ANALYSIS AND DESIGN OF PRESTRESSED POST-TENSIONED I TYPE GIRDER BRIDGE

Dissertation

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CERTIFICATE

This is to certify that the Major Project entitled **"Analysis and Design of Prestressed Post-Tensioned I type girder Bridge"** submitted by **Mr. Dhaivat A. Vasavada (03MCL18)**, towards the partial fulfillment of the requirements for the award of degree of **Master of Technology (CIVIL)** in field of **Computer Aided Structural Analysis and Design (CASAD)** of Nirma University of Science and Technology is the record of work carried out by him under my supervision and guidance. The work submitted has in my opinion reached a level required for being accepted for examination. The results embodied in this dissertation, to the best of my knowledge have not been submitted to any other university or institution for award of any degree or diploma.

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Abstract

Prestress can provide a solution to many structural problems. Prestress is a technique to create in advance accurately known permanent forces required. Prestressing in a member replaces structural steel in the member, which needs costly maintenance and corrodes under aggressive atmospheric conditions. Because of these short comings of structural steel, Pre-stressed concrete is ideal for the construction of medium and long span bridges. Pre-stressed concrete bridges offer a high degree of freedom from cracks, low maintenance and long life. The use of high strength concrete and high tensile steel results in slender sections, which are aesthetically superior, coupled with overall economy. In comparison with steel bridges, pre-stressed concrete bridges require very little maintenance and not easily damaged by fire.

In a conventional approach to the design of prestressed concrete girders, a section of the girder and cable profile is required to be assumed. Thereafter, going through a series of typical and rigorous calculations, stress check is made and it is established whether the section and cable profile are suitable or not. If the design is not suitable, a revised section of girder and/or cable profile is tried. Since the process is repetitive, it can be better achieved by a computer software rather than repetitive hand calculation.

In this program different cross-sectional dimensions of the girder and geometry and type of the of various types of strands, i.e. 7T13, 12T13, 19T13 etc. or any combination of these are to be entered for the purpose of preliminary design. From the available input data the program calculate sectional properties, Courban’s Factors, dead load analysis, live load analysis, prestressing forces, prestress losses and check for stresses at various sections of the prestress simply supported I- girder. The time dependent losses are considered as per IRC-18. At any specified section, checking of final stresses is possible. If the initial data fail to comply with

the required condition as per IRC, the program does not move further for the check for ultimate shear, moment resistance of the section, design of diaphragm, end block and deck slab.

Keywords: Prestress, I type girder, bridge, strand

Lay out of Chapters

Chapter 1

Covers the definition of prestress, methods of prestressing and application of prestressed concrete in the field of structural engineering. Various type of bridge superstructure are discussed at length.

Chapter 2

The literature collected for analysis and design of Prestressed Post-tensioned I type girder bridge.

Chapter 3

Theoretical background of the study is given in this chapter. The design procedure i.e. preliminary dimensioning, properties of section, distribution of loads, finding of bending moment and shear, losses in prestress, checking of stresses and check for ultimate strength are explained step by step.

Chapter 4

In this chapter longitudinal analysis of I girder is explained. It contains detailed sectional property calculation, load calculation, cable layout and calculation for prestress losses and finally checks for flexural stresses and shear checks.

Chapter 5

In this chapter design of deck slab, diaphragm and anchor block is done.

Chapter 6

This chapter describes with the help of flow chart structure of computer program made for simplified analysis and design of prestress I type girder bridge.

Chapter 7

Conclusions of the study and future scope are explained**.**

INDEX

INDEX OF FIGURES

INDEX OF TABLES

CHAPTER:-1
INTRODUCTION

1.1 GENERAL

There is probably no structural problem to which prestress cannot provide a solution, and often a revolutionary one. Prestress is more than a technique, it is a general principle, to create in advance accurately known permanent forces acting as required. If we restrict our areas to only Structural Engineering, prestressed concrete could provide practical answer to many problems. Prestressed concrete girder is of special interest for long span bridges, large auditoriums and cinema hall.

One of the best definitions of prestressed concrete is given by the ACI Committee on prestressed concrete: "Prestressed concrete: Concrete in which there have been introduced internal stresses of such magnitude and distribution that the stresses resulting from given external loadings are countered to a desired degree. In reinforced-concrete members the prestress is commonly introduced by tensioning the steel reinforcement."

1.2 METHODS OF PRESTRESSING

Several methods and techniques of prestressing are available and most of them can be classified within two major groups like pretensioning and post tensioning.

- Pretensioning is an act of stretching the prestressing tendons (wire, strands) to a predetermined tension and anchoring them to fixed bulkheads or moulds. The concrete is cast around the tendons and cured and after hardening, the tendons are released. The prefix "pre" in Pretensioning refers to the fact that the tendons are tensioned prior to hardening of concrete. Pretensioning is mostly used for the products of precast prestressed concrete elements. In this way the prestress is transferred to the concrete by bond.

- In post tensioning the tendons are stressed and anchored at the ends of concrete members after casting the member and after it has attained sufficient strength. In bonded post-tensioned member, after stressing and anchoring, the void between each tendon and the duct is filled with a mortar grout, which subsequently hardens. Grouting makes bond of the tendon to the surrounding concrete which ensures improvement in the resistance of member to cracking and reduction of the risks of corrosion of steel tendons. In unbonded post-tensioned member the duct is filed with grease instead of grout and thus the bond will be prevented throughout the length of the tendon. Here the tendon force will be applied to the concrete only at the anchorages. Unbonded tendons are generally coated with grease or bituminous material, wrapped with water-proof paper and placed in the forms prior to concrete castings. In this way the prestress is transferred to the concrete by bearing.

