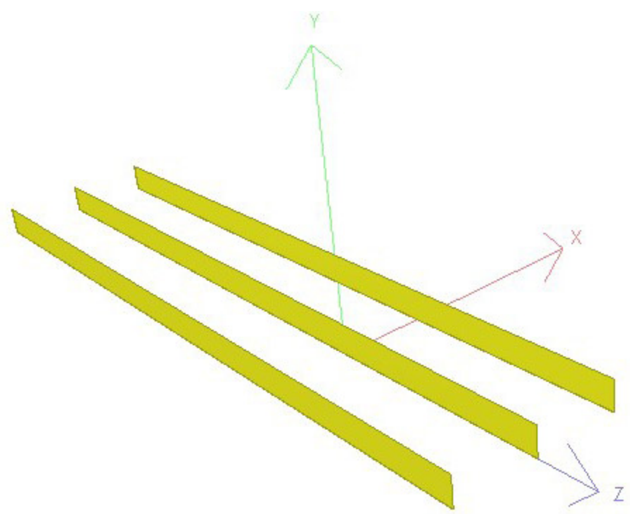


Fig. C. Steel Webs (Four Node Shell Element with Six Degree of Freedom)

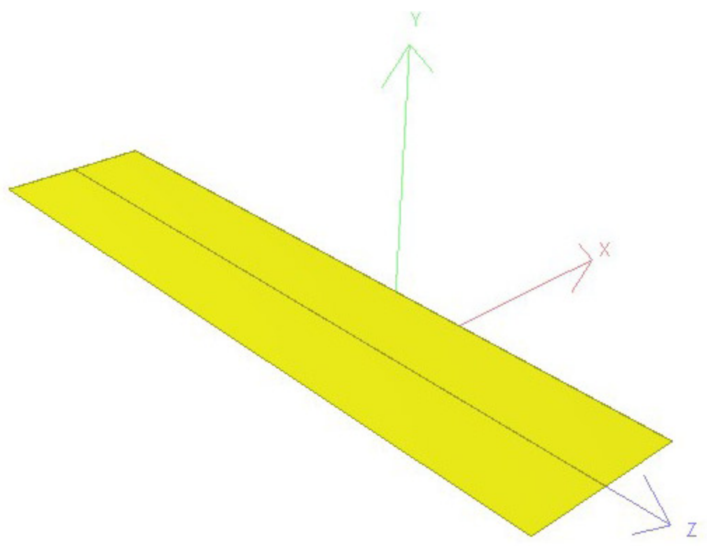


Fig. D. Steel Bottom Flanges (Four Node Shell Element with Six Degree of Freedom)

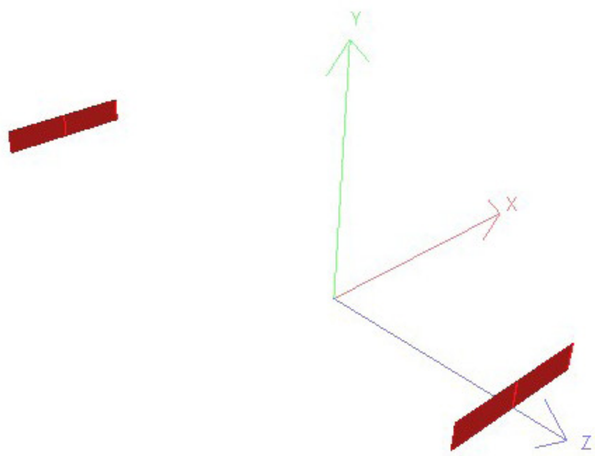


Fig. E. Steel Diaphragms (Four Node Shell Element with Six Degree of Freedom)

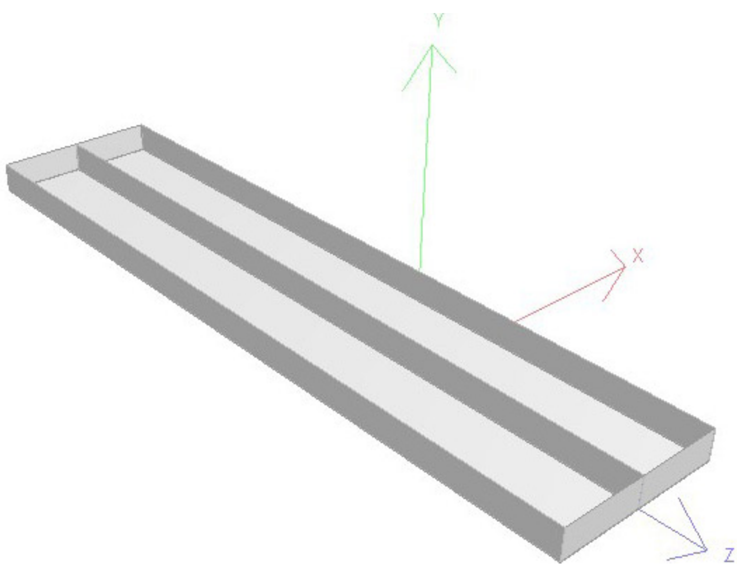


Fig. F. Assembled Bridge without Concrete Deck Slab

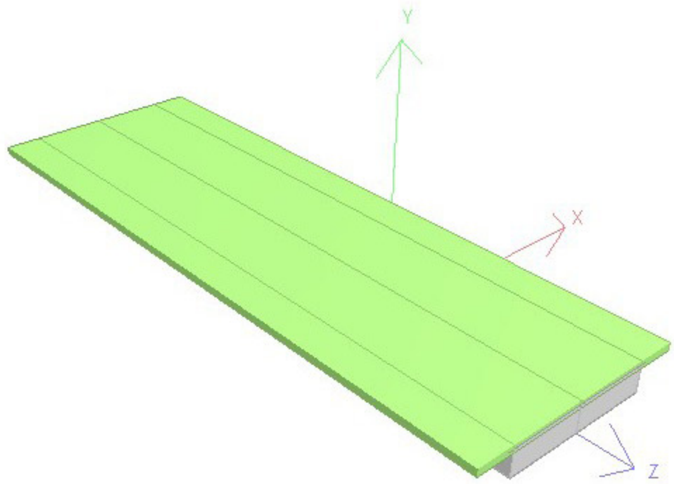


Fig. G Composite Multi-Cell Box-Girder Bridge [2c-20] (Top View)

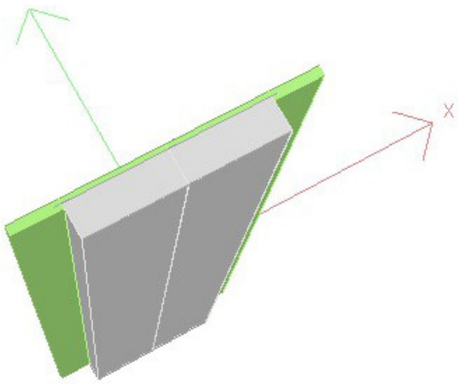


Fig. H Composite Multi-Cell Box-Girder Bridge [2c-20] (Bottom View)

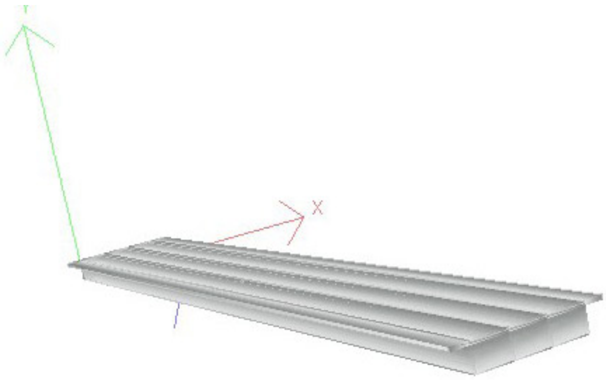


Fig. I Complete Composite Multi-Cell Box-Girder Bridge [3c-60] (Top View)

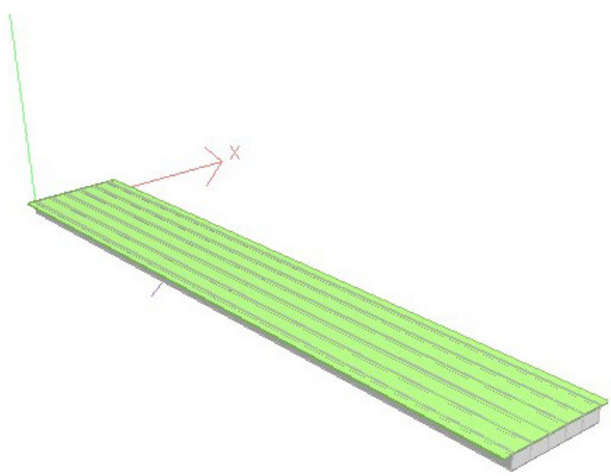


Fig. J Composite Multi-Cell Box-Girder Bridge [5c-80] (Top View)

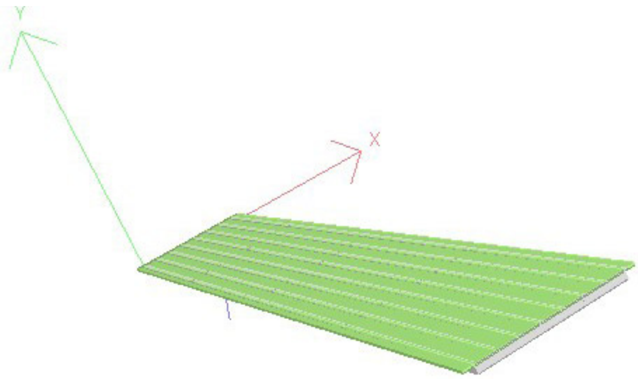


Fig. K Composite Multi-Cell Box-Girder Bridge [7c-100] (Top View)

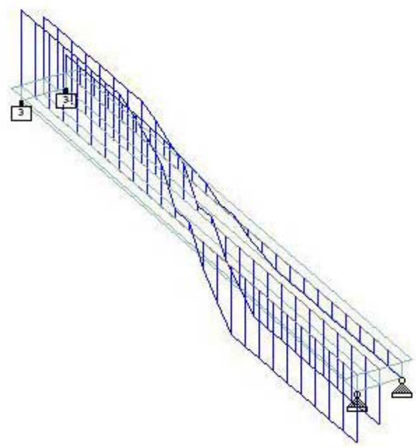


Fig. L. Distribution of Shear Force along Each Girder of Composite Multi-Cell Bridge

7.0 RESULTS

7.1 Expressions for Moment and Shear Distribution Factors

From the results of the parametric study, it became evident that both the moment and shear distribution factors for composite multicell box girder bridges are governed by the following parameters:

(1) Number of cells, N_C

(2) Number of lanes, N_L

(3) Bridge span length, L.

7.1.1 SIMPLIFIED FORMULAE

In order to develop the formulae in a systematic manner, certain assumptions must be made.

1) It is assumed that the effect of the form ax^b, where x is the value of the given parameter and constant a and b is to be determined based on the variation of the distribution factor with x.

2) It is assumed that the effects of different parameters are independent of each other. This assumption allows each parameter to be considered separately.

The final distribution factor will be modeled by an exponential formula of the form " $g = (a)(N_C{}^{b1})(N_L{}^{b2})(L^{b3})(\ldots..)$ ", where g = wheel load distribution factors, N_C, N_L , and L = parameters included in the formula; a = scale factor; and b1, b2, and b3 are determined from the variation of g with N_C, N_L , and L, respectively. Assuming that for two cases of bridge parameters are the same except for S, then

$$g1= (a)(N_{C1}{}^{b1})(N_L{}^{b2})(L^{b3})(\ldots..)$$

$$g2= (a)(N_{C2}{}^{b1})(N_L{}^{b2})(L^{b3})(\ldots..)$$

and therefore

$$(g1/g2) = (S1/S2)^{b1}$$

or

$$b1 = \ln(g1/g2)/\ln(S1/S2)$$

if n different values of N_C are examined and successive pairs are used to determine the value of b1, then (n-1_ different values of b1 can be obtained. If these b1 values are close to each other, an exponential curve was used to accurately model the variation of the distribution factor with N_C. In that case, the average of (n-1) values of b1 is used to achieve the best match. Once all exponents (i.e., b1,b2, etc.)are determined, the value of a can be obtained from average bridge, i.e.

$$a = g0/[(N_{C0})^{b1}(N_{L0}{}^{b2}) (L0^{b3}) (\ldots..)]$$

This procedure was followed during the entire course of the study to develop new formulas as needed. In certain cases where exponential function was not suitable to model the effect of a parameter, a slight variation from this procedure was used to achieve the required accuracy. However, this procedure worked quite well in most cases, and the developed formulas demonstrate accuracy.

Using above procedure along with statistical package for best fit of MATLAB, the following empirical expressions were generated for moment and shear distribution factors for simply supported straight composite multi cell box girder bridges

7.1.2 VERIFICATION AND EVALUATION

Since certain assumptions were made in the derivation of the formulas and some bridge parameters were ignored altogether, it is important to verify the accuracy of these formulas when applied to real bridges. The distribution factors obtained from accurate method were compared with the results of the formulas. The ratio of the formula results to accurate distribution factors was calculated and examined to assess the accuracy of the formula. Average, standard deviation, and minimum and maximum values of the ratios were obtained for each formula. The formula that has the smallest deviation is considered to be the most accurate. The minimum and maximum values present the extreme predictions that each formula produces based on the database of the actual bridges. Although these values may change if a different database is used, the values allow us to identify the shortcomings of a formula that the formula can be fine-tuned to be more accurate. These short comings are not readily identified by the average and standard deviation values.

7.1.3 FINE-TUNING FORMULA

A scatter gram and a histogram (bar graph) were developed for the visual inspection of the results of a formula. The results obtained from the formula are compared with the results obtained from more accurate analysis, and the values that were inspected were the ratios of the distribution factors obtained from the formula to the one obtained from more accurate analysis. This value was referred to as the g-ratio. This visual inspection allows us to identify the revisions and fine-tuning that can improve the results.

Visual inspection and judgment were the key to this fine-tuning process. The trends were examined, and the formulas were fine-tuned by trial and error. When overall accuracy was acceptable but the mean value needed to be adjusted, a constant was added to the formulae.

7.2 Formulae for AASHTO Loading:

These moment and shear distribution formulae for composite multi-cell box-girder bridges were developed by Khaled M.Sennah and John B. Kennedy for AASHTO Loading. These formulae were used in my study as base formulae so as to develop similar kind of formulae for IRC Loading.

Moment Distribution Factors

For the outer girder of one-lane bridges:

$$D_{MS} = (N_C)^{(-0.7)} \tag{7.1}$$

For the outer girder of two, three and four lane bridges:

$$D_{MS} = 1.15(N_L)^{(0.5)}(N_C)^{(-0.7)}(L)^{(0.05)} \tag{7.2}$$

For the intermediate girder of one lane bridges:

$$D_{MS} = 1.6(N_C)^{(-0.8)}(L)^{(-0.04)} \tag{7.3}$$

For the intermediate girder of two, three and four lane bridges:

$$D_{MS} = 2.3(N_L)^{(0.65)}(N_C)^{(-0.9)}(L)^{(-0.04)} \tag{7.4}$$

Shear Distribution Factors

The expressions listed next for the shear distribution factors are applicable only to one and two-lane composite cellular bridges. However, the designer can design three and four lane bridges using the cross sections for one and two lane bridges when the number of cells, $N_{C,}$ are less than or equal to four.

For the outer web due to partial truck loading:

$$D_{SS} = \frac{N_C^3}{(0.7N_L - 20.2)} + \frac{N_C^2}{(-0.06N_L - 1.96)} + \frac{N_C}{(0.02N_L - 0.54)} + 0.5N_L + 2.8 + \frac{L}{(1616N_L - 2370)} \tag{7.5}$$

For the outer web away from load due to partial truck loading:

$$D_{SS} = (0.89N_L - 2.06)(N_C)^{(-0.28N_L+0.36)}(3.4N_L - 2.15)(L)^{(-1.25N_L+1.24)}$$

(7.6)

For the outer web due to fully loaded lanes:

$$D_{SS} = (0.8N_L - 0.15)(N_C)^{(0.01N_L-0.96)}(L)^{(-0.02N_L+0.13)}$$

(7.8)

For the central web due to fully loaded lanes:

$$D_{SS} = (3N_L - 1.2)(N_C)^{(-0.14N_L-0.58)}(L)^{(-0.07N_L+0.04)}$$

(7.9)

7.3 Formulae for IRC Loading:

7.3.1 AIM

The aim of deducing these formulae is get the general formulae for the design of composite multi-cell box-girder bridges under IRC Loading, as this investigation has not been done before for IRC Loading. These formulae will help in calculating the distributed moment and shear in each girder of composite multi-cell box girder.

7.3.2 PROCEDURE USED

By taking the help of AASHTO Loading formulae of shear and moment distribution factor as a base formula and applying the procedure which was discussed in the starting, the following formulae were deduced in the given form:

$$g = c + a(S^{b1})(L^{b2})(.......)$$

These formulae were deduced from the distribution data's of moment and shear, generated by the help of STAAD.Pro analysis of 20 prototype bridges of varying parameters as mentioned before.