1.3 MATERIALS FOR PRESTRESSED CONCRETE

1.3.1 Need of high strength concrete

High-strength concrete is necessary in prestressed concrete, as the material offers high resistance in tension, shear, bond and bearing. In the zone of anchorages, the bearing stresses being higher, high-strength concrete is always preferred to minimize costs. High strength concrete has a high compressive strength at a reasonably early age, with comparatively higher tensile strength than ordinary concrete, less liable to shrinkage cracks, and has a higher modulus of elasticity and smaller ultimate creep strain, resulting in a smaller loss of prestress in steel. Also have many desirable properties, such as durability, and impermeability and abrasion resistance. The use of high-strength concrete results in a reduction in the cross section dimensions of prestressed concrete structural elements. With a reduced dead-weight of the material, longer spans become technically and economically practicable.

1.3.2 Need of high strength steel

The early attempts to use mild steel in prestressed concrete were not successful, as a working stress of 120 N/mm^2 in mild steel is more or less completely lost due to elastic deformation, creep and shrinkage of concrete.

The normal loss of stress in steel is generally about 100 to 240 N/mm^2 and it is clear that if this loss of stress is to be a small portion of the initial stress, the stress in steel in the initial stages must be very high, about 1200 to 2000 N/mm^2. These high stress ranges are possible only with the use of high-strength steel.

For that reason in prestressed concrete members, the high-tensile steel used generally consists of wires, bars, or strands. The higher tensile strength is generally achieved by increasing the carbon content in steel in comparison with mild steel.

1.4 PRESTRESSED VERSUS REINFORCED CONCRETE

The most outstanding difference between the two is taking advantage of materials of higher strength for pre-stressed concrete. In order to utilize the full strength of the high-tensile steel, it is necessary to resort to pre-stressing to pre-stretch it. Pre-stressing the steel and anchoring eliminates cracks in concrete. Thus the entire section of the concrete becomes effective in pre-stressed concrete, whereas only the portion of section above the neutral axis is supposed to act in the case of reinforced concrete.

The use of curved tendons will have to carry some of the shear in a member. In addition, pre-compression in the concrete tends to reduce the diagonal tension. Thus it is possible to use smaller section in pre-stressed concrete to carry the same amount of external shear in a beam.

High strength concrete cannot be economically utilized in reinforced concrete construction and it is found necessary with pre-stressed concrete. In reinforced concrete, using concrete of high strength will result in a smaller section calling for more reinforcement and will end with more costly design. In pre-stressed concrete, high strength concrete is required to match with high strength steel in order to yield economical proportions. Stronger concrete is also necessary to resist high stresses at the anchorages and to give strength to the thinner section so frequently employed for pre-stressed concrete.

The advantages and disadvantages of pre-stressed concrete as compared to reinforced concrete are discussed in respect to their serviceability, safety and economics.

1.4.1 Serviceability

Pre-stressed concrete structures do not crack under working loads, and whatever cracks may be developed under over loads will be closed up as soon as the load is removed, unless the load is excessive. Under dead load, deflection is compensating by the cambering effect of pre-stress. Under live load, deflection is also smaller because of the effectiveness of the entire uncracked concrete section, which has a moment of inertia 2 to 3 times that of the cracked section. So far as serviceability is concerned, the only shortcoming of pre-stressed concrete is its lack of weight.

1.4.2 Safety

When properly designed by the present conventional methods, Pre-stressed concrete structures have overload capacities similar to and perhaps slightly higher than those of reinforced concrete. For the usual design, they deflect appreciably before ultimate failure, thus giving ample warning before impending collapse. The ability to resist shocks and impact loads have been shown to be as good in pre-stressed as in reinforced concrete. The resistance to corrosion is better than that of reinforced concrete for the same amount of cover, owing to denser concrete and the non existence of cracks. If cracks occur, corrosion can be more serious in pre-stressed concrete.

1.4.3 Economics

From an economic point of view, it is evident that smaller quantities of materials, both steel and concrete, are required to carry the same load, since the materials are of higher strength. There is also a definite saving in stirrups, since shear in pre-stressed concrete is reduced by the inclination of the tendons, and the diagonal tension is further minimized by the presence of the pre-stress. The reduced weight of the member will help in economizing the sections, the smaller dead load and depth of member will result in saving materials from other portions of the structure. In pre-cast members, a reduction of weight saves handling and transportation cost.

In spite of the above economy possible with pre-stressed concrete, its use cannot be advocated for all conditions. First of all, the stronger materials will have a higher unit cost. More auxiliary materials are required for pre-stressing, such as end anchorage and grouts. More formwork is also needed, since non-rectangular shapes are often necessary for pre-stressed concrete. More labourers are required to place the steel in pre-stressed concrete, especially when the amount of work is small. More attention to design is involved, and more supervision is necessary, the amount of additional work will depend upon the experience of the engineer and the construction crews, but it will not be serious if the same typical design is repeated many times.

1.5 GENERAL FEATURES OF PRESTRESSED CONCRETE BRIDGES

Pre-stressed concrete is ideal for the construction of medium and long span bridges. Ever since the development of the pre-stressed concrete, the material has found extensive application in the construction in the long span bridges gradually replacing steel, which needs costly maintenance due to the disadvantages of corrosion under aggressive atmospheric conditions. Pre-stressed concrete bridges offer a high degree of freedom from cracks, low maintenance, long life and they are particularly suitable for dynamic loads and vibrations.

Pre-stressed concrete is ideally suited for long span continuous bridges. Pre-stressed concrete has been widely used throughout the world for simply supported, continuous, balanced cantilever, suspension bridges in the span range from 20 to 500m.

1.6 ADVANTAGES OF PRE-STRESSED CONCRETE BRIDGES

Pre-stressed concrete made up of high strength concrete and high tensile steel has distinct advantages when used for bridge construction. The salient benefits resulting from the use of pre-stressed concrete in bridges are outlined as follows:-

- The use of high strength concrete and high tensile steel results in slender sections, which are aesthetically superior, coupled with overall economy.