7.3.3 IRC LOADING EXPESSIONS

Moment Distribution Factors

For the outer girder of one-lane bridges:

$$D_{MS} = 0.6552(N_C)^{(0.0171)} \quad (7.10)$$

For the outer girder of two, three and four lane bridges:

$$D_{MS} = 0.8194(N_L)^{(0.5169)}(N_C)^{(-0.6618)}(L)^{(0.0045)} \quad (7.11)$$

For the intermediate girder of one lane bridges:

$$D_{MS} = 2.1(N_C)^{(-0.861)}(L)^{(-0.0392)} \quad (7.12)$$

For the intermediate girder of two, three and four lane bridges:

$$D_{MS} = 2.562(N_L)^{(0.573)}(N_C)^{(-0.953)}(L)^{(0.0114)} \quad (7.13)$$

Shear Distribution Factors

The expressions listed next for the shear distribution factors are applicable only to one and two-lane composite cellular bridges. However, the designer can design three and four lane bridges using the cross sections for one and two lane bridges when the number of cells, $N_{C,}$are less than or equal to four.

For the outer web due to partial truck loading:

$$D_{SS} = \frac{N_C^3}{(0.7N_L - 20.2)} + \frac{N_C^2}{(-0.06N_L - 1.96)} + \frac{N_C}{(0.02N_L - 0.54)} + 0.5N_L + 2.5511 + \frac{L}{(1616N_L - 2370)} \quad (7.14)$$

For the outer web away from load due to partial truck loading:

$$D_{SS} = (1.0804N_L - 2.0912)(N_C)^{(-0.4127N_L+0.3601)}(3.401N_L - 2.151)(L)^{(-1.250N_L+1.24)} \tag{7.15}$$

For the outer web due to fully loaded lanes:

$$D_{SS} = (0.9293\,N_L - 0.234)(N_C)^{(0.158\,N_L - 1.078)}(L)^{(-0.01\,N_L + 0.11)} \tag{7.16}$$

For the central web due to fully loaded lanes:

$$D_{SS} = (2.794\,N_L - 1.374)(N_C)^{(-0.045\,N_L - 0.646)}(L)^{(-0.073\,N_L + 0.0371)} \tag{7.17}$$

It should be mentioned that the above expressions for shear distribution factors do not provide only the maximum shear distribution factors for the outer and central webs but also the shear factors corresponding to each loading case. Thus, the use of these expressions for shear would lead to the design of shear connectors, bridge bearings and supporting frames or abutments. When necessary, the shear forces in the intermediate webs can be found by linear interpolation.

7.4 C++ Program

For the above expressions a simple program is developed in C^{++} which is listed bellow

```
#include <conio.h>
#include <iostream.h>
#include <math.h>
int options_MDF()
{
  int choice;
  cout<<endl<<endl<<endl;
```

```
  cout<<"1: for the outer girder of one lane bridge"<<endl;
  cout<<"2: for the outer girder of two, three and four lane bridges"<<endl;
  cout<<"3: for the intermediate girder of one lane bridge"<<endl;
  cout<<"4: for the intermediate girder two, three and four lane
bridges"<<endl<<endl<<endl;
  cout<<"enter your choice..........";
  cin>>choice;
  return(choice);
}
void MomentDF(float Nc, float Nl, float L)
{
 float Dms;
 int choice=options_MDF();
 switch(choice)
  {
     case 1:
Dms=0.6552*pow(Nc,0.0171);
                                                                  break;
     case 2:
Dms=0.8194*pow(Nl,0.5169)*pow(Nc,-0.6618)*pow(L,.0045);
                                                                  break;
     case 3:

Dms=2.1*pow(Nc,-0.861)*pow(L,-0.0392);
                                                                  break;
     case 4:

Dms=2.562*pow(Nl,0.573)*pow(Nc,-0.953)*pow(L,0.0114);
                                                                  break;
  }
 cout<<endl<<" The Moment Distribution Factor is...........";
 cout<<Dms;
 getch();
}
int options_SDF()
{
  int choice;
  cout<<endl<<endl<<endl;
  cout<<"1: for the outer web due to partial wheel loading"<<endl;
  cout<<"2: for the outer web away from load due to partial wheel loading"<<endl;
  cout<<"3: for the outer web due to fully loaded lanes"<<endl;
  cout<<"4: for the central web due to fully loaded lanes"<<endl<<endl<<endl;
  cout<<"enter your choice..........";
  cin>>choice;
  return(choice);
}
void ShearDF(float Nc, float Nl, float L)
{
 float Dss;
```

```
 int choice=options_SDF();
 switch(choice)
  {
     case 1:
    Dss=pow(Nc,3)/(0.07*Nl-20.2)+pow(Nc,2)/(-0.06*Nl+1.96)+Nc/(0.02*Nl-
0.54)+0.5*Nl+2.5511+L/(-1616*Nl+2370);
                                                                          break;
     case 2:
                                                                          Dss=(1.0804*Nl-
2.0912)*pow(Nc,(-0.4127*Nl+0.3601))+(3.401*Nl-2.151)*pow(L,(-
1.2504*Nl+1.24015));
                                                                          break;
     case 3:
                                                                          Dss=(0.9293*Nl-
0.234)*pow(Nc,0.158*Nl-1.078)*pow(L,-0.01*Nl+0.11);
                                                                          break;
     case 4:
                                                                          Dss=(2.794*Nl-
1.374)*pow(Nc,-0.045*Nl-0.646)*pow(L,-0.073*Nl+0.0371);
                                                                          break;
  }
 cout<<endl<<" The Shear Distribution Factor is...........";
 cout<<Dss;
 getch();
}
void main()
{
 float Nc,Nl,L;
 float Dms;
 int choice;
 clrscr();
 cout<<"1: Moment distribution factor"<<endl;
 cout<<"2: Shear distribution factor"<<endl;
 cout<<"enter your choice.........";
 cin>>choice;
 clrscr();
 switch(choice)
 {
   case 1:
                                                                         cout<<"
Moment distribution factor"<<endl<<endl<<endl;
                                                                         cout<<"enter  value
of number of cells";
                                                                         cin>>Nc;
                                                                         cout<<"enter
number of lanes";
                                                                         cin>>Nl;
                                                                         cout<<"enter  value
of span length in meters";
```

```
cin>>L;
MomentDF(Nc,Nl,L);
break;
case 2:
cout<<"
Shear distribution factor"<<endl<<endl<<endl;
cout<<"
only for one and two lane and for Nc<=4"<<endl<<endl<<endl;
cout<<"enter value
of number of cells";
cin>>Nc;
cout<<"enter
number of lanes";
cin>>Nl;
cout<<"enter value
of span length in meters";
cin>>L;
ShearDF(Nc,Nl,L);
break;

}
}
```

7.5 Distribution Factors from Formulae

The moment and shear distribution factor which were deduced from the IRC formulae are given below in the tabular form. Here the result of 2 lanes case is only presented.

2 L -2c-20	Girder 1	Girder 2	Girder 3
Dms	0.7512	2.037	0.7512
Dss	1.2544	1.825	1.2544

2 L -2c-40	Girder 1	Girder 2	Girder 3
Dms	.7535	2.0533	.7535
Dss	1.335	1.693	1.335

2 L -2c-60	Girder 1	Girder 2	Girder 3
Dms	.7549	2.063	.7549
Dss	1.3848	1.6199	1.3848

2 L -2c-80	Girder 1	Girder 2	Girder 3
Dms	.7559	2.069	.7559
Dss	1.4211	1.5699	1.4211

2 L -2c-100	Girder 1	Girder 2	Girder 3
Dms	.7566	2.075	.7566

Dss	1.4499	1.5323	1.4499

2 L -3c-20	Girder 1	Girder 2	Girder 3	Girder 4
Dms	.5744	1.3842	1.3842	.5744
Dss	.9210	1.3547	1.3547	.9210

2 L -3c-40	Girder 1	Girder 2	Girder 3	Girder 4
Dms	.5762	1.3952	1.3952	.5762
Dss	.9803	1.2562	1.2562	.9803

2 L -3c-60	Girder 1	Girder 2	Girder 3	Girder 4
Dms	.5772	1.402	1.402	.5772
Dss	1.016	1.2019	1.2019	1.016

2 L -3c-80	Girder 1	Girder 2	Girder 3	Girder 4
Dms	.578	1.4063	1.4063	.578
Dss	1.0434	1.1649	1.1649	1.0434

2 L -3c-100	Girder 1	Girder 2	Girder 3	Girder 4
Dms	.5785	1.4098	1.4098	.5785
Dss	1.0646	1.1369	1.1369	1.0646

2 L -5c-20	Girder 1	Girder 2	Girder 3	Girder 4	Girder 5	Girder 6
Dms	.4096	.8507	.8507	.8507	.8507	.4096

2 L -5c-40	Girder 1	Girder 2	Girder 3	Girder 4	Girder 5	Girder 6
Dms	.4109	.8575	.8575	.8575	.8575	.4109

2 L -5c-60	Girder 1	Girder 2	Girder 3	Girder 4	Girder 5	Girder 6
Dms	.4116	.8614	.8614	.8614	.8614	.4116

2 L -5c-80	Girder 1	Girder 2	Girder 3	Girder 4	Girder 5	Girder 6
Dms	.4122	.8643	.8643	.8643	.8643	.4122

2 L -5c-100	Girder 1	Girder 2	Girder 3	Girder 4	Girder 5	Girder 6
Dms	.4126	.8665	.8665	.8665	.8665	.4126

2 L -7c-20	Girder 1	Girder 2	Girder 3	Girder 4	Girder 5	Girder 6	Girder 7	Girder 8
Dms	.3278	.6173	.6173	.6173	.6173	.6173	.6173	.3278

2 L -7c-40	Girder 1	Girder 2	Girder 3	Girder 4	Girder 5	Girder 6	Girder 7	Girder 8
Dms	.3289	.6222	.6222	.6222	.6222	.6222	.6222	.3289

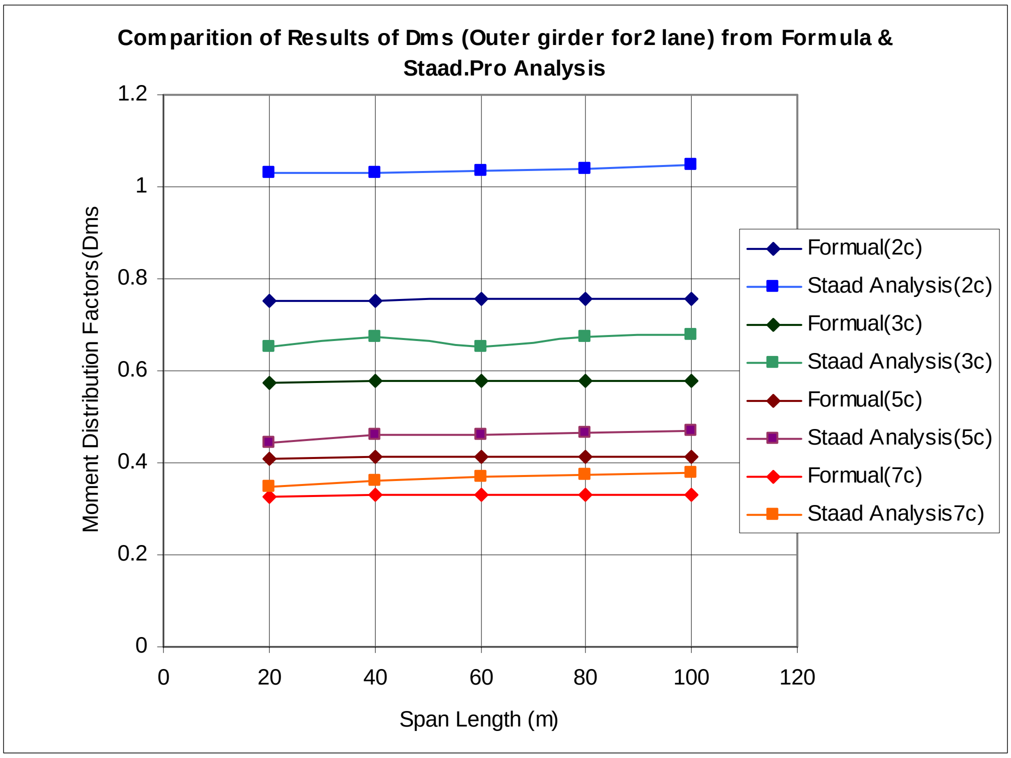
2 L -7c-60	Girder 1	Girder 2	Girder 3	Girder 4	Girder 5	Girder 6	Girder 7	Girder 8
Dms	.3295	.6251	.6251	.6251	.6251	.6251	.6251	.3295

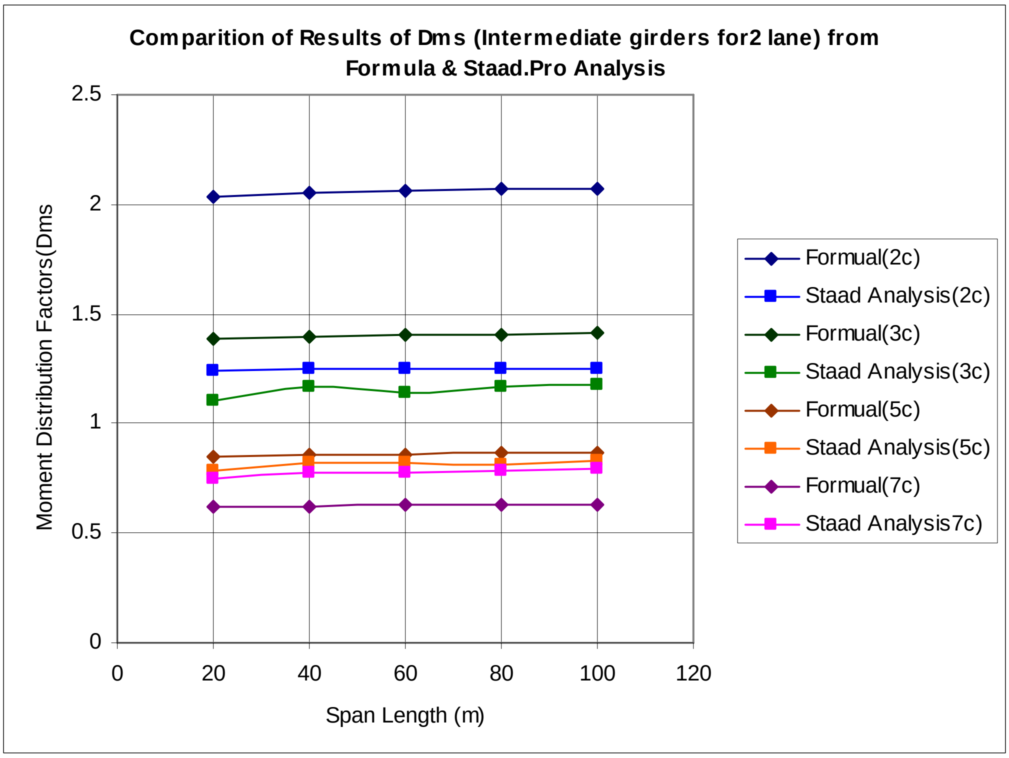
2 L -7c-80	Girder 1	Girder 2	Girder 3	Girder 4	Girder 5	Girder 6	Girder 7	Girder 8
Dms	.3299	.6272	.6272	.6272	.6272	.6272	.6272	.3299

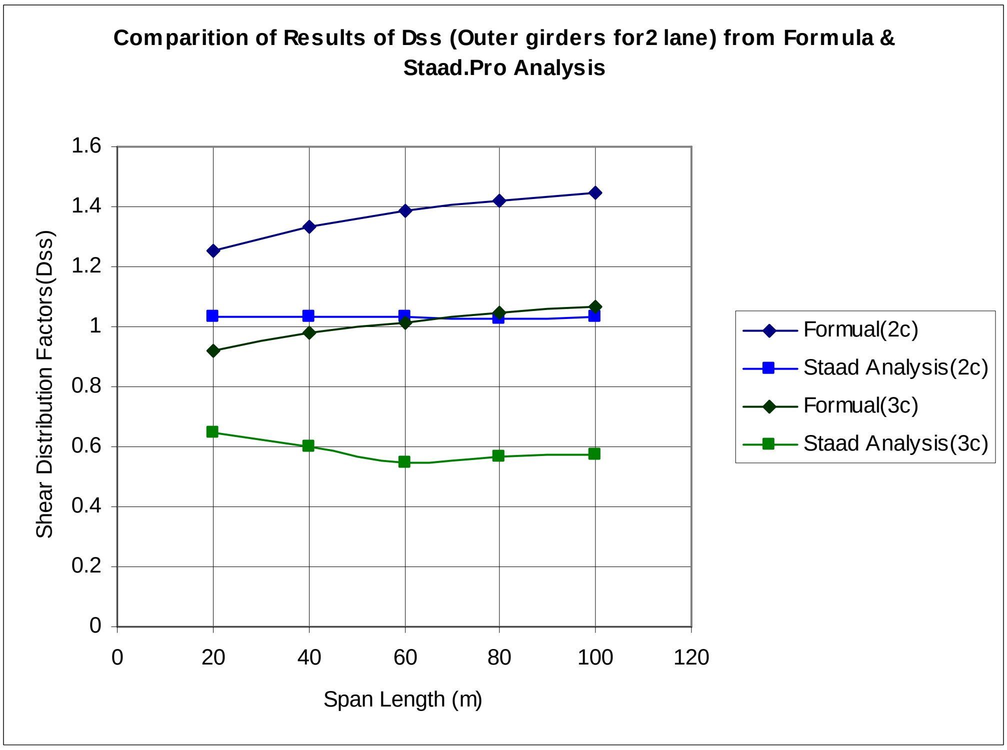
2 L -7c-100	Girder 1	Girder 2	Girder 3	Girder 4	Girder 5	Girder 6	Girder 7	Girder 8
Dms	.3302	.6288	.6288	.6288	.6288	.6288	.6288	.3302

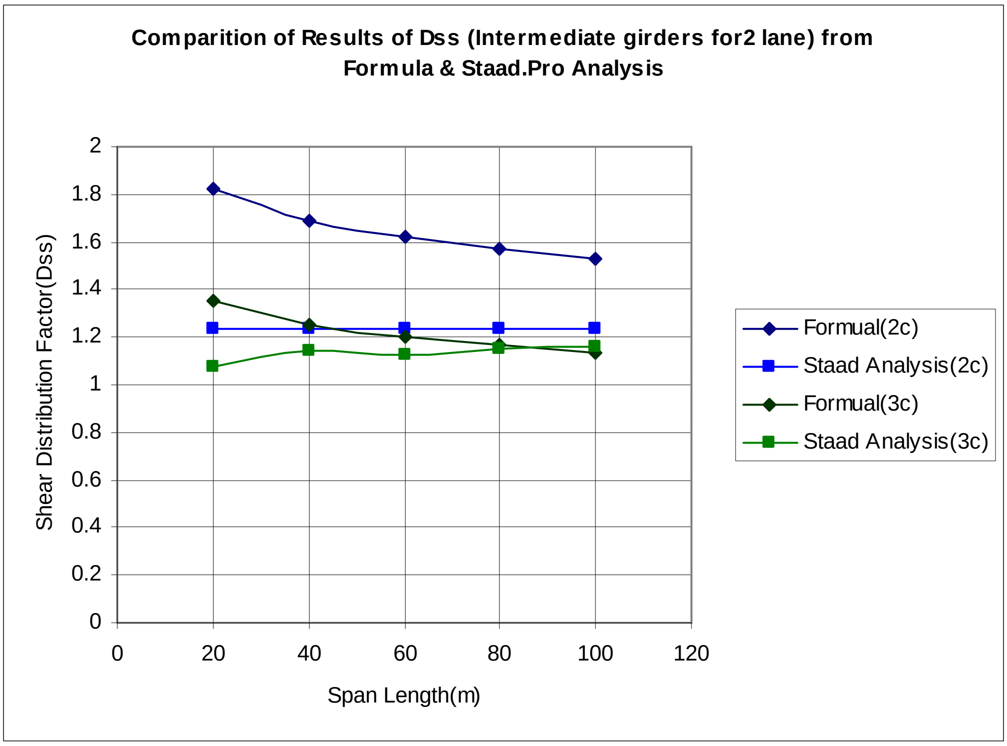
7.6 Comparison of Results

The moment and shear distribution factors which were deduced by the help of STAAD.Pro analysis and IRC formulae were compared to each other by the help of graphs as shown below. The graphs shows the comparison of distribution factors for the case of 2 Lane.




Comparition of Results of Dms (Intermediate girders for2 lane) from Formula & Staad.Pro Analysis
Moment Distribution Factors(Dms
2.5
2
1.5
1
0.5
0
0
20
40
60
80
100
120
Span Length (m)
Formual(2c)
Staad Analysis(2c)
Formual(3c)
Staad Analysis(3c)
Formual(5c)
Staad Analysis(5c)
Formual(7c)
Staad Analysis7c)


Comparition of Results of Dss (Outer girders for2 lane) from Formula & Staad.Pro Analysis
Shear Distribution Factors(Dss)
1.6
1.4
1.2
1
0.8
0.6
0.4
0.2
0
0
20
40
60
80
100
120
Span Length (m)
Formual(2c)
Staad Analysis(2c)
Formual(3c)
Staad Analysis(3c)



The results obtained from STAAD.Pro analysis and from the developed equations for the determination of the Load Distribution Coefficients for moment on an average varies by about 4 to14 percent. Whereas in the case of Distribution Coefficients for shear found to differ by an average of about 6 to 20 percent in the case of 2 lane compared above in this parametric study. Therefore, as per the guide lines the developed equations for the Load Distribution Coefficients for moment and shear can be adopted in the design of composite multi cell box girder bridges.

8.0 CONCLUSIONS AND SUGGESTIONS

8.1 Conclusion

An extensive parametrical investigation was conducted to determine the effect of several variables such as span length, number of lanes, and number of cells on moment and shear distributions in simply supported straight composite concrete deck-steel multicell box girder bridges. Expressions were deduced for computing the moment distribution factors for each girder and the shear distribution factor at each web. Thou, experimental investigation were not performed due to lack of time but the results from this investigation provide valuable design information currently unavailable in Indian bridge codes for the design of composite cellular bridges. So, based on the study the following conclusion can be drawn the use of the proposed expressions for shear distribution factor needs more fine tuning to give more accurate results, so that it would lead to a more economical and reliable design of shear connectors, bridge bearings and supporting frames or abutments. Where as the expression proposed for moment distribution factors gives results with in the required accuracy.

8.2 Suggested Direction for Further Research

Since due to lack of time experimental work was not conducted, a research in this area can be done by making a composite concrete deck-steel cell bridge model and testing it under various static loading conditions.

The distribution factors for deflection, torsional moment and study of the variation of cross bracings on distribution factors are the areas which have not been touched in this work can be used for further research in this field.

Same kind of study can be done for the cases of continuous and curved composite concrete deck-steel cell bridges according to IRC loading.

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Definition of Moving Load System

This set of commands may be used to define the moving load system.

```
General format:

DEFINE  MOVING  LOAD    (FILE file-name)

        |LOAD  f1,f2,..fn ( DISTANCE  d1,d2,..dn-1  (WIDTH w) )  |
TYPE j  {                                                         }
        |load-name   (f)                                          |

( DISTANCE   d1,d2,..dn-1   (WIDTH w) )   < optionally as 2nd set >
```

Note that the MOVING LOAD system may be defined in two possible ways - directly within the input file or using an external file.

<u>Define Moving Load within input file</u>

Use the first TYPE specification. Input Data must be all in one line (as shown above) or two sets of lines. If two sets, then the second set must begin with DIS as shown above. If two sets, then Load and Dist lines may end each line but last of each set with a hyphen

```
TYPE  j  LOAD   f1,f2,...fn                         (set 1)
DISTANCE   d1,d2,...d(n-1)   (WIDTH  w)             (set 2)
```

Where,

j = moving load system type number (integer limit of 100 types)

n = number of loads (e.g. axles), 2 to 50. f_i = value of conc. i^{th} load

d_i = distance between the $(i+1)^{th}$ load and the i^{th} load in the direction of movement

w = spacing between loads perpendicular to the direction of movement. If left out, one dimensional loading is assumed. (e.g. the width of vehicle). Note that this parameter will double the total load since the f_i is applied to each wheel.

Define Moving Load using an external file

Use the second TYPE specification.

TYPE j load-name (f)

Where,load-name Is the name of the moving load system (maximum of 12 characters).and

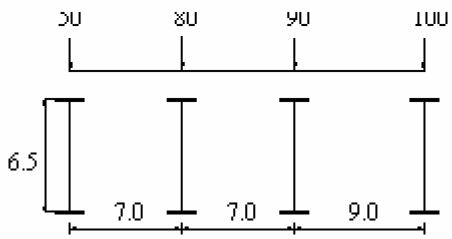
f = Optional multiplying factor to scale up or down the value of the loads. (default = 1.0) Following is a typical file containing the data.

CS200 ----- name of load system (load-name, must start in column1)

50. 80. 90. 100. ----- loads (all on one 79 char input line)

7. 7. 9. ----- distance between loads (one line)

6.5 ----- width



Note that several load systems may be repeated within the same file.

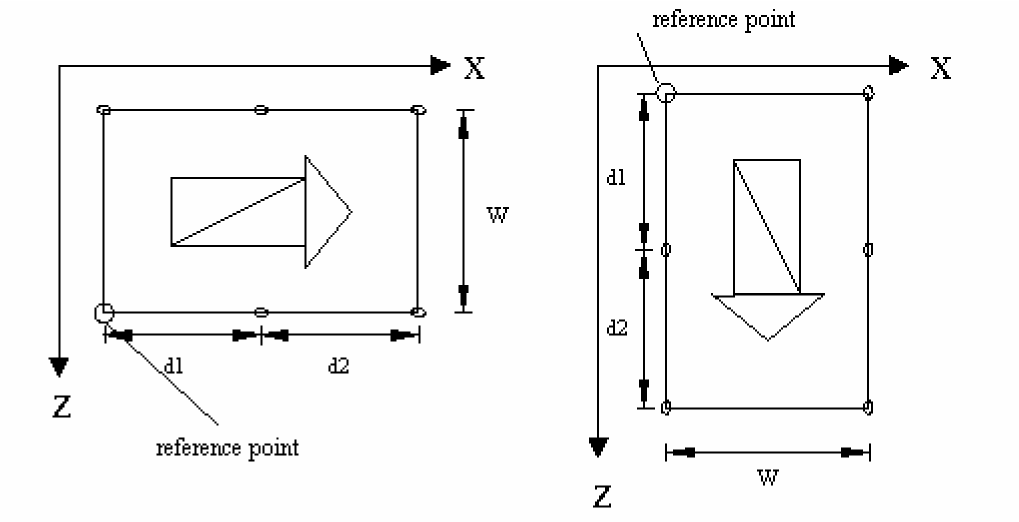
The STAAD moving load generator assumes:

1) All loads are acting in the negative global vertical (Y or Z) direction. The user is advised to set up the structure model accordingly.

2) Resultant direction of movement is determined from the X, Y and Z increments of movements as provided by the user.

Reference Load

The first specified concentrated load in the moving load system is designated as the reference load. While generating subsequent primary load cases, the initial position of the load system and the direction of movement are defined with respect to the reference load location. Also note that, when selecting the reference load location with a positive value of Width specified, then the following two views define the reference load location.



Movement parallel to global X axis *Movement parallel to global Z axis*

Notice that in the left view, the reference point is on the positive Z wheel track side; whereas in the right view, the reference point is on the least positive X wheel track side.

STAAD.PRO EDITOR FILES

1l-2c-20

```
STAAD SPACE
START JOB INFORMATION
ENGINEER DATE 28-Mar-05
END JOB INFORMATION
INPUT WIDTH 79
UNIT METER KN
JOINT COORDINATES
57 0 0.8 -14; 58 0 0 -14; 59 2.27 0 -14; 60 2.27 0.8 -14; 89 -2.27 0.8 -14;
104 -2.27 0 -14; 119 -3.4 0.8 -14; 134 3.4 0.8 -14; 135 0 0.8 6; 136 0 0 6;
137 2.27 0 6; 138 2.27 0.8 6; 139 -2.27 0.8 6; 140 -2.27 0 6; 141 -3.4 0.8 6;
142 3.4 0.8 6; 143 -2.27 0.8 -4; 144 0 0.8 -4; 145 2.27 0.8 -4;
MEMBER INCIDENCES
1 89 143; 2 57 144; 3 60 145; 17 143 139; 18 144 135; 19 145 138;
ELEMENT INCIDENCES SHELL
4 119 141 139 89; 5 89 139 135 57; 6 57 135 138 60; 7 60 138 142 134;
8 89 104 140 139; 9 104 140 136 58; 10 57 58 136 135; 11 58 136 137 59;
12 60 59 137 138; 13 89 104 58 57; 14 57 58 59 60; 15 139 140 136 135;
16 135 136 137 138;
DEFINE MATERIAL START
```

```
ISOTROPIC MATERIAL1
E 2.05e+008
POISSON 0.25
DENSITY 77
ALPHA 1.2e-011
DAMP 2.8026e-044
ISOTROPIC MATERIAL2
E 2.5e+007
POISSON 0.25
DENSITY 24
ALPHA 1.2e-011
DAMP 2.8026e-044
ISOTROPIC MATERIAL3
E 2.05e+008
POISSON 0.25
DENSITY 24
ALPHA 1.2e-011
DAMP 2.8026e-044
END DEFINE MATERIAL
MEMBER PROPERTY
1 TO 3 17 TO 19 PRIS YD 0.016 ZD 0.3
ELEMENT PROPERTY
8 TO 16 THICKNESS 0.01
4 TO 7 THICKNESS 0.225
CONSTANTS
MATERIAL MATERIAL1 MEMB 1 TO 3 17 TO 19
MATERIAL MATERIAL2 MEMB 4 TO 7
MATERIAL MATERIAL3 MEMB 8 TO 16
SUPPORTS
59 104 FIXED BUT FZ MX MY MZ
137 140 PINNED
DEFINE MOVING LOAD
TYPE 1 LOAD 93.2 139.8 139.8 198.1 198.1 198.1 198.1
DIST 3.96 1.52 2.13 1.37 3.05 1.37 WID 2.72
LOAD GENERATION 10
TYPE 1 -2.2 0.8 -14 ZINC 4.54
PERFORM ANALYSIS
PRINT MEMBER FORCES LIST 1 TO 3 17 TO 19
FINISH
```

1l-2c-20c

```
STAAD SPACE
START JOB INFORMATION
ENGINEER DATE 28-Mar-05
END JOB INFORMATION
INPUT WIDTH 79
UNIT METER KN
JOINT COORDINATES
57 0 0.8 -14; 58 0 0 -14; 59 2.27 0 -14; 60 2.27 0.8 -14; 89 -2.27 0.8 -14;
104 -2.27 0 -14; 119 -3.4 0.8 -14; 134 3.4 0.8 -14; 135 0 0.8 6; 136 0 0 6;
137 2.27 0 6; 138 2.27 0.8 6; 139 -2.27 0.8 6; 140 -2.27 0 6; 141 -3.4 0.8 6;
142 3.4 0.8 6; 143 -2.27 0.8 -4; 144 0 0.8 -4; 145 2.27 0.8 -4;
MEMBER INCIDENCES
1 89 143; 2 57 144; 3 60 145; 17 143 139; 18 144 135; 19 145 138;
ELEMENT INCIDENCES SHELL
4 119 141 139 89; 5 89 139 135 57; 6 57 135 138 60; 7 60 138 142 134;
8 89 104 140 139; 9 104 140 136 58; 10 57 58 136 135; 11 58 136 137 59;
12 60 59 137 138; 13 89 104 58 57; 14 57 58 59 60; 15 139 140 136 135;
16 135 136 137 138;
DEFINE MATERIAL START
ISOTROPIC MATERIAL1
E 2.05e+008
POISSON 0.25
DENSITY 77
ALPHA 1.2e-011
DAMP 2.8026e-044
ISOTROPIC MATERIAL2
E 2.5e+007
POISSON 0.25
DENSITY 24
ALPHA 1.2e-011
DAMP 2.8026e-044
ISOTROPIC MATERIAL3
E 2.05e+008
POISSON 0.25
```

```
DENSITY 24
ALPHA 1.2e-011
DAMP 2.8026e-044
END DEFINE MATERIAL
MEMBER PROPERTY
1 TO 3 17 TO 19 PRIS YD 0.016 ZD 0.3
ELEMENT PROPERTY
8 TO 16 THICKNESS 0.01
4 TO 7 THICKNESS 0.225
CONSTANTS
MATERIAL MATERIAL1 MEMB 1 TO 3 17 TO 19
MATERIAL MATERIAL2 MEMB 4 TO 7
MATERIAL MATERIAL3 MEMB 8 TO 16
SUPPORTS
59 104 FIXED BUT FZ MX MY MZ
137 140 PINNED
DEFINE MOVING LOAD
TYPE 1 LOAD 93.2 139.8 139.8 198.1 198.1 198.1 198.1
DIST 3.96 1.52 2.13 1.37 3.05 1.37 WID 2.72
LOAD GENERATION 10
TYPE 1 0 0.8 -14 ZINC 4.54
PERFORM ANALYSIS
PRINT MEMBER FORCES LIST 1 TO 3
FINISH
```