- Pre-stressed concrete bridges can be designed as class 1-type structures without any tensile stresses under service loads resulting in a crack free structure.

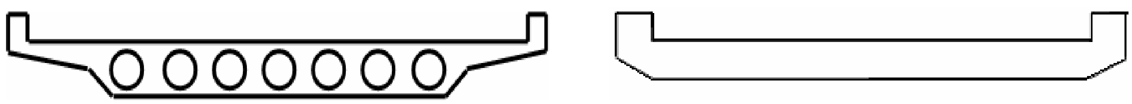
- In comparison with steel bridges, pre-stressed concrete bridges require very little maintenance and not easily damaged by fire.

- Pre-stressed concrete is ideally suited for composite bridge construction in which pre-cast pre-stressed girders support the cast-in-situ slab deck. This type of construction is very popular since it involves minimum disruption of traffic.

➢ Post-tensioned pre-stressed concrete finds extensive applications in long span continuous girder bridges of variable cross-section resulting in sleek structures and with considerable savings in the overall cost of the construction.

1.7 TYPES OF BRIDGE SUPERSTRUCTURES

Typical arrangements of bridge decks with post-tensioned girders suitable for simply supported constructions are shown in Fig 1.1 to 1.3. Basically the arrangements may be one of the following types:-



(a) (b)

Fig 1.1 Slab type superstructure (Solid and voided)

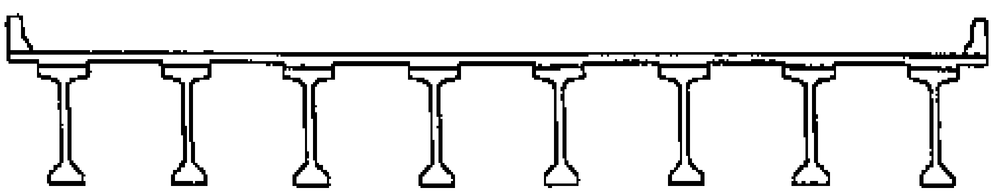


Fig 1.2 Beam and slab type superstructure

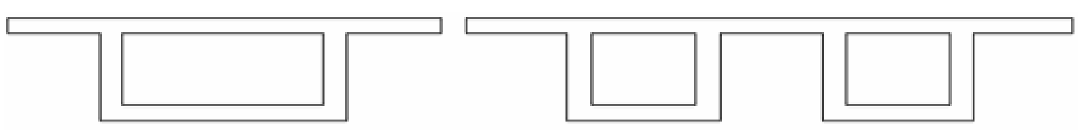


Fig 1.3 Box type superstructure

1.7.1 Slab type superstructure (Solid or voided)

Cast in situ voided prestressed concrete slabs as shown in Fig 1.1 may be adopted for spans of 15 to 20 m, of depth up to about 1.2 m, are economical. Solid composite slab decks shown in Fig 1.1(b), consist of precast units of various shapes, The units are either of inverted T or symmetrical I section, placed side by side, and stressed together transversely after in situ filing.

1.7.2 Beam and slab type superstructure

This type of cross-section of a bridge deck often governs the weight, maximum span, and cost of the bridge. Multi-beam pre-stressed concrete cross-section of the types shown in Fig 1.2 is economical for spans up to approximately 20 to 40 m, in the very limit.

1.7.3 Box type superstructure

Box type superstructure Fig 1.3 is uneconomical for simply-supported spans unless the span has necessarily to be a large and/or the construction depth is very limited. Its main advantage is that it facilitates placement of cable with maximum eccentricity, offers resolute section property for sagging as well as hogging moment, and is rigid for efficient transverse load distribution and torsion. This type of superstructure may be adopted for spans of 30 to 70 m.

1.8 SCOPE OF WORK

An attempt is made here to design a PRESTRESSED POST-TENSIONED BRIDGE SUPER STRUCTURE of I-girder type section, having 40m span for 3-lane Bridge. The girder is designed for Class AA, Class A and 70R Loadings. The deck slab analysis covers the longitudinal analysis for all above loading. The transverse analysis is done using Courbon’s method.

Reference has been made of relevant IRC codes of practice.

The design is intended for simply supported ends for the following limit states:

- Limit state of serviceability for maximum compression
- Limit state of collapse : flexure
- Limit state of collapse : shear
- Limit state of serviceability : deflection
- Limit state of serviceability : cracking

Computer program is prepared for sectional property calculations, load calculations, losses, extreme fiber stress check, shear check and design of various components like girder, deck slab, End block and diaphragm.

CHAPTER:-2
LITERATURE REVIEW

2.1 GENERAL

Survey from various research papers has been carried out to support the present work. Literature survey has been carried out for I girder bridge configuration and related girder design aspects, effects of diaphragm in slab on girder bridge and need of computer program in prestressed concrete girder design.

Z.Lounis, M.S.Mirza, M.Z.Cohn(1996). ”Segmental and Conventional Precast Prestressed Concrete I-Bridge Girders”, Journal of Bridge Engineering, ASCE, Vol.2 No.3, pp. 73-82

In this paper a new set of optimum I-sections are generated using an efficient nonlinear programming algorithm. The main results of the investigation are summarized as below:

1. The optimum girder shape is found to be a bulb-tee section for posttensioned girders and a nearly symmetrical I-section for conventional pretensioned girders.

2. The span and girder spacing capabilities presented in this paper may be used as design aids for developing preliminary bridge design alternatives with either post-tensioned or precast I-girders.

3. The selection of the optimal structural system and span length will be based on the total bridge cost including the costs of superstructure and substructure, in addition to the other requirements such as limited superstructure depth, elimination of intermediate piers and aesthetics.