1l-2c-40

```
STAAD SPACE 1L-2C-40
START JOB INFORMATION
ENGINEER DATE 15-Apr-05
END JOB INFORMATION
INPUT WIDTH 79
UNIT MMS KN
JOINT COORDINATES
1 0 1600 0; 2 0 0 0; 3 2270 0 0; 4 2270 1600 0; 45 4540 0 0; 46 4540 1600 0;
47 0 1600 20000; 50 2270 1600 20000; 52 4540 1600 20000; 53 0 1600 40000;
54 0 0 40000; 55 2270 0 40000; 56 2270 1600 40000; 57 4540 0 40000;
58 4540 1600 40000; 59 -1130 1600 0; 61 -1130 1600 40000; 62 5670 1600 0;
64 5670 1600 40000;
MEMBER INCIDENCES
1 1 47; 2 47 53; 3 4 50; 4 50 56; 5 46 52; 6 52 58;
ELEMENT INCIDENCES SHELL
7 59 61 53 1; 8 1 2 54 53; 9 2 54 55 3; 10 3 55 56 4; 11 3 55 57 45;
12 45 57 58 46; 13 46 58 64 62; 14 46 4 56 58; 15 4 1 53 56; 16 53 54 55 56;
17 56 55 57 58; 18 1 2 3 4; 19 4 3 45 46;
UNIT METER KN
DEFINE MATERIAL START
ISOTROPIC MATERIAL1
E 2.05e+008
POISSON 0.25
DENSITY 77
ALPHA 1.2e-011
DAMP 2.8026e-044
ISOTROPIC MATERIAL2
E 2.5e+007
POISSON 0.25
DENSITY 24
ALPHA 1.2e-011
DAMP 2.8026e-044
END DEFINE MATERIAL
UNIT MMS KN
MEMBER PROPERTY
1 TO 6 PRIS YD 28 ZD 375
ELEMENT PROPERTY
7 13 TO 15 THICKNESS 225
8 10 12 16 TO 19 THICKNESS 14
9 11 THICKNESS 12
SUPPORTS
54 57 PINNED
2 45 FIXED BUT FZ MX MY MZ
UNIT METER KN
CONSTANTS
MATERIAL MATERIAL1 MEMB 1 TO 6 8 TO 12 16 TO 19
MATERIAL MATERIAL2 MEMB 7 13 TO 15
DEFINE MOVING LOAD
```

```
TYPE 1 LOAD 87.2 130.8 130.8 185.3 185.3 185.3 185.3 218 218
DIST 3.96 1.52 2.13 1.37 3.05 1.37 2.74 1.22 WID 2.72
LOAD GENERATION 10
TYPE 1 0.07 1.6 0 ZINC 4.54
PERFORM ANALYSIS
PERFORM ANALYSIS PRINT LOAD DATA
FINISH
```

1l-2c-60

```
STAAD SPACE 1L-2C-40
START JOB INFORMATION
ENGINEER DATE 15-Apr-05
END JOB INFORMATION
INPUT WIDTH 79
UNIT MMS KN
JOINT COORDINATES
1 0 1600 0; 2 0 0 0; 3 2270 0 0; 4 2270 1600 0; 45 4540 0 0; 46 4540 1600 0;
47 0 1600 30000; 50 2270 1600 30000; 52 4540 1600 30000; 53 0 1600 60000;
54 0 0 60000; 55 2270 0 60000; 56 2270 1600 60000; 57 4540 0 60000;
58 4540 1600 60000; 59 -1130 1600 0; 61 -1130 1600 60000; 62 5670 1600 0;
64 5670 1600 60000;
MEMBER INCIDENCES
1 1 47; 2 47 53; 3 4 50; 4 50 56; 5 46 52; 6 52 58;
ELEMENT INCIDENCES SHELL
7 59 61 53 1; 8 1 2 54 53; 9 2 54 55 3; 10 3 55 56 4; 11 3 55 57 45;
12 45 57 58 46; 13 46 58 64 62; 14 46 4 56 58; 15 4 1 53 56; 16 53 54 55 56;
17 56 55 57 58; 18 1 2 3 4; 19 4 3 45 46;
UNIT METER KN
DEFINE MATERIAL START
ISOTROPIC MATERIAL1
E 2.05e+008
POISSON 0.25
DENSITY 77
ALPHA 1.2e-011
DAMP 2.8026e-044
ISOTROPIC MATERIAL2
E 2.5e+007
POISSON 0.25
DENSITY 24
ALPHA 1.2e-011
DAMP 2.8026e-044
END DEFINE MATERIAL
UNIT MMS KN
MEMBER PROPERTY
1 TO 6 PRIS YD 28 ZD 375
ELEMENT PROPERTY
7 13 TO 15 THICKNESS 225
8 10 12 16 TO 19 THICKNESS 14
9 11 THICKNESS 12
SUPPORTS
54 57 PINNED
2 45 FIXED BUT FZ MX MY MZ
UNIT METER KN
CONSTANTS
MATERIAL MATERIAL1 MEMB 1 TO 6 8 TO 12 16 TO 19
MATERIAL MATERIAL2 MEMB 7 13 TO 15
DEFINE MOVING LOAD
TYPE 1 LOAD 87.2 130.8 185.3 185.3 185.3 185.3 218 218
DIST 3.96 1.52 2.13 1.37 3.05 1.37 2.74 1.22 WID 2.72
LOAD GENERATION 10
TYPE 1 0.07 1.6 0 ZINC 4.54
PERFORM ANALYSIS
PERFORM ANALYSIS PRINT LOAD DATA
FINISH
```

1l-2c-80

```
STAAD SPACE 1L-2C-40
START JOB INFORMATION
ENGINEER DATE 15-Apr-05
END JOB INFORMATION
INPUT WIDTH 79
UNIT MMS KN
JOINT COORDINATES
```

```
1 0 1600 0; 2 0 0 0; 3 2270 0 0; 4 2270 1600 0; 45 4540 0 0; 46 4540 1600 0;
47 0 1600 40000; 50 2270 1600 40000; 52 4540 1600 40000; 53 0 1600 80000;
54 0 0 80000; 55 2270 0 80000; 56 2270 1600 80000; 57 4540 0 80000;
58 4540 1600 80000; 59 -1130 1600 0; 61 -1130 1600 80000; 62 5670 1600 0;
64 5670 1600 80000;
MEMBER INCIDENCES
1 1 47; 2 47 53; 3 4 50; 4 50 56; 5 46 52; 6 52 58;
ELEMENT INCIDENCES SHELL
7 59 61 53 1; 8 1 2 54 53; 9 2 54 55 3; 10 3 55 56 4; 11 3 55 57 45;
12 45 57 58 46; 13 46 58 64 62; 14 46 4 56 58; 15 4 1 53 56; 16 53 54 55 56;
17 56 55 57 58; 18 1 2 3 4; 19 4 3 45 46;
UNIT METER KN
DEFINE MATERIAL START
ISOTROPIC MATERIAL1
E 2.05e+008
POISSON 0.25
DENSITY 77
ALPHA 1.2e-011
DAMP 2.8026e-044
ISOTROPIC MATERIAL2
E 2.5e+007
POISSON 0.25
DENSITY 24
ALPHA 1.2e-011
DAMP 2.8026e-044
END DEFINE MATERIAL
UNIT MMS KN
MEMBER PROPERTY
1 TO 6 PRIS YD 52 ZD 530
ELEMENT PROPERTY
7 13 TO 15 THICKNESS 225
8 10 12 16 TO 19 THICKNESS 22
9 11 THICKNESS 17
SUPPORTS
54 57 PINNED
2 45 FIXED BUT FZ MX MY MZ
UNIT METER KN
CONSTANTS
MATERIAL MATERIAL1 MEMB 1 TO 6 8 TO 12 16 TO 19
MATERIAL MATERIAL2 MEMB 7 13 TO 15
DEFINE MOVING LOAD
TYPE 1 LOAD 87.2 130.8 130.8 185.3 185.3 185.3 185.3 218 218
DIST 3.96 1.52 2.13 1.37 3.05 1.37 2.74 1.22 WID 2.72
LOAD GENERATION 10
TYPE 1 0.07 1.6 0 ZINC 4.54
PERFORM ANALYSIS
PERFORM ANALYSIS PRINT LOAD DATA
FINISH
```

1l-2c-100

```
STAAD SPACE 1L-2C-40
START JOB INFORMATION
ENGINEER DATE 15-Apr-05
END JOB INFORMATION
INPUT WIDTH 79
UNIT MMS KN
JOINT COORDINATES
1 0 1600 0; 2 0 0 0; 3 2270 0 0; 4 2270 1600 0; 45 4540 0 0; 46 4540 1600 0;
47 0 1600 50000; 50 2270 1600 50000; 52 4540 1600 50000; 53 0 1600 100000;
54 0 0 100000; 55 2270 0 100000; 56 2270 1600 100000; 57 4540 0 100000;
58 4540 1600 100000; 59 -1130 1600 0; 61 -1130 1600 100000; 62 5670 1600 0;
64 5670 1600 100000;
MEMBER INCIDENCES
1 1 47; 2 47 53; 3 4 50; 4 50 56; 5 46 52; 6 52 58;
ELEMENT INCIDENCES SHELL
7 59 61 53 1; 8 1 2 54 53; 9 2 54 55 3; 10 3 55 56 4; 11 3 55 57 45;
12 45 57 58 46; 13 46 58 64 62; 14 46 4 56 58; 15 4 1 53 56; 16 53 54 55 56;
17 56 55 57 58; 18 1 2 3 4; 19 4 3 45 46;
UNIT METER KN
DEFINE MATERIAL START
ISOTROPIC MATERIAL1
E 2.05e+008
POISSON 0.25
```

```
DENSITY 77
ALPHA 1.2e-011
DAMP 2.8026e-044
ISOTROPIC MATERIAL2
E 2.5e+007
POISSON 0.25
DENSITY 24
ALPHA 1.2e-011
DAMP 2.8026e-044
END DEFINE MATERIAL
UNIT MMS KN
MEMBER PROPERTY
1 TO 6 PRIS YD 64 ZD 600
ELEMENT PROPERTY
7 13 TO 15 THICKNESS 225
8 10 12 16 TO 19 THICKNESS 26
9 11 THICKNESS 20
SUPPORTS
54 57 PINNED
2 45 FIXED BUT FZ MX MY MZ
UNIT METER KN
CONSTANTS
MATERIAL MATERIAL1 MEMB 1 TO 6 8 TO 12 16 TO 19
MATERIAL MATERIAL2 MEMB 7 13 TO 15
DEFINE MOVING LOAD
TYPE 1 LOAD 87.2 130.8 130.8 185.3 185.3 185.3 185.3 218 218
DIST 3.96 1.52 2.13 1.37 3.05 1.37 2.74 1.22 WID 2.72
LOAD GENERATION 10
TYPE 1 0.07 1.6 0 ZINC 4.54
PERFORM ANALYSIS
PERFORM ANALYSIS PRINT LOAD DATA
FINISH
```

<u>2l-3c-20</u>

```
STAAD SPACE 2L-3C-40
START JOB INFORMATION
ENGINEER DATE 15-Apr-05
END JOB INFORMATION
INPUT WIDTH 79
UNIT MMS KN
JOINT COORDINATES
1 0 1600 0; 2 0 0 0; 3 2325 0 0; 4 2325 1600 0; 45 4650 0 0; 46 4650 1600 0;
67 6975 0 0; 68 6975 1600 0; 89 -1162.5 1600 0; 100 8137.5 1600 0;
101 0 1600 10000; 102 2325 1600 10000; 103 4650 1600 10000;
104 6975 1600 10000; 107 0 1600 20000; 108 2325 1600 20000;
109 4650 1600 20000; 110 6975 1600 20000; 111 -1162.5 1600 20000;
112 8137.5 1600 20000; 113 0 0 20000; 114 2325 0 20000; 115 4650 0 20000;
116 6975 0 20000;
MEMBER INCIDENCES
81 1 101; 82 101 107; 83 4 102; 84 102 108; 85 46 103; 86 103 109; 87 68 104;
88 104 110;
ELEMENT INCIDENCES SHELL
89 89 1 107 111; 90 1 2 113 107; 91 3 2 113 114; 92 4 3 114 108;
93 45 3 114 115; 94 46 45 115 109; 95 67 45 115 116; 96 68 67 116 110;
97 100 68 110 112; 98 68 46 109 110; 99 46 4 108 109; 100 4 1 107 108;
101 107 113 114 108; 102 108 114 115 109; 103 109 115 116 110; 104 1 2 3 4;
105 4 3 45 46; 106 46 45 67 68;
MEMBER PROPERTY AMERICAN
81 TO 88 PRIS YD 16 ZD 300
ELEMENT PROPERTY
89 97 TO 100 THICKNESS 225
90 92 94 96 101 TO 106 THICKNESS 10
91 93 95 THICKNESS 10
CONSTANTS
E 2.5e+007 MEMB 89 97 TO 100
E 2.05e+008 MEMB 81 TO 88 90 TO 96 101 TO 106
POISSON 0.25 MEMB 81 TO 106
DENSITY 77 MEMB 81 TO 88 90 TO 96 101 TO 106
ALPHA 1.2e-011 MEMB 81 TO 106
DENSITY 24 MEMB 89 97 TO 100
SUPPORTS
113 116 PINNED
2 67 FIXED BUT FZ MX MY MZ
```

```
DEFINE MOVING LOAD
TYPE 1 LOAD 93.2 139.8 139.8 198.1 198.1 198.1 198.1
DIST 3960 1520 2130 1370 3050 1370 WID 2720
LOAD GENERATION 10
TYPE 1 1397.5 1600 0 ZINC 6975
PERFORM ANALYSIS
FINISH
```

2l-3c-40

```
STAAD SPACE 2L-3C-40
START JOB INFORMATION
ENGINEER DATE 15-Apr-05
END JOB INFORMATION
INPUT WIDTH 79
UNIT MMS KN
JOINT COORDINATES
1 0 1600 0; 2 0 0 0; 3 2325 0 0; 4 2325 1600 0; 45 4650 0 0; 46 4650 1600 0;
67 6975 0 0; 68 6975 1600 0; 89 -1162.5 1600 0; 100 8137.5 1600 0;
101 0 1600 20000; 102 2325 1600 20000; 103 4650 1600 20000;
104 6975 1600 20000; 107 0 1600 40000; 108 2325 1600 40000;
109 4650 1600 40000; 110 6975 1600 40000; 111 -1162.5 1600 40000;
112 8137.5 1600 40000; 113 0 0 40000; 114 2325 0 40000; 115 4650 0 40000;
116 6975 0 40000;
MEMBER INCIDENCES
81 1 101; 82 101 107; 83 4 102; 84 102 108; 85 46 103; 86 103 109; 87 68 104;
88 104 110;
ELEMENT INCIDENCES SHELL
89 89 1 107 111; 90 1 2 113 107; 91 3 2 113 114; 92 4 3 114 108;
93 45 3 114 115; 94 46 45 115 109; 95 67 45 115 116; 96 68 67 116 110;
97 100 68 110 112; 98 68 46 109 110; 99 46 4 108 109; 100 4 1 107 108;
101 107 113 114 108; 102 108 114 115 109; 103 109 115 116 110; 104 1 2 3 4;
105 4 3 45 46; 106 46 45 67 68;
DEFINE MATERIAL START
ISOTROPIC MATERIAL1
E 2.05e+008
POISSON 0.25
DENSITY 77
ALPHA 1.2e-011
DAMP 2.8026e-044
ISOTROPIC MATERIAL2
E 2.5e+007
POISSON 0.25
DENSITY 24
ALPHA 1.2e-011
DAMP 2.8026e-044
END DEFINE MATERIAL
MEMBER PROPERTY
81 TO 88 PRIS YD 28 ZD 375
ELEMENT PROPERTY
89 97 TO 100 THICKNESS 225
90 92 94 96 101 TO 106 THICKNESS 14
91 93 95 THICKNESS 12
CONSTANTS
MATERIAL MATERIAL1 MEMB 81 TO 88 90 TO 96 101 TO 106
MATERIAL MATERIAL2 MEMB 89 97 TO 100
SUPPORTS
113 116 PINNED
2 67 FIXED BUT FZ MX MY MZ
DEFINE MOVING LOAD
TYPE 1 LOAD 87.2 130.8 130.8 185.3 185.3 185.3 185.3 218 218
DIST 3960 1520 2130 1370 3050 1370 2740 1220 WID 2720
LOAD GENERATION 15
TYPE 1 1397.5 1600 0 ZINC 6975
PERFORM ANALYSIS
PERFORM ANALYSIS PRINT STATICS LOAD
FINISH
```

2l-3c-60

```
STAAD SPACE 2L-3C-40
START JOB INFORMATION
ENGINEER DATE 15-Apr-05
END JOB INFORMATION
INPUT WIDTH 79
UNIT MMS KN
```

```
JOINT COORDINATES
1 0 1600 0; 2 0 0 0; 3 2325 0 0; 4 2325 1600 0; 45 4650 0 0; 46 4650 1600 0;
67 6975 0 0; 68 6975 1600 0; 89 -1162.5 1600 0; 100 8137.5 1600 0;
101 0 1600 30000; 102 2325 1600 30000; 103 4650 1600 30000;
104 6975 1600 30000; 107 0 1600 60000; 108 2325 1600 60000;
109 4650 1600 60000; 110 6975 1600 60000; 111 -1162.5 1600 60000;
112 8137.5 1600 60000; 113 0 0 60000; 114 2325 0 60000; 115 4650 0 60000;
116 6975 0 60000;
MEMBER INCIDENCES
81 1 101; 82 101 107; 83 4 102; 84 102 108; 85 46 103; 86 103 109; 87 68 104;
88 104 110;
ELEMENT INCIDENCES SHELL
89 89 1 107 111; 90 1 2 113 107; 91 3 2 113 114; 92 4 3 114 108;
93 45 3 114 115; 94 46 45 115 109; 95 67 45 115 116; 96 68 67 116 110;
97 100 68 110 112; 98 68 46 109 110; 99 46 4 108 109; 100 4 1 107 108;
101 107 113 114 108; 102 108 114 115 109; 103 109 115 116 110; 104 1 2 3 4;
105 4 3 45 46; 106 46 45 67 68;
DEFINE MATERIAL START
ISOTROPIC MATERIAL1
E 2.05e+008
POISSON 0.25
DENSITY 77
ALPHA 1.2e-011
DAMP 2.8026e-044
ISOTROPIC MATERIAL2
E 2.5e+007
POISSON 0.25
DENSITY 24
ALPHA 1.2e-011
DAMP 2.8026e-044
END DEFINE MATERIAL
MEMBER PROPERTY
81 TO 88 PRIS YD 40 ZD 450
ELEMENT PROPERTY
89 97 TO 100 THICKNESS 225
90 92 94 96 101 TO 106 THICKNESS 18
91 93 95 THICKNESS 15
CONSTANTS
MATERIAL MATERIAL1 MEMB 81 TO 88 90 TO 96 101 TO 106
MATERIAL MATERIAL2 MEMB 89 97 TO 100
SUPPORTS
113 116 PINNED
2 67 FIXED BUT FZ MX MY MZ
DEFINE MOVING LOAD
TYPE 1 LOAD 87.2 130.8 130.8 185.3 185.3 185.3 185.3 218 218
DIST 3960 1520 2130 1370 3050 1370 2740 1220 WID 2720
LOAD GENERATION 15
TYPE 1 1397.5 1600 0 ZINC 6975
PERFORM ANALYSIS
FINISH
```

2l-3c-80

```
STAAD SPACE 2L-3C-40
START JOB INFORMATION
ENGINEER DATE 15-Apr-05
END JOB INFORMATION
INPUT WIDTH 79
UNIT MMS KN
JOINT COORDINATES
1 0 1600 0; 2 0 0 0; 3 2325 0 0; 4 2325 1600 0; 45 4650 0 0; 46 4650 1600 0;
67 6975 0 0; 68 6975 1600 0; 89 -1162.5 1600 0; 100 8137.5 1600 0;
101 0 1600 40000; 102 2325 1600 40000; 103 4650 1600 40000;
104 6975 1600 40000; 107 0 1600 80000; 108 2325 1600 80000;
109 4650 1600 80000; 110 6975 1600 80000; 111 -1162.5 1600 80000;
112 8137.5 1600 80000; 113 0 0 80000; 114 2325 0 80000; 115 4650 0 80000;
116 6975 0 80000;
MEMBER INCIDENCES
81 1 101; 82 101 107; 83 4 102; 84 102 108; 85 46 103; 86 103 109; 87 68 104;
88 104 110;
ELEMENT INCIDENCES SHELL
89 89 1 107 111; 90 1 2 113 107; 91 3 2 113 114; 92 4 3 114 108;
93 45 3 114 115; 94 46 45 115 109; 95 67 45 115 116; 96 68 67 116 110;
97 100 68 110 112; 98 68 46 109 110; 99 46 4 108 109; 100 4 1 107 108;
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101 107 113 114 108; 102 108 114 115 109; 103 109 115 116 110; 104 1 2 3 4;
105 4 3 45 46; 106 46 45 67 68;
MEMBER PROPERTY
81 TO 88 PRIS YD 52 ZD 530
ELEMENT PROPERTY
89 97 TO 100 THICKNESS 225
90 92 94 96 101 TO 106 THICKNESS 22
91 93 95 THICKNESS 17
CONSTANTS
E 2.5e+007 MEMB 89 97 TO 100
E 2.05e+008 MEMB 81 TO 88 90 TO 96 101 TO 106
POISSON 0.25 MEMB 81 TO 106
DENSITY 77 MEMB 81 TO 88 90 TO 96 101 TO 106
ALPHA 1.2e-011 MEMB 81 TO 106
DENSITY 24 MEMB 89 97 TO 100
SUPPORTS
113 116 PINNED
2 67 FIXED BUT FZ MX MY MZ
DEFINE MOVING LOAD
TYPE 1 LOAD 87.2 130.8 130.8 185.3 185.3 185.3 185.3 218 218
DIST 3960 1520 2130 1370 3050 1370 2740 1220 WID 2720
LOAD GENERATION 15
TYPE 1 1397.5 1600 0 ZINC 6975
PERFORM ANALYSIS
FINISH
```

2l-3c-100

```
STAAD SPACE 2L-3C-40
START JOB INFORMATION
ENGINEER DATE 15-Apr-05
END JOB INFORMATION
INPUT WIDTH 79
UNIT MMS KN
JOINT COORDINATES
1 0 1600 0; 2 0 0 0; 3 2325 0 0; 4 2325 1600 0; 45 4650 0 0; 46 4650 1600 0;
67 6975 0 0; 68 6975 1600 0; 89 -1162.5 1600 0; 100 8137.5 1600 0;
101 0 1600 50000; 102 2325 1600 50000; 103 4650 1600 50000;
104 6975 1600 50000; 107 0 1600 100000; 108 2325 1600 100000;
109 4650 1600 100000; 110 6975 1600 100000; 111 -1162.5 1600 100000;
112 8137.5 1600 100000; 113 0 0 100000; 114 2325 0 100000; 115 4650 0 100000;
116 6975 0 100000;
MEMBER INCIDENCES
81 1 101; 82 101 107; 83 4 102; 84 102 108; 85 46 103; 86 103 109; 87 68 104;
88 104 110;
ELEMENT INCIDENCES SHELL
89 89 1 107 111; 90 1 2 113 107; 91 3 2 113 114; 92 4 3 114 108;
93 45 3 114 115; 94 46 45 115 109; 95 67 45 115 116; 96 68 67 116 110;
97 100 68 110 112; 98 68 46 109 110; 99 46 4 108 109; 100 4 1 107 108;
101 107 113 114 108; 102 108 114 115 109; 103 109 115 116 110; 104 1 2 3 4;
105 4 3 45 46; 106 46 45 67 68;
MEMBER PROPERTY
81 TO 88 PRIS YD 64 ZD 600
ELEMENT PROPERTY
89 97 TO 100 THICKNESS 225
90 92 94 96 101 TO 106 THICKNESS 26
91 93 95 THICKNESS 20
CONSTANTS
E 2.5e+007 MEMB 89 97 TO 100
E 2.05e+008 MEMB 81 TO 88 90 TO 96 101 TO 106
POISSON 0.25 MEMB 81 TO 106
DENSITY 77 MEMB 81 TO 88 90 TO 96 101 TO 106
ALPHA 1.2e-011 MEMB 81 TO 106
DENSITY 24 MEMB 89 97 TO 100
SUPPORTS
113 116 PINNED
2 67 FIXED BUT FZ MX MY MZ
DEFINE MOVING LOAD
TYPE 1 LOAD 87.2 130.8 130.8 185.3 185.3 185.3 185.3 218 218
DIST 3960 1520 2130 1370 3050 1370 2740 1220 WID 2720
LOAD GENERATION 15
TYPE 1 1397.5 1600 0 ZINC 6975
PERFORM ANALYSIS
FINISH
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1l-3c-100c

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STAAD SPACE 2L-3C-40
START JOB INFORMATION
ENGINEER DATE 15-Apr-05
END JOB INFORMATION
INPUT WIDTH 79
UNIT MMS KN
JOINT COORDINATES
1 0 1600 0; 2 0 0 0; 3 2325 0 0; 4 2325 1600 0; 45 4650 0 0; 46 4650 1600 0;
67 6975 0 0; 68 6975 1600 0; 89 -1162.5 1600 0; 100 8137.5 1600 0;
101 0 1600 50000; 102 2325 1600 50000; 103 4650 1600 50000;
104 6975 1600 50000; 107 0 1600 100000; 108 2325 1600 100000;
109 4650 1600 100000; 110 6975 1600 100000; 111 -1162.5 1600 100000;
112 8137.5 1600 100000; 113 0 0 100000; 114 2325 0 100000; 115 4650 0 100000;
116 6975 0 100000;
MEMBER INCIDENCES
81 1 101; 82 101 107; 83 4 102; 84 102 108; 85 46 103; 86 103 109; 87 68 104;
88 104 110;
ELEMENT INCIDENCES SHELL
89 89 1 107 111; 90 1 2 113 107; 91 3 2 113 114; 92 4 3 114 108;
93 45 3 114 115; 94 46 45 115 109; 95 67 45 115 116; 96 68 67 116 110;
97 100 68 110 112; 98 68 46 109 110; 99 46 4 108 109; 100 4 1 107 108;
101 107 113 114 108; 102 108 114 115 109; 103 109 115 116 110; 104 1 2 3 4;
105 4 3 45 46; 106 46 45 67 68;
MEMBER PROPERTY
81 TO 88 PRIS YD 64 ZD 600
ELEMENT PROPERTY
89 97 TO 100 THICKNESS 225
90 92 94 96 101 TO 106 THICKNESS 26
91 93 95 THICKNESS 20
CONSTANTS
E 2.5e+007 MEMB 89 97 TO 100
E 2.05e+008 MEMB 81 TO 88 90 TO 96 101 TO 106
POISSON 0.25 MEMB 81 TO 106
DENSITY 77 MEMB 81 TO 88 90 TO 96 101 TO 106
ALPHA 1.2e-011 MEMB 81 TO 106
DENSITY 24 MEMB 89 97 TO 100
SUPPORTS
113 116 PINNED
2 67 FIXED BUT FZ MX MY MZ
DEFINE MOVING LOAD
TYPE 1 LOAD 87.2 130.8 130.8 185.3 185.3 185.3 185.3 218 218
DIST 3960 1520 2130 1370 3050 1370 2740 1220 WID 2720
LOAD GENERATION 15
TYPE 1 3487.5 1600 0 ZINC 6975
PERFORM ANALYSIS
FINISH
```

2l-5c-60c

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STAAD SPACE 3L-5C-40
START JOB INFORMATION
ENGINEER DATE 15-Apr-05
END JOB INFORMATION
INPUT WIDTH 79
UNIT MMS KN
JOINT COORDINATES
1 0 1600 0; 2 0 0 0; 3 2175 0 0; 4 2175 1600 0; 85 4350 0 0; 86 4350 1600 0;
127 6525 0 0; 128 6525 1600 0; 169 8700 0 0; 170 8700 1600 0; 211 10875 0 0;
212 10875 1600 0; 253 -1087.5 1600 0; 274 11962.5 1600 0; 275 0 1600 30000;
276 2175 1600 30000; 277 4350 1600 30000; 278 6525 1600 30000;
279 8700 1600 30000; 280 10875 1600 30000; 283 0 1600 60000;
284 2175 1600 60000; 285 4350 1600 60000; 286 6525 1600 60000;
287 8700 1600 60000; 288 10875 1600 60000; 289 -1087.5 1600 60000;
290 11962.5 1600 60000; 291 0 0 60000; 292 2175 0 60000; 293 4350 0 60000;
294 6525 0 60000; 295 8700 0 60000; 296 10875 0 60000;
MEMBER INCIDENCES
486 1 275; 487 275 283; 488 4 276; 489 276 284; 490 86 277; 491 277 285;
492 128 278; 493 278 286; 494 170 279; 495 279 287; 496 212 280; 497 280 288;
ELEMENT INCIDENCES SHELL
498 1 253 289 283; 499 1 2 291 283; 500 3 2 291 292; 501 4 3 292 284;
502 85 3 292 293; 503 86 85 293 285; 504 127 85 293 294; 505 128 127 294 286;
506 169 127 294 295; 507 170 169 295 287; 508 211 169 295 296;
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509 212 211 296 288; 510 274 212 288 290; 511 212 170 287 288;
512 170 128 286 287; 513 128 86 285 286; 514 86 4 284 285; 515 4 1 283 284;
516 283 291 292 284; 517 284 292 293 285; 518 285 293 294 286;
519 286 294 295 287; 520 287 295 296 288; 521 1 2 3 4; 522 4 3 85 86;
523 86 85 127 128; 524 128 127 169 170; 525 170 169 211 212;
MEMBER PROPERTY
486 TO 497 PRIS YD 40 ZD 450
ELEMENT PROPERTY
498 510 TO 515 THICKNESS 225
499 501 503 505 507 509 516 TO 525 THICKNESS 18
500 502 504 506 508 THICKNESS 15
UNIT METER KN
CONSTANTS
E 2.5e+007 MEMB 498 510 TO 515
E 2.05e+008 MEMB 486 TO 497 499 TO 509 516 TO 525
POISSON 0.25 MEMB 486 TO 525
DENSITY 77 MEMB 486 TO 497 499 TO 509 516 TO 525
ALPHA 1.2e-011 MEMB 486 TO 525
DENSITY 24 MEMB 498 510 TO 515
UNIT MMS KN
SUPPORTS
291 296 PINNED
2 211 FIXED BUT FZ MX MY MZ
DEFINE MOVING LOAD
TYPE 1 LOAD 87.2 130.8 130.8 185.3 185.3 185.3 185.3 218 218
DIST 3960 1520 2130 1370 3050 1370 2740 1220 WID 2720
LOAD GENERATION 15
TYPE 1 2877.5 1600 0 ZINC 6525
TYPE 1 7997.5 1600 0 ZINC 6525
PERFORM ANALYSIS
PERFORM ANALYSIS PRINT LOAD DATA
FINISH
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2l-7c-60c

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STAAD SPACE 4L-7C-20
START JOB INFORMATION
ENGINEER DATE 07-Apr-05
END JOB INFORMATION
INPUT WIDTH 79
UNIT MMS KN
JOINT COORDINATES
1 0 800 0; 2 0 0 0; 3 2100 0 0; 4 2100 800 0; 5 4200 0 0; 6 4200 800 0;
7 6300 0 0; 8 6300 800 0; 9 8400 0 0; 10 8400 800 0; 11 10500 0 0;
12 10500 800 0; 13 12600 0 0; 14 12600 800 0; 15 14700 0 0; 16 14700 800 0;
337 -1050 800 0; 358 15750 800 0; 359 0 800 30000; 360 2100 800 30000;
361 4200 800 30000; 362 6300 800 30000; 363 8400 800 30000;
364 10500 800 30000; 365 12600 800 30000; 366 14700 800 30000; 369 0 800 60000;
370 2100 800 60000; 371 4200 800 60000; 372 6300 800 60000; 373 8400 800 60000;
374 10500 800 60000; 375 12600 800 60000; 376 14700 800 60000;
377 -1050 800 60000; 378 15750 800 60000; 379 0 0 60000; 380 2100 0 60000;
381 4200 0 60000; 382 6300 0 60000; 383 8400 0 60000; 384 10500 0 60000;
385 12600 0 60000; 386 14700 0 60000;
MEMBER INCIDENCES
1 1 359; 2 359 369; 3 4 360; 4 360 370; 5 6 361; 6 361 371; 7 8 362; 8 362 372;
9 10 363; 10 363 373; 11 12 364; 12 364 374; 13 14 365; 14 365 375; 15 16 366;
16 366 376;
ELEMENT INCIDENCES SHELL
17 1 337 377 369; 18 1 2 379 369; 19 2 3 380 379; 20 3 4 370 380;
21 3 5 381 380; 22 5 6 371 381; 23 5 7 382 381; 24 7 8 372 382; 25 7 9 383 382;
26 9 10 373 383; 27 9 11 384 383; 28 11 12 374 384; 29 11 13 385 384;
30 13 14 375 385; 31 13 15 386 385; 32 15 16 376 386; 33 16 358 378 376;
34 16 14 375 376; 35 14 12 374 375; 36 12 10 373 374; 37 10 8 372 373;
38 8 6 371 372; 39 6 4 370 371; 40 4 1 369 370; 41 369 379 380 370;
42 370 380 381 371; 43 371 381 382 372; 44 372 382 383 373; 45 373 383 384 374;
46 374 384 385 375; 47 375 385 386 376; 48 1 2 3 4; 49 4 3 5 6; 50 6 5 7 8;
51 8 7 9 10; 52 10 9 11 12; 53 12 11 13 14; 54 14 13 15 16;
MEMBER PROPERTY
1 TO 16 PRIS YD 40 ZD 450
ELEMENT PROPERTY
17 33 TO 40 THICKNESS 225
18 20 22 24 26 28 30 32 41 TO 54 THICKNESS 18
19 21 23 25 27 29 31 THICKNESS 15
UNIT METER KN
CONSTANTS
```

E 2.5e+007 MEMB 17 33 TO 40
E 2.05e+008 MEMB 1 TO 16 18 TO 32 41 TO 54
POISSON 0.25 MEMB 1 TO 54
DENSITY 77 MEMB 1 TO 16 18 TO 32 41 TO 54
ALPHA 1.2e-011 MEMB 1 TO 54
DENSITY 24 MEMB 17 33 TO 40
UNIT MMS KN
SUPPORTS
2 15 FIXED BUT FZ MX MY MZ
379 386 PINNED
DEFINE MOVING LOAD
TYPE 1 LOAD 87.2 130.8 130.8 185.3 185.3 185.3 185.3 218 218
DIST 3960 1520 2130 1370 3050 1370 2740 1220 WID 2720
LOAD GENERATION 15
TYPE 1 4790 800 0 ZINC 6300
TYPE 1 9910 800 0 ZINC 6300
PERFORM ANALYSIS
PERFORM ANALYSIS PRINT LOAD DATA
FINISH

3l-5c-20

STAAD SPACE 3L-5C-40
START JOB INFORMATION
ENGINEER DATE 15-Apr-05
END JOB INFORMATION
INPUT WIDTH 79
UNIT MMS KN
JOINT COORDINATES
1 0 1600 0; 2 0 0 0; 3 2175 0 0; 4 2175 1600 0; 85 4350 0 0; 86 4350 1600 0;
127 6525 0 0; 128 6525 1600 0; 169 8700 0 0; 170 8700 1600 0; 211 10875 0 0;
212 10875 1600 0; 253 -1087.5 1600 0; 274 11962.5 1600 0; 275 0 1600 10000;
276 2175 1600 10000; 277 4350 1600 10000; 278 6525 1600 10000;
279 8700 1600 10000; 280 10875 1600 10000; 283 0 1600 20000;
284 2175 1600 20000; 285 4350 1600 20000; 286 6525 1600 20000;
287 8700 1600 20000; 288 10875 1600 20000; 289 -1087.5 1600 20000;
290 11962.5 1600 20000; 291 0 0 20000; 292 2175 0 20000; 293 4350 0 20000;
294 6525 0 20000; 295 8700 0 20000; 296 10875 0 20000;
MEMBER INCIDENCES
486 1 275; 487 275 283; 488 4 276; 489 276 284; 490 86 277; 491 277 285;
492 128 278; 493 278 286; 494 170 279; 495 279 287; 496 212 280; 497 280 288;
ELEMENT INCIDENCES SHELL
498 1 253 289 283; 499 1 2 291 283; 500 3 2 291 292; 501 4 3 292 284;
502 85 3 292 293; 503 86 85 293 285; 504 127 85 293 294; 505 128 127 294 286;
506 169 127 294 295; 507 170 169 295 287; 508 211 169 295 296;
509 212 211 296 288; 510 274 212 288 290; 511 212 170 287 288;
512 170 128 286 287; 513 128 86 285 286; 514 86 4 284 285; 515 4 1 283 284;
516 283 291 292 284; 517 284 292 293 285; 518 285 293 294 286;
519 286 294 295 287; 520 287 295 296 288; 521 1 2 3 4; 522 4 3 85 86;
523 86 85 127 128; 524 128 127 169 170; 525 170 169 211 212;
MEMBER PROPERTY
486 TO 497 PRIS YD 16 ZD 300
ELEMENT PROPERTY
498 510 TO 515 THICKNESS 225
499 TO 509 516 TO 525 THICKNESS 10
UNIT METER KN
CONSTANTS
E 2.5e+007 MEMB 498 510 TO 515
E 2.05e+008 MEMB 486 TO 497 499 TO 509 516 TO 525
POISSON 0.25 MEMB 486 TO 525
DENSITY 77 MEMB 486 TO 497 499 TO 509 516 TO 525
ALPHA 1.2e-011 MEMB 486 TO 525
DENSITY 24 MEMB 498 510 TO 515
UNIT MMS KN
SUPPORTS
291 296 PINNED
2 211 FIXED BUT FZ MX MY MZ
DEFINE MOVING LOAD
TYPE 1 LOAD 93.2 139.8 139.8 198.1 198.1 198.1 198.1
DIST 3960 1520 2130 1370 3050 1370 WID 2720
LOAD GENERATION 10
TYPE 1 1472.5 1600 0 ZINC 6525
TYPE 1 5392.5 1600 0 ZINC 6525
PERFORM ANALYSIS
PERFORM ANALYSIS PRINT LOAD DATA

FINISH

3l-5c-40