Sami W.Tabsh, Kalpana Sahajwani(1997). ”Approximate Analysis of Irregular Slab-on-Girder Bridges”, Journal of Bridge Engineering, ASCE, Vol.2 No.1, pp. 11-17

Thirty-one different composite steel I-beam bridges having irregular beam spacing are analyzed in this study under the effect of live load. The bridges are analyzed by two methods, an approximate procedure and the finite-element method. Wheel load Girder Distribution

Factors (GDFs) are computed for both flexure and shear in interior girders. The approximate method is based on isolating strips of the deck slab in the transverse direction directly under the wheel loads and treating them as beams on elastic supports. The finite-element analysis is used to verify the accuracy of the approximate method. The results of this study indicate that GDFs obtained by the approximate method for flexure and shear in simply supported I-beams are very similar with results obtained using a detailed three-dimensional finite-element analysis. On average, the approximate method yielded GDF values for simple bridges 5.7% and 3.1% less than the finite-element results for flexure and shear, respectively. Based on this paper, it can be concluded that the approximate method of analysis discussed in this study can be safely used to analyze irregular I-beam bridges with unequal girder spacing if a more detailed finite-element analysis is not available.

Tanya Green, Nur Yazdani, Lisa Spainhour(2004). "Contribution of Intermediate Diaphragms in Enhancing Precast Bridge Girder Performance", Journal of Performance of Constructed Facilities, ASCE, Vol. 18 No. 3, pp. 142-146

Based upon the parametric analysis of theoretical modeling of the Florida Bulb Tee 78 precast concrete bridge girders, the following conclusions are made in this paper:

1. The presence of intermediate diaphragms helps in the stiffening of precast bridge girders and the reduction of maximum girder deflections. However, such reductions decrease with increased skew angles. The addition of intermediate diaphragms has an overall effect of reducing the deflections by about 19% for straight bridges, about 11% for 15–30° skew bridges, and about 6% for 60° skew bridges.

2. The effects of intermediate diaphragms together with a positive temperature differential also show beneficial stiffening of the girders. Such girder stiffening also decreases with an increased bridge skew angle. The maximum girder

3. Deflections can be reduced by 5.9 to 14.3%, depending on the skew angle and bearing stiffness.

4. The effects of intermediate diaphragms with negative thermal changes are also beneficial to the girder in terms of deflection reduction, although at a lesser extent than the positive

thermal change cases. Increasing skew angles result in the reduction of such a stiffening effect.

5. The increase in girder stiffness will decrease maximum girder deflections.

Based on these results, it is clear that significant benefits can be gained when considering the presence of secondary elements and these elements positively influencing structural performance.

R.E.Abendroth, F.W.Klaiber, M.W.Shafer(1995). ”Diaphragm Effectiveness in Prestressed-Concrete Girder Bridges”, Journal of Structural Engineering, ASCE, Vol.121 No.9, pp. 1362-1369

Using experimental Bridge model and Finite element model of Bridge, P/C girder's different end condition at supports resulted in significant change in rotational end restraint. With the help of experiment, it was concluded that the vertical load distribution was independent of the type and location of intermediate diaphragms, while the horizontal load distribution was a function of diaphragm type and location. The X-brace plus strut as a intermediate diaphragm were determined to be structurally equivalent to the R/C intermediate diaphragms.

Christopher D.Eamon, Andrzej S.Nowak(2002). ”Effects of Edge-Stiffening Elements and Diaphragms on Bridge Resistance and Load Distribution”, Journal of Bridge Engineering, ASCE, Vol.7 No.5, pp. 258-266

The effect of secondary elements (barriers, sidewalks, and diaphragms) on the load distribution and resistance of girder bridge structures was investigated using the finite-element method. In the elastic range, secondary elements affect both the position of maximum girder moment and the moment magnitude and can result in a 10 to 40% decrease in Girder Distribution Factor (GDF) for typical cases. For the inelastic range, typical element stiffness combinations reduce GDF by an additional 5–20%, while bridge system ultimate capacity is increased significantly, from 1.1 to 2.2 times that of the base bridge. Based on these results, it is clear that significant benefits can be gained when considering the presence of secondary elements and these elements will positively influencing structural performance, valuable for consideration of an overload.

R.R.Parikh, B.J.Shah, D.D.Patel(2001). "Computer Aided Design of Prestressed Concrete Girder", Trends in Prestressed Concrete, pp. 159-165

In this paper an attempt is made to show the various aspects of computer software package developed by the authors to analyze and design a PSC girder for Bridge. Prestressed concrete is now days extensively used in the construction of large span bridge structures. The analysis and design of a prestressed concrete member is not only a tedious task but voluminous too. The general purpose packages available for analysis and design of structures are discussed. Use of Computer Aided Design application for prestressed concrete work will result in a considerable saving in time, in addition to the minor change and economy.

Krishnaraju N. (2003). Prestressed Concrete, 10th edition, Tata MacGraw-Hill Publishing Company Limited, New Delhi.

Author covers theory and design of prestressed concrete. Explains in a very lucid language the concepts of prestressing and salient properties of high strength concrete and steel which form main constituents of prestressed concrete. Various prestressing systems are examined, analysis of stresses, losses of prestressed and deflection characteristics of prestressed member under serviceability limit states are explained. The flexural, shear and torsional resistance of prestressed concrete members and the design of anchorage zone reinforcement are examined. Development and application of limit state design concepts for the design of prestressed concrete members, design of prestressed sections, pretensioned and post-tensioned flexural members is also discussed. The book concludes with a brief introduction to the optimum design of prestressed concrete structures.

Krishnaraju N. (2003). Design of Prestressed Concrete Bridges, 8th edition, Tata MacGraw-Hill Publishing Company Limited, New Delhi.

Author carried out the design of bridge superstructures such as slab bridge girder type R.C.C. Bridge, steel concrete composite bridge, continuous bridge, rigid-frame bridges etc. The different load distribution theories such as Courbon's theory, Gouyon-massonet method, Hendry-jaeger method etc. for the girder type bridge superstructures is discussed. The book discusses different Indian standard loads, used for the design of highway bridges.