```
STAAD SPACE 3L-5C-40
START JOB INFORMATION
ENGINEER DATE 15-Apr-05
END JOB INFORMATION
INPUT WIDTH 79
UNIT MMS KN
JOINT COORDINATES
1 0 1600 0; 2 0 0 0; 3 2175 0 0; 4 2175 1600 0; 85 4350 0 0; 86 4350 1600 0;
127 6525 0 0; 128 6525 1600 0; 169 8700 0 0; 170 8700 1600 0; 211 10875 0 0;
212 10875 1600 0; 253 -1087.5 1600 0; 274 11962.5 1600 0; 275 0 1600 20000;
276 2175 1600 20000; 277 4350 1600 20000; 278 6525 1600 20000;
279 8700 1600 20000; 280 10875 1600 20000; 283 0 1600 40000;
284 2175 1600 40000; 285 4350 1600 40000; 286 6525 1600 40000;
287 8700 1600 40000; 288 10875 1600 40000; 289 -1087.5 1600 40000;
290 11962.5 1600 40000; 291 0 0 40000; 292 2175 0 40000; 293 4350 0 40000;
294 6525 0 40000; 295 8700 0 40000; 296 10875 0 40000;
MEMBER INCIDENCES
486 1 275; 487 275 283; 488 4 276; 489 276 284; 490 86 277; 491 277 285;
492 128 278; 493 278 286; 494 170 279; 495 279 287; 496 212 280; 497 280 288;
ELEMENT INCIDENCES SHELL
498 1 253 289 283; 499 1 2 291 283; 500 3 2 291 292; 501 4 3 292 284;
502 85 3 292 293; 503 86 85 293 285; 504 127 85 293 294; 505 128 127 294 286;
506 169 127 294 295; 507 170 169 295 287; 508 211 169 295 296;
509 212 211 296 288; 510 274 212 288 290; 511 212 170 287 288;
512 170 128 286 287; 513 128 86 285 286; 514 86 4 284 285; 515 4 1 283 284;
516 283 291 292 284; 517 284 292 293 285; 518 285 293 294 286;
519 286 294 295 287; 520 287 295 296 288; 521 1 2 3 4; 522 4 3 85 86;
523 86 85 127 128; 524 128 127 169 170; 525 170 169 211 212;
MEMBER PROPERTY
486 TO 497 PRIS YD 28 ZD 375
ELEMENT PROPERTY
498 510 TO 515 THICKNESS 225
499 501 503 505 507 509 516 TO 525 THICKNESS 14
500 502 504 506 508 THICKNESS 12
UNIT METER KN
CONSTANTS
E 2.5e+007 MEMB 498 510 TO 515
E 2.05e+008 MEMB 486 TO 497 499 TO 509 516 TO 525
POISSON 0.25 MEMB 486 TO 525
DENSITY 77 MEMB 486 TO 497 499 TO 509 516 TO 525
ALPHA 1.2e-011 MEMB 486 TO 525
DENSITY 24 MEMB 498 510 TO 515
UNIT MMS KN
SUPPORTS
291 296 PINNED
2 211 FIXED BUT FZ MX MY MZ
DEFINE MOVING LOAD
TYPE 1 LOAD 87.2 130.8 130.8 185.3 185.3 185.3 185.3 218 218
DIST 3960 1520 2130 1370 3050 1370 2740 1220 WID 2720
LOAD GENERATION 15
TYPE 1 1472.5 1600 0 ZINC 6525
TYPE 1 5392.5 1600 0 ZINC 6525
PERFORM ANALYSIS
PERFORM ANALYSIS PRINT LOAD DATA
FINISH
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3l-5c-60

```
STAAD SPACE 3L-5C-40
START JOB INFORMATION
ENGINEER DATE 15-Apr-05
END JOB INFORMATION
INPUT WIDTH 79
UNIT MMS KN
JOINT COORDINATES
1 0 1600 0; 2 0 0 0; 3 2175 0 0; 4 2175 1600 0; 85 4350 0 0; 86 4350 1600 0;
127 6525 0 0; 128 6525 1600 0; 169 8700 0 0; 170 8700 1600 0; 211 10875 0 0;
212 10875 1600 0; 253 -1087.5 1600 0; 274 11962.5 1600 0; 275 0 1600 30000;
276 2175 1600 30000; 277 4350 1600 30000; 278 6525 1600 30000;
279 8700 1600 30000; 280 10875 1600 30000; 283 0 1600 60000;
284 2175 1600 60000; 285 4350 1600 60000; 286 6525 1600 60000;
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287 8700 1600 60000; 288 10875 1600 60000; 289 -1087.5 1600 60000;
290 11962.5 1600 60000; 291 0 0 60000; 292 2175 0 60000; 293 4350 0 60000;
294 6525 0 60000; 295 8700 0 60000; 296 10875 0 60000;
MEMBER INCIDENCES
486 1 275; 487 275 283; 488 4 276; 489 276 284; 490 86 277; 491 277 285;
492 128 278; 493 278 286; 494 170 279; 495 279 287; 496 212 280; 497 280 288;
ELEMENT INCIDENCES SHELL
498 1 253 289 283; 499 1 2 291 283; 500 3 2 291 292; 501 4 3 292 284;
502 85 3 292 293; 503 86 85 293 285; 504 127 85 293 294; 505 128 127 294 286;
506 169 127 294 295; 507 170 169 295 287; 508 211 169 295 296;
509 212 211 296 288; 510 274 212 288 290; 511 212 170 287 288;
512 170 128 286 287; 513 128 86 285 286; 514 86 4 284 285; 515 4 1 283 284;
516 283 291 292 284; 517 284 292 293 285; 518 285 293 294 286;
519 286 294 295 287; 520 287 295 296 288; 521 1 2 3 4; 522 4 3 85 86;
523 86 85 127 128; 524 128 127 169 170; 525 170 169 211 212;
MEMBER PROPERTY
486 TO 497 PRIS YD 40 ZD 450
ELEMENT PROPERTY
498 510 TO 515 THICKNESS 225
499 501 503 505 507 509 516 TO 525 THICKNESS 18
500 502 504 506 508 THICKNESS 15
UNIT METER KN
CONSTANTS
E 2.5e+007 MEMB 498 510 TO 515
E 2.05e+008 MEMB 486 TO 497 499 TO 509 516 TO 525
POISSON 0.25 MEMB 486 TO 525
DENSITY 77 MEMB 486 TO 497 499 TO 509 516 TO 525
ALPHA 1.2e-011 MEMB 486 TO 525
DENSITY 24 MEMB 498 510 TO 515
UNIT MMS KN
SUPPORTS
291 296 PINNED
2 211 FIXED BUT FZ MX MY MZ
DEFINE MOVING LOAD
TYPE 1 LOAD 87.2 130.8 130.8 185.3 185.3 185.3 185.3 218 218
DIST 3960 1520 2130 1370 3050 1370 2740 1220 WID 2720
LOAD GENERATION 15
TYPE 1 1472.5 1600 0 ZINC 6525
TYPE 1 5392.5 1600 0 ZINC 6525
PERFORM ANALYSIS
PERFORM ANALYSIS PRINT LOAD DATA
FINISH

3l-5c-80

STAAD SPACE 3L-5C-40
START JOB INFORMATION
ENGINEER DATE 15-Apr-05
END JOB INFORMATION
INPUT WIDTH 79
UNIT MMS KN
JOINT COORDINATES
1 0 1600 0; 2 0 0 0; 3 2175 0 0; 4 2175 1600 0; 85 4350 0 0; 86 4350 1600 0;
127 6525 0 0; 128 6525 1600 0; 169 8700 0 0; 170 8700 1600 0; 211 10875 0 0;
212 10875 1600 0; 253 -1087.5 1600 0; 274 11962.5 1600 0; 275 0 1600 40000;
276 2175 1600 40000; 277 4350 1600 40000; 278 6525 1600 40000;
279 8700 1600 40000; 280 10875 1600 40000; 283 0 1600 80000;
284 2175 1600 80000; 285 4350 1600 80000; 286 6525 1600 80000;
287 8700 1600 80000; 288 10875 1600 80000; 289 -1087.5 1600 80000;
290 11962.5 1600 80000; 291 0 0 80000; 292 2175 0 80000; 293 4350 0 80000;
294 6525 0 80000; 295 8700 0 80000; 296 10875 0 80000;
MEMBER INCIDENCES
486 1 275; 487 275 283; 488 4 276; 489 276 284; 490 86 277; 491 277 285;
492 128 278; 493 278 286; 494 170 279; 495 279 287; 496 212 280; 497 280 288;
ELEMENT INCIDENCES SHELL
498 1 253 289 283; 499 1 2 291 283; 500 3 2 291 292; 501 4 3 292 284;
502 85 3 292 293; 503 86 85 293 285; 504 127 85 293 294; 505 128 127 294 286;
506 169 127 294 295; 507 170 169 295 287; 508 211 169 295 296;
509 212 211 296 288; 510 274 212 288 290; 511 212 170 287 288;
512 170 128 286 287; 513 128 86 285 286; 514 86 4 284 285; 515 4 1 283 284;
516 283 291 292 284; 517 284 292 293 285; 518 285 293 294 286;
519 286 294 295 287; 520 287 295 296 288; 521 1 2 3 4; 522 4 3 85 86;
523 86 85 127 128; 524 128 127 169 170; 525 170 169 211 212;
MEMBER PROPERTY
486 TO 497 PRIS YD 52 ZD 530

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ELEMENT PROPERTY
498 510 TO 515 THICKNESS 225
499 501 503 505 507 509 516 TO 525 THICKNESS 22
500 502 504 506 508 THICKNESS 17
UNIT METER KN
CONSTANTS
E 2.5e+007 MEMB 498 510 TO 515
E 2.05e+008 MEMB 486 TO 497 499 TO 509 516 TO 525
POISSON 0.25 MEMB 486 TO 525
DENSITY 77 MEMB 486 TO 497 499 TO 509 516 TO 525
ALPHA 1.2e-011 MEMB 486 TO 525
DENSITY 24 MEMB 498 510 TO 515
UNIT MMS KN
SUPPORTS
291 296 PINNED
2 211 FIXED BUT FZ MX MY MZ
DEFINE MOVING LOAD
TYPE 1 LOAD 87.2 130.8 130.8 185.3 185.3 185.3 185.3 218 218
DIST 3960 1520 2130 1370 3050 1370 2740 1220 WID 2720
LOAD GENERATION 15
TYPE 1 1472.5 1600 0 ZINC 6525
TYPE 1 5392.5 1600 0 ZINC 6525
PERFORM ANALYSIS
PERFORM ANALYSIS PRINT LOAD DATA
FINISH
```

3l-5c-100

```
STAAD SPACE 3L-5C-40
START JOB INFORMATION
ENGINEER DATE 15-Apr-05
END JOB INFORMATION
INPUT WIDTH 79
UNIT MMS KN
JOINT COORDINATES
1 0 1600 0; 2 0 0 0; 3 2175 0 0; 4 2175 1600 0; 85 4350 0 0; 86 4350 1600 0;
127 6525 0 0; 128 6525 1600 0; 169 8700 0 0; 170 8700 1600 0; 211 10875 0 0;
212 10875 1600 0; 253 -1087.5 1600 0; 274 11962.5 1600 0; 275 0 1600 50000;
276 2175 1600 50000; 277 4350 1600 50000; 278 6525 1600 50000;
279 8700 1600 50000; 280 10875 1600 50000; 283 0 1600 100000;
284 2175 1600 100000; 285 4350 1600 100000; 286 6525 1600 100000;
287 8700 1600 100000; 288 10875 1600 100000; 289 -1087.5 1600 100000;
290 11962.5 1600 100000; 291 0 0 100000; 292 2175 0 100000; 293 4350 0 100000;
294 6525 0 100000; 295 8700 0 100000; 296 10875 0 100000;
MEMBER INCIDENCES
486 1 275; 487 275 283; 488 4 276; 489 276 284; 490 86 277; 491 277 285;
492 128 278; 493 278 286; 494 170 279; 495 279 287; 496 212 280; 497 280 288;
ELEMENT INCIDENCES SHELL
498 1 253 289 283; 499 1 2 291 283; 500 3 2 291 292; 501 4 3 292 284;
502 85 3 292 293; 503 86 85 293 285; 504 127 85 293 294; 505 128 127 294 286;
506 169 127 294 295; 507 170 169 295 287; 508 211 169 295 296;
509 212 211 296 288; 510 274 212 288 290; 511 212 170 287 288;
512 170 128 286 287; 513 128 86 285 286; 514 86 4 284 285; 515 4 1 283 284;
516 283 291 292 284; 517 284 292 293 285; 518 285 293 294 286;
519 286 294 295 287; 520 287 295 296 288; 521 1 2 3 4; 522 4 3 85 86;
523 86 85 127 128; 524 128 127 169 170; 525 170 169 211 212;
MEMBER PROPERTY
486 TO 497 PRIS YD 64 ZD 600
ELEMENT PROPERTY
498 510 TO 515 THICKNESS 225
499 501 503 505 507 509 516 TO 525 THICKNESS 26
500 502 504 506 508 THICKNESS 20
UNIT METER KN
CONSTANTS
E 2.5e+007 MEMB 498 510 TO 515
E 2.05e+008 MEMB 486 TO 497 499 TO 509 516 TO 525
POISSON 0.25 MEMB 486 TO 525
DENSITY 77 MEMB 486 TO 497 499 TO 509 516 TO 525
ALPHA 1.2e-011 MEMB 486 TO 525
DENSITY 24 MEMB 498 510 TO 515
UNIT MMS KN
SUPPORTS
291 296 PINNED
2 211 FIXED BUT FZ MX MY MZ
DEFINE MOVING LOAD
```

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TYPE 1 LOAD 87.2 130.8 130.8 185.3 185.3 185.3 185.3 218 218
DIST 3960 1520 2130 1370 3050 1370 2740 1220 WID 2720
LOAD GENERATION 15
TYPE 1 1472.5 1600 0 ZINC 6525
TYPE 1 5392.5 1600 0 ZINC 6525
PERFORM ANALYSIS
PERFORM ANALYSIS PRINT LOAD DATA
FINISH
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3l-7c-60c

```
STAAD SPACE 4L-7C-20
START JOB INFORMATION
ENGINEER DATE 07-Apr-05
END JOB INFORMATION
INPUT WIDTH 79
UNIT MMS KN
JOINT COORDINATES
1 0 800 0; 2 0 0 0; 3 2100 0 0; 4 2100 800 0; 5 4200 0 0; 6 4200 800 0;
7 6300 0 0; 8 6300 800 0; 9 8400 0 0; 10 8400 800 0; 11 10500 0 0;
12 10500 800 0; 13 12600 0 0; 14 12600 800 0; 15 14700 0 0; 16 14700 800 0;
337 -1050 800 0; 358 15750 800 0; 359 0 800 30000; 360 2100 800 30000;
361 4200 800 30000; 362 6300 800 30000; 363 8400 800 30000;
364 10500 800 30000; 365 12600 800 30000; 366 14700 800 30000; 369 0 800 60000;
370 2100 800 60000; 371 4200 800 60000; 372 6300 800 60000; 373 8400 800 60000;
374 10500 800 60000; 375 12600 800 60000; 376 14700 800 60000;
377 -1050 800 60000; 378 15750 800 60000; 379 0 0 60000; 380 2100 0 60000;
381 4200 0 60000; 382 6300 0 60000; 383 8400 0 60000; 384 10500 0 60000;
385 12600 0 60000; 386 14700 0 60000;
MEMBER INCIDENCES
1 1 359; 2 359 369; 3 4 360; 4 360 370; 5 6 361; 6 361 371; 7 8 362; 8 362 372;
9 10 363; 10 363 373; 11 12 364; 12 364 374; 13 14 365; 14 365 375; 15 16 366;
16 366 376;
ELEMENT INCIDENCES SHELL
17 1 337 377 369; 18 1 2 379 369; 19 2 3 380 379; 20 3 4 370 380;
21 3 5 381 380; 22 5 6 371 381; 23 5 7 382 381; 24 7 8 372 382; 25 7 9 383 382;
26 9 10 373 383; 27 9 11 384 383; 28 11 12 374 384; 29 11 13 385 384;
30 13 14 375 385; 31 13 15 386 385; 32 15 16 376 386; 33 16 358 378 376;
34 16 14 375 376; 35 14 12 374 375; 36 12 10 373 374; 37 10 8 372 373;
38 8 6 371 372; 39 6 4 370 371; 40 4 1 369 370; 41 369 379 380 370;
42 370 380 381 371; 43 371 381 382 372; 44 372 382 383 373; 45 373 383 384 374;
46 374 384 385 375; 47 375 385 386 376; 48 1 2 3 4; 49 4 3 5 6; 50 6 5 7 8;
51 8 7 9 10; 52 10 9 11 12; 53 12 11 13 14; 54 14 13 15 16;
MEMBER PROPERTY
1 TO 16 PRIS YD 40 ZD 450
ELEMENT PROPERTY
17 33 TO 40 THICKNESS 225
18 20 22 24 26 28 30 32 41 TO 54 THICKNESS 18
19 21 23 25 27 29 31 THICKNESS 15
UNIT METER KN
CONSTANTS
E 2.5e+007 MEMB 17 33 TO 40
E 2.05e+008 MEMB 1 TO 16 18 TO 32 41 TO 54
POISSON 0.25 MEMB 1 TO 54
DENSITY 77 MEMB 1 TO 16 18 TO 32 41 TO 54
ALPHA 1.2e-011 MEMB 1 TO 54
DENSITY 24 MEMB 17 33 TO 40
UNIT MMS KN
SUPPORTS
2 15 FIXED BUT FZ MX MY MZ
379 386 PINNED
DEFINE MOVING LOAD
TYPE 1 LOAD 87.2 130.8 130.8 185.3 185.3 185.3 185.3 218 218
DIST 3960 1520 2130 1370 3050 1370 2740 1220 WID 2720
LOAD GENERATION 15
TYPE 1 3430 800 0 ZINC 6300
TYPE 1 7350 800 0 ZINC 6300
TYPE 1 11270 800 0 ZINC 6300
PERFORM ANALYSIS
PERFORM ANALYSIS PRINT LOAD DATA
FINISH
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4l-7c-20

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STAAD SPACE 4L-7C-20
START JOB INFORMATION
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```
ENGINEER DATE 07-Apr-05
END JOB INFORMATION
INPUT WIDTH 79
UNIT MMS KN
JOINT COORDINATES
1 0 800 0; 2 0 0 0; 3 2100 0 0; 4 2100 800 0; 5 4200 0 0; 6 4200 800 0;
7 6300 0 0; 8 6300 800 0; 9 8400 0 0; 10 8400 800 0; 11 10500 0 0;
12 10500 800 0; 13 12600 0 0; 14 12600 800 0; 15 14700 0 0; 16 14700 800 0;
337 -1050 800 0; 358 15750 800 0; 359 0 800 10000; 360 2100 800 10000;
361 4200 800 10000; 362 6300 800 10000; 363 8400 800 10000;
364 10500 800 10000; 365 12600 800 10000; 366 14700 800 10000; 369 0 800 20000;
370 2100 800 20000; 371 4200 800 20000; 372 6300 800 20000; 373 8400 800 20000;
374 10500 800 20000; 375 12600 800 20000; 376 14700 800 20000;
377 -1050 800 20000; 378 15750 800 20000; 379 0 0 20000; 380 2100 0 20000;
381 4200 0 20000; 382 6300 0 20000; 383 8400 0 20000; 384 10500 0 20000;
385 12600 0 20000; 386 14700 0 20000;
MEMBER INCIDENCES
1 1 359; 2 359 369; 3 4 360; 4 360 370; 5 6 361; 6 361 371; 7 8 362; 8 362 372;
9 10 363; 10 363 373; 11 12 364; 12 364 374; 13 14 365; 14 365 375; 15 16 366;
16 366 376;
ELEMENT INCIDENCES SHELL
17 1 337 377 369; 18 1 2 379 369; 19 2 3 380 379; 20 3 4 370 380;
21 3 5 381 380; 22 5 6 371 381; 23 5 7 382 381; 24 7 8 372 382; 25 7 9 383 382;
26 9 10 373 383; 27 9 11 384 383; 28 11 12 374 384; 29 11 13 385 384;
30 13 14 375 385; 31 13 15 386 385; 32 15 16 376 386; 33 16 358 378 376;
34 16 14 375 376; 35 14 12 374 375; 36 12 10 373 374; 37 10 8 372 373;
38 8 6 371 372; 39 6 4 370 371; 40 4 1 369 370; 41 369 379 380 370;
42 370 380 381 371; 43 371 381 382 372; 44 372 382 383 373; 45 373 383 384 374;
46 374 384 385 375; 47 375 385 386 376; 48 1 2 3 4; 49 4 3 5 6; 50 6 5 7 8;
51 8 7 9 10; 52 10 9 11 12; 53 12 11 13 14; 54 14 13 15 16;
MEMBER PROPERTY
1 TO 16 PRIS YD 16 ZD 300
ELEMENT PROPERTY
17 33 TO 40 THICKNESS 225
18 TO 32 41 TO 54 THICKNESS 10
UNIT METER KN
CONSTANTS
E 2.5e+007 MEMB 17 33 TO 40
E 2.05e+008 MEMB 1 TO 16 18 TO 32 41 TO 54
POISSON 0.25 MEMB 1 TO 54
DENSITY 77 MEMB 1 TO 16 18 TO 32 41 TO 54
ALPHA 1.2e-011 MEMB 1 TO 54
DENSITY 24 MEMB 17 33 TO 40
UNIT MMS KN
SUPPORTS
2 15 FIXED BUT FZ MX MY MZ
379 386 PINNED
DEFINE MOVING LOAD
TYPE 1 LOAD 93.2 139.8 139.8 198.1 198.1 198.1 198.1
DIST 3960 1520 2130 1370 3050 1370 WID 2720
LOAD GENERATION 10
TYPE 1 1510 800 0 ZINC 6300
TYPE 1 5410 800 0 ZINC 6300
PERFORM ANALYSIS
PERFORM ANALYSIS PRINT LOAD DATA
FINISH
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4l-7c-40

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STAAD SPACE 4L-7C-20
START JOB INFORMATION
ENGINEER DATE 07-Apr-05
END JOB INFORMATION
INPUT WIDTH 79
UNIT MMS KN
JOINT COORDINATES
1 0 800 0; 2 0 0 0; 3 2100 0 0; 4 2100 800 0; 5 4200 0 0; 6 4200 800 0;
7 6300 0 0; 8 6300 800 0; 9 8400 0 0; 10 8400 800 0; 11 10500 0 0;
12 10500 800 0; 13 12600 0 0; 14 12600 800 0; 15 14700 0 0; 16 14700 800 0;
337 -1050 800 0; 358 15750 800 0; 359 0 800 20000; 360 2100 800 20000;
361 4200 800 20000; 362 6300 800 20000; 363 8400 800 20000;
364 10500 800 20000; 365 12600 800 20000; 366 14700 800 20000; 369 0 800 40000;
370 2100 800 40000; 371 4200 800 40000; 372 6300 800 40000; 373 8400 800 40000;
374 10500 800 40000; 375 12600 800 40000; 376 14700 800 40000;
377 -1050 800 40000; 378 15750 800 40000; 379 0 0 40000; 380 2100 0 40000;
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381 4200 0 40000; 382 6300 0 40000; 383 8400 0 40000; 384 10500 0 40000;
385 12600 0 40000; 386 14700 0 40000;
MEMBER INCIDENCES
1 1 359; 2 359 369; 3 4 360; 4 360 370; 5 6 361; 6 361 371; 7 8 362; 8 362 372;
9 10 363; 10 363 373; 11 12 364; 12 364 374; 13 14 365; 14 365 375; 15 16 366;
16 366 376;
ELEMENT INCIDENCES SHELL
17 1 337 377 369; 18 1 2 379 369; 19 2 3 380 379; 20 3 4 370 380;
21 3 5 381 380; 22 5 6 371 381; 23 5 7 382 381; 24 7 8 372 382; 25 7 9 383 382;
26 9 10 373 383; 27 9 11 384 383; 28 11 12 374 384; 29 11 13 385 384;
30 13 14 375 385; 31 13 15 386 385; 32 15 16 376 386; 33 16 358 378 376;
34 16 14 375 376; 35 14 12 374 375; 36 12 10 373 374; 37 10 8 372 373;
38 8 6 371 372; 39 6 4 370 371; 40 4 1 369 370; 41 369 379 380 370;
42 370 380 381 371; 43 371 381 382 372; 44 372 382 383 373; 45 373 383 384 374;
46 374 384 385 375; 47 375 385 386 376; 48 1 2 3 4; 49 4 3 5 6; 50 6 5 7 8;
51 8 7 9 10; 52 10 9 11 12; 53 12 11 13 14; 54 14 13 15 16;
MEMBER PROPERTY
1 TO 16 PRIS YD 28 ZD 375
ELEMENT PROPERTY
17 33 TO 40 THICKNESS 225
18 20 22 24 26 28 30 32 41 TO 54 THICKNESS 14
19 21 23 25 27 29 31 THICKNESS 12
UNIT METER KN
CONSTANTS
E 2.5e+007 MEMB 17 33 TO 40
E 2.05e+008 MEMB 1 TO 16 18 TO 32 41 TO 54
POISSON 0.25 MEMB 1 TO 54
DENSITY 77 MEMB 1 TO 16 18 TO 32 41 TO 54
ALPHA 1.2e-011 MEMB 1 TO 54
DENSITY 24 MEMB 17 33 TO 40
UNIT MMS KN
SUPPORTS
2 15 FIXED BUT FZ MX MY MZ
379 386 PINNED
DEFINE MOVING LOAD
TYPE 1 LOAD 87.2 130.8 130.8 185.3 185.3 185.3 185.3 218 218
DIST 3960 1520 2130 1370 3050 1370 2740 1220 WID 2720
LOAD GENERATION 15
TYPE 1 1510 800 0 ZINC 6300
TYPE 1 5430 800 0 ZINC 6300
PERFORM ANALYSIS
PERFORM ANALYSIS PRINT LOAD DATA
FINISH
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4l-7c-60

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STAAD SPACE 4L-7C-20
START JOB INFORMATION
ENGINEER DATE 07-Apr-05
END JOB INFORMATION
INPUT WIDTH 79
UNIT MMS KN
JOINT COORDINATES
1 0 800 0; 2 0 0 0; 3 2100 0 0; 4 2100 800 0; 5 4200 0 0; 6 4200 800 0;
7 6300 0 0; 8 6300 800 0; 9 8400 0 0; 10 8400 800 0; 11 10500 0 0;
12 10500 800 0; 13 12600 0 0; 14 12600 800 0; 15 14700 0 0; 16 14700 800 0;
337 -1050 800 0; 358 15750 800 0; 359 0 800 30000; 360 2100 800 30000;
361 4200 800 30000; 362 6300 800 30000; 363 8400 800 30000;
364 10500 800 30000; 365 12600 800 30000; 366 14700 800 30000; 369 0 800 60000;
370 2100 800 60000; 371 4200 800 60000; 372 6300 800 60000; 373 8400 800 60000;
374 10500 800 60000; 375 12600 800 60000; 376 14700 800 60000;
377 -1050 800 60000; 378 15750 800 60000; 379 0 0 60000; 380 2100 0 60000;
381 4200 0 60000; 382 6300 0 60000; 383 8400 0 60000; 384 10500 0 60000;
385 12600 0 60000; 386 14700 0 60000;
MEMBER INCIDENCES
1 1 359; 2 359 369; 3 4 360; 4 360 370; 5 6 361; 6 361 371; 7 8 362; 8 362 372;
9 10 363; 10 363 373; 11 12 364; 12 364 374; 13 14 365; 14 365 375; 15 16 366;
16 366 376;
ELEMENT INCIDENCES SHELL
17 1 337 377 369; 18 1 2 379 369; 19 2 3 380 379; 20 3 4 370 380;
21 3 5 381 380; 22 5 6 371 381; 23 5 7 382 381; 24 7 8 372 382; 25 7 9 383 382;
26 9 10 373 383; 27 9 11 384 383; 28 11 12 374 384; 29 11 13 385 384;
30 13 14 375 385; 31 13 15 386 385; 32 15 16 376 386; 33 16 358 378 376;
34 16 14 375 376; 35 14 12 374 375; 36 12 10 373 374; 37 10 8 372 373;
38 8 6 371 372; 39 6 4 370 371; 40 4 1 369 370; 41 369 379 380 370;
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42 370 380 381 371; 43 371 381 382 372; 44 372 382 383 373; 45 373 383 384 374;
46 374 384 385 375; 47 375 385 386 376; 48 1 2 3 4; 49 4 3 5 6; 50 6 5 7 8;
51 8 7 9 10; 52 10 9 11 12; 53 12 11 13 14; 54 14 13 15 16;
MEMBER PROPERTY
1 TO 16 PRIS YD 40 ZD 450
ELEMENT PROPERTY
17 33 TO 40 THICKNESS 225
18 20 22 24 26 28 30 32 41 TO 54 THICKNESS 18
19 21 23 25 27 29 31 THICKNESS 15
UNIT METER KN
CONSTANTS
E 2.5e+007 MEMB 17 33 TO 40
E 2.05e+008 MEMB 1 TO 16 18 TO 32 41 TO 54
POISSON 0.25 MEMB 1 TO 54
DENSITY 77 MEMB 1 TO 16 18 TO 32 41 TO 54
ALPHA 1.2e-011 MEMB 1 TO 54
DENSITY 24 MEMB 17 33 TO 40
UNIT MMS KN
SUPPORTS
2 15 FIXED BUT FZ MX MY MZ
379 386 PINNED
DEFINE MOVING LOAD
TYPE 1 LOAD 87.2 130.8 130.8 185.3 185.3 185.3 185.3 218 218
DIST 3960 1520 2130 1370 3050 1370 2740 1220 WID 2720
LOAD GENERATION 15
TYPE 1 1510 800 0 ZINC 6300
TYPE 1 5430 800 0 ZINC 6300
PERFORM ANALYSIS
PERFORM ANALYSIS PRINT LOAD DATA
FINISH

4l-7c-80

STAAD SPACE 4L-7C-20
START JOB INFORMATION
ENGINEER DATE 07-Apr-05
END JOB INFORMATION
INPUT WIDTH 79
UNIT MMS KN
JOINT COORDINATES
1 0 800 0; 2 0 0 0; 3 2100 0 0; 4 2100 800 0; 5 4200 0 0; 6 4200 800 0;
7 6300 0 0; 8 6300 800 0; 9 8400 0 0; 10 8400 800 0; 11 10500 0 0;
12 10500 800 0; 13 12600 0 0; 14 12600 800 0; 15 14700 0 0; 16 14700 800 0;
337 -1050 800 0; 358 15750 800 0; 359 0 800 40000; 360 2100 800 40000;
361 4200 800 40000; 362 6300 800 40000; 363 8400 800 40000;
364 10500 800 40000; 365 12600 800 40000; 366 14700 800 40000; 369 0 800 80000;
370 2100 800 80000; 371 4200 800 80000; 372 6300 800 80000; 373 8400 800 80000;
374 10500 800 80000; 375 12600 800 80000; 376 14700 800 80000;
377 -1050 800 80000; 378 15750 800 80000; 379 0 0 80000; 380 2100 0 80000;
381 4200 0 80000; 382 6300 0 80000; 383 8400 0 80000; 384 10500 0 80000;
385 12600 0 80000; 386 14700 0 80000;
MEMBER INCIDENCES
1 1 359; 2 359 369; 3 4 360; 4 360 370; 5 6 361; 6 361 371; 7 8 362; 8 362 372;
9 10 363; 10 363 373; 11 12 364; 12 364 374; 13 14 365; 14 365 375; 15 16 366;
16 366 376;
ELEMENT INCIDENCES SHELL
17 1 337 377 369; 18 1 2 379 369; 19 2 3 380 379; 20 3 4 370 380;
21 3 5 381 380; 22 5 6 371 381; 23 5 7 382 381; 24 7 8 372 382; 25 7 9 383 382;
26 9 10 373 383; 27 9 11 384 383; 28 11 12 374 384; 29 11 13 385 384;
30 13 14 375 385; 31 13 15 386 385; 32 15 16 376 386; 33 16 358 378 376;
34 16 14 375 376; 35 14 12 374 375; 36 12 10 373 374; 37 10 8 372 373;
38 8 6 371 372; 39 6 4 370 371; 40 4 1 369 370; 41 369 379 380 370;
42 370 380 381 371; 43 371 381 382 372; 44 372 382 383 373; 45 373 383 384 374;
46 374 384 385 375; 47 375 385 386 376; 48 1 2 3 4; 49 4 3 5 6; 50 6 5 7 8;
51 8 7 9 10; 52 10 9 11 12; 53 12 11 13 14; 54 14 13 15 16;
MEMBER PROPERTY
1 TO 16 PRIS YD 52 ZD 530
ELEMENT PROPERTY
17 33 TO 40 THICKNESS 225
18 20 22 24 26 28 30 32 41 TO 54 THICKNESS 22
19 21 23 25 27 29 31 THICKNESS 17
UNIT METER KN
CONSTANTS
E 2.5e+007 MEMB 17 33 TO 40

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E 2.05e+008 MEMB 1 TO 16 18 TO 32 41 TO 54
POISSON 0.25 MEMB 1 TO 54
DENSITY 77 MEMB 1 TO 16 18 TO 32 41 TO 54
ALPHA 1.2e-011 MEMB 1 TO 54
DENSITY 24 MEMB 17 33 TO 40
UNIT MMS KN
SUPPORTS
2 15 FIXED BUT FZ MX MY MZ
379 386 PINNED
DEFINE MOVING LOAD
TYPE 1 LOAD 87.2 130.8 130.8 185.3 185.3 185.3 185.3 218 218
DIST 3960 1520 2130 1370 3050 1370 2740 1220 WID 2720
LOAD GENERATION 15
TYPE 1 1510 800 0 ZINC 6300
TYPE 1 5430 800 0 ZINC 6300
PERFORM ANALYSIS
PERFORM ANALYSIS PRINT LOAD DATA
FINISH
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4l-7c-100