Y. GUYON (1963). Prestressed Concrete (Volume–I), 1st edition, Asia Publication House.

In this book author has given fairly detailed study of some particular applications of prestress so that it makes possible to bring out the general principles involved and to describe some methods which can be suitably modified for other applications. Also author describes in details importance of bond phenomena and stresses in the end zone of beam.

CHAPTER:-3
THEORY

3.1 ASSUMPTIONS AND LIMITATIONS

Due to limitations of the study and time constraints the following limitations are imposed on the study and study is restricted to a certain limit –

- Horizontal deviation of cable is neglected due to negligible effect on prestressing force.
- Slip of cables is assumed 6 mm
- It is assumed that no strand will fail.
- Jacking is done from both ends simultaneously.
- Only super structure is considered for analysis and substructure is not included in study.
- Only 3-lane carriageway bridges have been included in the study.
- Structure is designed only for gravity loads and earthquake forces are not considered.

3.2 DESIGN OF POST TENSIONED PRESTRESSED CONCRETE BRIDGE GIRDER

The design of prestressed concrete girder is carried out in the following steps:

1. Preliminary dimension of section
2. Section properties
3. Calculation of loads
4. Determination of prestressing force and cable profile
5. Stages of prestressing
6. Losses in prestress
7. Shear design
8. End block design

3.2.1 Preliminary dimension of section

One has to be guided by previous experience and available information to minimize the number of trials and adjustments in the dimensions of overall depth of the beam (D), thickness of flange (t_f), width of flange (b_e), breadth of web (b_w) and haunches to arrive at the appropriate section. As per fig 10 of IRC: 21-2000, which is given below, dimensions must be fixed.

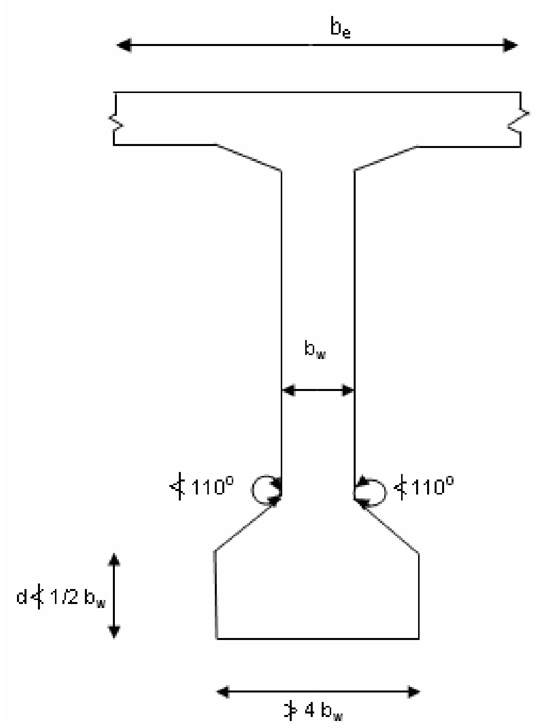


Fig. 3.1 Dimensions criteria for 'I' girders as per Fig 10 of IRC:21-2000

Some guidelines are provided in cl. 9.3 of IRC:18-2000 as under:-

'I' girders:-

- The thickness of the web (b_w) shall not be less than 200 mm plus the diameter of duct holes. Where cables cross within the web, suitable increase in the thickness over the above value shall be made.

- The effective width of the flange (b_e) of a I girder shall be taken as following, but not exceeding the actual flange width. (as per cl. 305.15.2 of IRC:21-2000)

  $b_e = b_w + 1/5\ l_0$

  Where l_0 = the distance between points of zero moments.

- The minimum thickness of the deck slab including that at cantilever tips shall be 200 mm.

Diaphragms/Cross Girders:-

- Diaphragms shall be provided depending upon design requirements. The thickness of diaphragms shall not be less than the minimum web thickness.

3.2.2 Section properties

Having determined the preliminary dimensions, the properties of section such as area, distance of c.g. from top and bottom of girder, moment of inertia, modulus of section etc. are found out. The properties of combined section considering deck slab monolithic with the girder are also found out, for which the effective width of flange of girder is decided as per cl. 305.15.2 of IRC: 21-2000.

Also, properties of combined section at transfer are calculated taking the concrete strength at transfer into account, given by formula, (Ref. limit state design of pre stressed conc. Vol. 1 by Yver Guyon)

$f_{cj} / f_7 = 2.93 – [\ 1.77 / \mathrm{sqrt}\ (\ \log_{10} J\ )\ ]$

j = age of concrete at transfer

f_{cj} = characteristic strength of concrete at j days

This gives $f_{c,28} = [\ 2.93 – 1.77 / \mathrm{sqrt}\ (\log_{10} 28)\ ] * f_7$

$f_7 = f_{c,28} / 1.459$

$f_{cj} = [\ 2.93 – 1.77 / \mathrm{sqrt}\ (\ \log_{10} j\ )\ ]\ f_{c,28} / 1.459$

From above equation characteristic strength of concrete at j^{th} days can be found.

3.2.3 Calculation of loads

When longitudinal beams are connected together by transverse members like deck slab, cross girders, diaphragms, soffit slab, etc., the distribution of bending moments between longitudinal girders shall be calculated by one of the following methods suggested by cl. 305.12.1 of IRC:21-2000.

(i) Finding the reactions on the longitudinal girders assuming the supports of the deck slab as unyielding. This method is applicable where there are only two longitudinal girders with no soffit slab

(ii) Distributing the loads between longitudinal girders by Courbon's method, strictly within its limitation, i.e., when the effective width of the deck is less than half the span and when the stiffness of the cross girders is very much greater than that of the longitudinal girders

(iii) Distributing the loads between longitudinal girders by any rational method of grid analysis, e.g., the method of harmonic analysis as given by Hendry and Jaeger or Morice and Little's version of the isotropic plate theory of Guyon and Massonet etc.

Here in this analysis and design Courbon's method is used. The basic fundamental concept of Courbon's method is assumed the cross girder (diaphragm) to be infinitely rigid and worked out the proportions of live loads on these girders based on that consideration. According to his theory, no flexure of transverse deck is possible because of the presence of infinitely rigid diaphragms, and a concentric load, instead of one pushing down only nearby girders, causes deflection of all the girders. The equation for the distribution coefficient is described in section 4.4.1 on page number 37.

3.2.3.1 Computation of Dead load

The dead load carried by a bridge member consists of its own weight and the portion of the weight of the superstructure and any fixed loads supported by the member.
The following dead loads are taken into account, and the moments and shear caused by them are found out.

a. Self weight of girder
b. Stiffener and diaphragms
c. Deck slab and deck shuttering
d. Wearing coat on clear road
e. Railings, kerb or crash barrier
f. Watermain

The dead load due to deck shuttering, wearing coat, railings, crash barrier, watermain are directly assumed.

3.2.3.2 Live load

Live loads caused by vehicles, which pass over the bridge are transient in nature. These loads cannot be estimated precisely, and the designer has a very little control over them. Once the bridge is open to traffic cl.207 of IRC: 6-2000 gives details of loading to be accounted for the design.

The live loads usually consist of a set of wheel loads, which are patch loads due to tyre contact area. These patch loads may be treated as point loads acting at the center of the contact area. This simplification is found to be acceptable in the analysis.

According to IRC classification, the main live loads for road bridges can be put into the following four types:-

IRC Class AA Loading: - IRC Class AA loading comprises either a tracked vehicle of 700kN or vehicle of 400kN loads. Fig.3.2 shows the Class AA tracked vehicle. All bridges located on National highways and State highways have to be designed for this heavy loading.

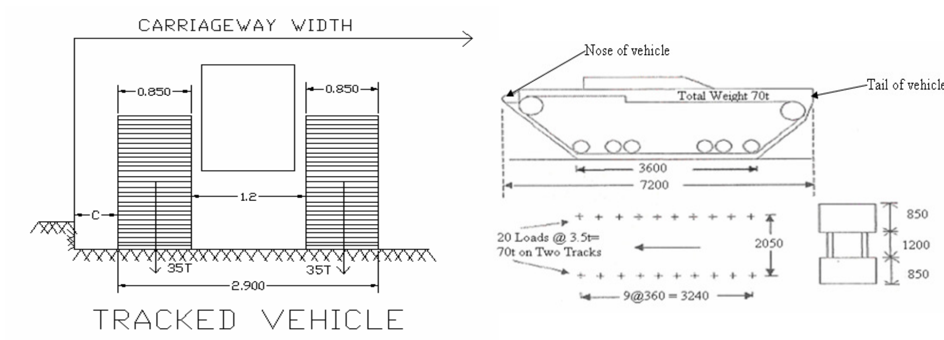


Fig. 3.2 Class AA tracked vehicle

IRC Class A Loading: - IRC Class A loading consist of a wheel load train comprising a truck with trailers of specified axle spacing and loads as detailed in fig. This type of loading is adopted on all roads on which permanent bridges and culverts are constructed. The axle loads of Class A loading is shown in fig. 3.3.

IRC Class B Loading: - Class B loading comprises a truck and truck and trailers similar to that of Class A loading but with lesser intensity of wheel loads. The axle loads of Class B loading is shown in fig. 3.3. This type of loading is adopted for temporary structures and timber bridges.

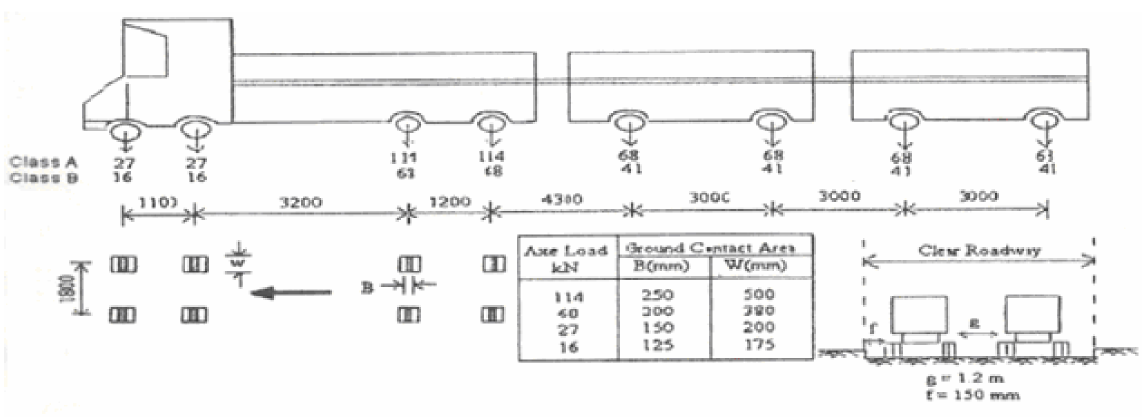


Fig. 3.3 Class A and Class B Loading

IRC Class 70R Loading: - This consists of a tracked vehicle of 700kN or a wheeled vehicle of total load is 1000 kN. The tracked vehicle is somewhat similar to Class AA, except that the contact length of truck is 4.87m, the nose to tail length of the vehicle is 7.92m. And specified minimum spacing between successive vehicles is 30m. The wheeled vehicle is 15.22m long and has seven axles with the loads totaling to 1000kN. A bogie loading of 400 KN is also specified with wheel loads of 100KN each. These details of Class 70R loading are shown in fig 3.4.