```
STAAD SPACE 4L-7C-20
START JOB INFORMATION
ENGINEER DATE 07-Apr-05
END JOB INFORMATION
INPUT WIDTH 79
UNIT MMS KN
JOINT COORDINATES
1 0 800 0; 2 0 0 0; 3 2100 0 0; 4 2100 800 0; 5 4200 0 0; 6 4200 800 0;
7 6300 0 0; 8 6300 800 0; 9 8400 0 0; 10 8400 800 0; 11 10500 0 0;
12 10500 800 0; 13 12600 0 0; 14 12600 800 0; 15 14700 0 0; 16 14700 800 0;
337 -1050 800 0; 358 15750 800 0; 359 0 800 25000; 360 2100 800 25000;
361 4200 800 25000; 362 6300 800 25000; 363 8400 800 25000;
364 10500 800 25000; 365 12600 800 25000; 366 14700 800 25000; 369 0 800 50000;
370 2100 800 50000; 371 4200 800 50000; 372 6300 800 50000; 373 8400 800 50000;
374 10500 800 50000; 375 12600 800 50000; 376 14700 800 50000; 379 0 800 75000;
380 2100 800 75000; 381 4200 800 75000; 382 6300 800 75000; 383 8400 800 75000;
384 10500 800 75000; 385 12600 800 75000; 386 14700 800 75000;
389 0 800 100000; 390 2100 800 100000; 391 4200 800 100000;
392 6300 800 100000; 393 8400 800 100000; 394 10500 800 100000;
395 12600 800 100000; 396 14700 800 100000; 397 -1050 800 100000;
398 15750 800 100000; 399 0 0 100000; 400 2100 0 100000; 401 4200 0 100000;
402 6300 0 100000; 403 8400 0 100000; 404 10500 0 100000; 405 12600 0 100000;
406 14700 0 100000;
MEMBER INCIDENCES
1 1 359; 2 359 369; 3 369 379; 4 379 389; 5 4 360; 6 360 370; 7 370 380;
8 380 390; 9 6 361; 10 361 371; 11 371 381; 12 381 391; 13 8 362; 14 362 372;
15 372 382; 16 382 392; 17 10 363; 18 363 373; 19 373 383; 20 383 393;
21 12 364; 22 364 374; 23 374 384; 24 384 394; 25 14 365; 26 365 375;
27 375 385; 28 385 395; 29 16 366; 30 366 376; 31 376 386; 32 386 396;
ELEMENT INCIDENCES SHELL
33 1 337 397 389; 34 1 2 399 389; 35 2 3 400 399; 36 3 4 390 400;
37 3 5 401 400; 38 5 6 391 401; 39 5 7 402 401; 40 7 8 392 402; 41 7 9 403 402;
42 9 10 393 403; 43 9 11 404 403; 44 11 12 394 404; 45 11 13 405 404;
46 13 14 395 405; 47 13 15 406 405; 48 15 16 396 406; 49 16 358 398 396;
50 16 14 395 396; 51 14 12 394 395; 52 12 10 393 394; 53 10 8 392 393;
54 8 6 391 392; 55 6 4 390 391; 56 4 1 389 390; 57 389 399 400 390;
58 390 400 401 391; 59 391 401 402 392; 60 392 402 403 393; 61 393 403 404 394;
62 394 404 405 395; 63 395 405 406 396; 64 1 2 3 4; 65 4 3 5 6; 66 6 5 7 8;
67 8 7 9 10; 68 10 9 11 12; 69 12 11 13 14; 70 14 13 15 16;
MEMBER PROPERTY
1 TO 32 PRIS YD 64 ZD 600
ELEMENT PROPERTY
33 49 TO 56 THICKNESS 225
34 36 38 40 42 44 46 48 57 TO 70 THICKNESS 26
35 37 39 41 43 45 47 THICKNESS 20
UNIT METER KN
CONSTANTS
E 2.5e+007 MEMB 33 49 TO 56
E 2.05e+008 MEMB 1 TO 32 34 TO 48 57 TO 70
POISSON 0.25 MEMB 1 TO 70
DENSITY 77 MEMB 1 TO 32 34 TO 48 57 TO 70
ALPHA 1.2e-011 MEMB 1 TO 70
DENSITY 24 MEMB 33 49 TO 56
UNIT MMS KN
```

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SUPPORTS
399 406 PINNED
2 15 FIXED BUT FZ MX MY MZ
DEFINE MOVING LOAD
TYPE 1 LOAD 87.2 130.8 130.8 185.3 185.3 185.3 185.3 218 218
DIST 3960 1520 2130 1370 3050 1370 2740 1220 WID 2720
LOAD GENERATION 15
TYPE 1 1510 800 0 ZINC 6300
TYPE 1 5430 800 0 ZINC 6300
PERFORM ANALYSIS
PERFORM ANALYSIS PRINT LOAD DATA
FINISH
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STAAD SPACE IDEAL
START JOB INFORMATION
ENGINEER DATE 13-May-05
END JOB INFORMATION
INPUT WIDTH 79
UNIT MMS KN
JOINT COORDINATES
1 0 0 0; 2 0 800 0; 3 1135 0 0; 4 0 0 60000; 5 0 800 60000; 6 1135 0 60000;
7 1135 800 0; 8 1135 800 60000; 9 -1130 800 0; 10 -1130 800 60000;
11 -1135 0 0; 12 -1135 0 60000; 13 0 800 30000;
MEMBER INCIDENCES
1 2 13; 7 13 5;
ELEMENT INCIDENCES SHELL
2 9 2 5 10; 3 2 7 8 5; 4 2 1 4 5; 5 11 1 4 12; 6 1 3 6 4;
MEMBER PROPERTY
1 7 PRIS YD 40 ZD 450
ELEMENT PROPERTY
2 3 THICKNESS 225
4 THICKNESS 18
5 6 THICKNESS 15
SUPPORTS
2 5 PINNED
UNIT METER KN
CONSTANTS
E 2.5e+007 MEMB 2 3
E 2.05e+008 MEMB 1 4 TO 7
POISSON 0.25 MEMB 1 TO 7
DENSITY 77 MEMB 1 4 TO 7
ALPHA 1.2e-011 MEMB 1 TO 7
DENSITY 24 MEMB 2 3
UNIT MMS KN
DEFINE MOVING LOAD
TYPE 1 LOAD 87.2 130.8 130.8 185.3 185.3 185.3 185.3 218 218
DIST 3960 1520 2130 1370 3050 1370 2740 1220 WID 2720
LOAD GENERATION 15
TYPE 1 0 800 0 ZINC 4540
PERFORM ANALYSIS
PERFORM ANALYSIS PRINT LOAD DATA
FINISH
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STAAD SPACE IDEAL
START JOB INFORMATION
ENGINEER DATE 13-May-05
END JOB INFORMATION
INPUT WIDTH 79
UNIT MMS KN
JOINT COORDINATES
1 0 0 0; 2 0 800 0; 3 1135 0 0; 4 0 0 60000; 5 0 800 60000; 6 1135 0 60000;
7 1135 800 0; 8 1135 800 60000; 9 -1130 800 0; 10 -1130 800 60000;
11 0 800 30000;
MEMBER INCIDENCES
4 2 11; 9 11 5;
ELEMENT INCIDENCES SHELL
5 2 1 4 5; 6 3 1 4 6; 7 9 2 5 10; 8 2 7 8 5;
MEMBER PROPERTY
4 9 PRIS YD 40 ZD 450
ELEMENT PROPERTY
7 8 THICKNESS 225
5 THICKNESS 18
```

```
6 THICKNESS 15
SUPPORTS
2 5 PINNED
UNIT METER KN
CONSTANTS
E 2.5e+007 MEMB 7 8
E 2.05e+008 MEMB 4 TO 6 9
POISSON 0.25 MEMB 4 TO 9
DENSITY 77 MEMB 4 TO 6 9
ALPHA 1.2e-011 MEMB 4 TO 9
DENSITY 24 MEMB 7 8
UNIT MMS KN
DEFINE MOVING LOAD
TYPE 1 LOAD 87.2 130.8 130.8 185.3 185.3 185.3 185.3 218 218
DIST 3960 1520 2130 1370 3050 1370 2740 1220 WID 2720
LOAD GENERATION 15
TYPE 1 0 800 0 ZINC 4540
PERFORM ANALYSIS
PERFORM ANALYSIS PRINT LOAD DATA
PRINT MEMBER FORCES LIST 4
FINISH
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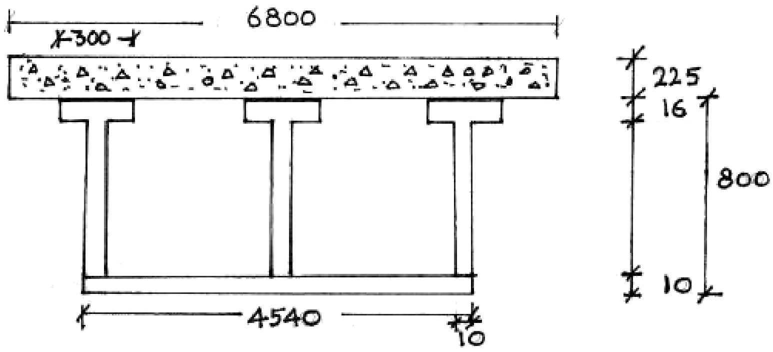
MANUAL ANAYSIS

In this manual analysis the case of 1L-2c-20 is solved manually by transform area method of elastic analysis to get the moment of resistance of the section (Fig 1).



(All dimensions are in mm)

Fig.1. Cross-section of 2c-20 composite multi-cell box girder bridge

Data's known;

Young's Modulus of Elasticity of Steel;E_s= 205 GPa

Young's Modulus of Elasticity of Concrete; E_c= 25 GPa

Yield Stress of Steel; f_y = 300N/mm^2

Cube strength of Concrete f_{cu}= 25 N/mm^2

So, modular ratio   m= E_s/ E_c

m = 205/25 = 8.2

Dividing the width 'B' of concrete deck slab by 'm' will transform concrete area to steel area. (Fig.2)

New B' = B/m = 6800/82 = 829.26mm

Now we distribute the transformed area equally on the top of three flanges.

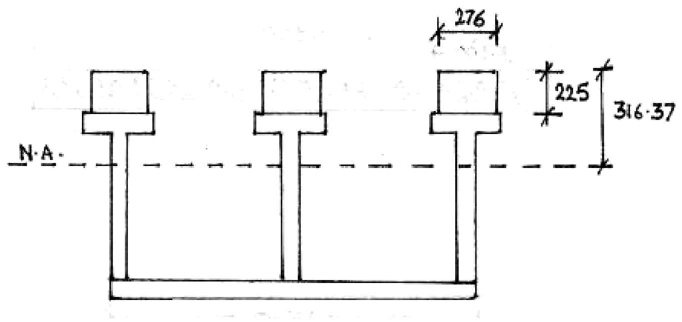
So, b = 829.26/3= 276mm

Now, finding the combined neutral axis of composite section

$$\bar{x}_{comp} = \frac{3\times276\times225\times112.5+3\times300\times16\times233+3\times774\times10\times628+4540\times10\times1020}{3\times276\times225+3\times300\times16+3\times774\times10+4540\times10}$$

$$\bar{x}_{comp} = \frac{20958750+3355200+14582160+46308000}{186300+14400+23220+45400} = \frac{85204110}{269320} = 316.37\text{mm}$$

$\bar{x}_{steel} = 773.85$



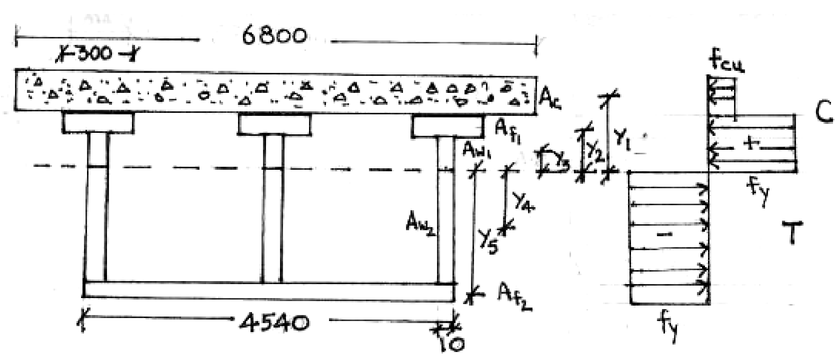
(All dimensions are in mm)

Fig.2. Transformed Cross-section of 2c-20 composite multi-cell box girder bridge

So, neutral axis is below the concrete slab and passes through webs.

Now assuming the stress block to be rectangular to make the calculation simple

Moment of resistance of the section (M.R);



(All dimensions are in mm)

Fig.3.Composite multi-cell box girder bridge along with stress diagram

$$M.R. = f_{cu}A_c y_1 + f_y[A_{f1}y_2 + A_{w1}y_3 + A_{w2}y_4 + A_{f2}y_5]$$

y_1 =203.87mm

y_2 =83.37mm

y_3 =37.69mm

y_4 =349.315mm

y_5 =703.63mm

$$M.R. = 25 \times 6800 \times 225 \times 203.87 + 300[14400 \times 83.37 + 75.37 \times 10 \times 37.69$$
$$+ 698.63 \times 10 \times 349.315 + 10 \times 4540 \times 703.63$$

$$M.R. = 7.79 \times 10^9 + 300[35614156.34] = 18490 \text{ KNm}$$

So, the results obtain from STAAD.Pro are with in the limit of the section.