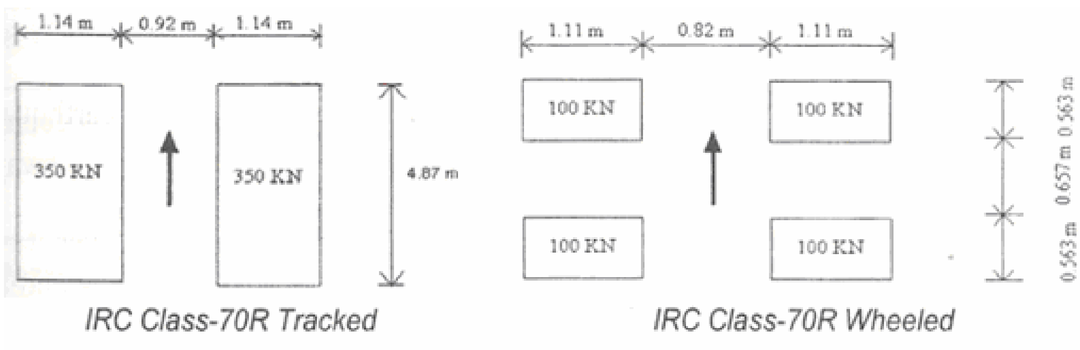


Fig. 3.4 Class-70R Loading

Maximum bending moments and shear force at different sections e.g. L/8, L/4, 3L/8, L/2 and support are found out.

3.2.3.3 Impact force (as per cl. 211 of IRC: 6-2000)

The moment and shear thus obtained are multiplied by "Impact Factor".

Provision for impact or dynamic action shall be made by a increment of the live load by an impact allowance expressed as a fraction or a percentage of the applied live loads.

For Class A or Class B Loading

In the member of any bridge designed either for Class A or Class B loading, impact factor shall be determined from the following equations which are applicable for span between 3m and 45m

Impact factor for R.C. bridge = 4.5 / (6+L)

Where, L is the length of span in m.

For Class AA and Class -70R loading

The value of the impact percentage shall be taken as follows:

(a) For span less than 9m

(i) For tracked vehicles: 25% for span up to 5m linearly reducing to 10 % for span of 9m
(ii) For wheeled vehicles: 25 %

(b) For span of 9m or more

(i) For tracked vehicles: 10 % up to a span of 40m and in accordance with the curve in Fig 5 of cl. 211 of IRC: 6-2000 for span excess of 40m.
(ii) For wheeled vehicles: 25 % for span up to 12m and in accordance with the curve in Fig 5 of cl. 211 of IRC: 6-2000 for span excess of 12m

No impact allowance shall be added to footway loading.

3.2.4 Determination of prestressing force and cable profile

As per cl. 6 of IRC: 18-2000, stage prestressing is permissible. The number of stage of prestressing and grouting shall be reduced to the minimum, preferably not more than two. However concrete shall have attained strength of not less than 20 Mpa before any prestressing is applied.

If prestressing operations are carried out in two stages. In the first stage pre stressing, a certain number of cables are stressed so that the girder is safe in taking dead load and pre stress only.

After a reasonable period of casting of deck slab the second stage pre stressing is done. The effect of this stage of pre stressing is shared by the girder and the deck slab, act together as a combined section.

The maximum permissible jacking force is 0.75 times the maximum ultimate tensile strength of the strand.

The positions of first and second stage cables are assumed for mid span and support sections. The clear spacing between adjacent cables is 75 mm minimum.

3.2.5 Stages of prestressing

Design of a prestressed concrete girder is carried out under number of stages of loading during its lifetime. However, there are two discrete stages for which the girder is mainly designed.

(1) Service dead load + pre stress with full losses

(2) Service dead load + live load + pre stress with full losses.

The girder thus designed is further checked for ultimate loads as per IRC code provisions.

3.2.6 Losses in prestress

Decrease in prestress in steel due to friction, seating of anchorages, and elastic shortening shall be deemed to be instantaneous, while that due to creep, shrinkage and relaxation of steel is time dependent. Both losses shall be calculated on the following basis:

3.2.6.1 Friction Losses (cl.11.6 of IRC:18-2000)

Steel stress in prestressing tendons $\sigma_{po}(x)$ at any distance x from the jacking end can be calculated from the formula

$\sigma_{po} = \sigma_{po}(x)\, e^{(\mu\theta + kx)}$

where, σ_{po} = steel stress at jacking end

μ = coefficient of friction

θ = cumulative angle change in radians

k = wobble coefficient per m length of steel

3.2.6.2 Losses due to Elastic shortening of concrete (cl.11.1 of IRC:18-2000)

The loss due to elastic shortening of concrete shall be computed based on the sequence of tensioning. However, for design purpose, the resultant loss of pre stress in tendons tensioned one by one may be calculated on the basis of half the product of modular ratio and the stress in concrete adjacent to the tendons averaged along the length.

3.2.6.3 Losses due to Relaxation of steel (cl.11.4 of IRC:18-2000)

Losses due to relaxation of steel shall be taken in accordance with the table give below.

Initial stress	Relaxation loss for Normal relaxation steel (%)	Relaxation loss for low relaxation steel (%)
0.5fp	0	0
0.6fp	2.5	1.25
0.7fp	5.0	2.5
0.8fp	9.0	4.5

fp = minimum Ultimate Tensile Stress (UTS) of steel

Table 3.1 Table 4A of cl. 11.4 of IRC:18-2000

3.2.6.4 Losses due to Creep & Shrinkage of concrete (cl.11.2 & 11.3 of IRC:18-2000)

This loss in tendon, stressed at a particular age of concrete, is the product of the Residual Shrinkage Strain or Creep Strain in concrete from that day onwards and E modulus of elasticity of cable steel. The values of strain due to residual shrinkage and creep are given in table 2 and table 3 of IRC:18-2000.

Maturity of concrete at the time of stressing as a percentage of f_{ck}	Creep strain per 10 Mpa
40	9.4×10^{-4}
50	8.3×10^{-4}
60	7.2×10^{-4}
70	6.1×10^{-4}
75	5.6×10^{-4}
80	5.1×10^{-4}
90	4.4×10^{-4}
100	4.0×10^{-4}
110	3.6×10^{-4}

Table 3.2 Table 2 of cl. 11.2 of IRC:18-2000

Age of concrete at the time Of stressing, in days	Stain due to residual shrinkage
3	4.3×10^{-4}
7	3.5×10^{-4}
10	3.0×10^{-4}
14	2.5×10^{-4}
21	2.0×10^{-4}
28	1.9×10^{-4}
90	1.5×10^{-4}

Table 3.3 Table 3 of cl. 11.2 of IRC:18-2000

3.2.7 Shear design

Check for the shear stress (as per cl 14 of IRC:18-2000) will be made at all specified section. Ultimate shear resistance of the section is calculated for uncracked and cracked conditions and the lesser of the two values is compared with the design shear force due to ultimate loads. If the shear force is greater the shear steel is provided. If it is less or shear steel is less than the minimum provision, minimum shear steel is provided.

3.2.8 End block design

Based on the size of the bearing plate, the bearing stress is calculated. Bursting force in the end block is calculated and for that force, the required reinforcement in the anchorage zone is provided. Checking of bearing stress behind anchorage is also carried out as per IRC-18.

3.3 DESIGN CHECKS

Once the design steps are over, the following design checks are provided.

1. Check for stresses at transfer and service load
2. Check for shear
3. Check for M.R of section
4. Check for serviceability – maximum deflection
5. Check for cracking

3.3.1 Check for stresses at transfer and service load

Design dead load bending moments (M_d), live load bending moments (M_l), prestressing force (P_0) at transfer, prestressing force (P_s) at service and its eccentricity (e), then stresses are calculated as under:

At transfer,

f(top) $= P_0/A - P_0.e / Z + M_d / Z$

f(bottom) $= P_0/A + P_0.e / Z - M_d / Z$

At service,

f(top) $= P_s/A - P_s.e / Z - M_d / Z + M_l / Z$

f(bottom) $= P_s/A + P_s.e / Z + M_d / Z - M_l / Z$

3.3.1.1 Permissible stresses in tension and compression (cl.7 of IRC:18-2000)

At full transfer temporary compressive stress in the extreme fiber of concrete (including stage pre stressing) shall not exceed 0.5_{fcj}, subject to a maximum of 20Mpa and the temporary tensile stress in the extreme fiber of concrete shall not exceed $1/10^{th}$ of the permissible temporary compressive stress in concrete.

The compressive stress in concrete under service loads shall not exceed $0.33f_{ck}$.
No tensile stress shall be permitted in the concrete during service.

Thus if, f_{ck} = 40 Mpa and transfer is at 14 days

At transfer
Maximum permissible comp. stress (temporary)

$= 0.5 * [2.93 – 1.77 / \text{sqrt}(\log_{10}14)] * 40 / 1.459 = 17.50$ Mpa

Maximum permissible tensile stress

= 0.1* permissible compressive stress = 1.75 Mpa

At service (permanent stress)
Maximum permissible comp. stress = $0.33 * f_{ck}$

$= 0.33 * 40$

= 13.33 Mpa

No tensile stress is permitted in the concrete during service.

3.3.2 Check for shear (cl. 14 of IRC:18-2000)

The calculations for shear are only required for the ultimate load. At any section the ultimate shear resistance of the concrete alone, V_c shall be considered for the section both un-cracked and cracked in flexure irrespective of the magnitude of cracking moment (M_t) and the lesser value taken.

Ultimate loads = 1.25 G + 2.0 SG + 2.5 Q (as per cl.12 of IRC:18-2000)
Here, G = permanent dead load
SG = superimposed dead load
Q = live load including impact

The shear force due to upward component of pre stress is deducted from the shear calculated on the basis of ultimate loads.

Thus net shear force (V) = shear force due to ultimate Loads – $P_i \sin\theta_i$

Where, P_i, θ_i are the pre stressing force and inclination of the i^{th} cable.

The shear resistance of concrete for a section **un-cracked** in flexure is given by:

$V_{co} = 0.67\ b\ d\ \ sqrt\ (f_t^2 + 0.8\ f_{cp}\ f_t)$ (cl..14.1.2 of IRC:18-2000)

Where, b = breath of web

d = overall depth

f_t = maximum principle tensile stress given by $0.24\sqrt{f_{ck}}$

f_{cp}= comp. stress at centroidal axis due to prestress taken as positive

The shear resistance of concrete for a section **cracked** in flexure is given by:

$V_{cr} = 0.037\ b\ d_b\ \sqrt{f_{ck}} + (M_t/M)\ V$ (cl.14.1.3 of IRC:18-2000)

Where,

d_b = distance from the extreme compression fiber to the centroid of the tendons at the section considered

M_t = cracking moment at the section considered

= $(0.37\sqrt{f_{ck}} + 0.8f_{pt})I/y$ in which f_{pt} is the stress due to pre stress only at the tensile fiber distance y from the centroid of the concrete section which has second moment of area I i.e.

$f_{pt} = (P/A) + (P.e/Z_{bot})$

V and M = shear and corresponding moment at the section considered due to ultimate load

V_{cr} = should be taken as not less than $0.1bd\sqrt{f_{ck}}$.

Shear Reinforcement design:

If V < Vc/2,	No shear reinforcement need to be provided.
If Vc/2 < V < Vc,	Minimum shear reinforcement is provided such that,
	$(A_{sv} / S_v) * (0.87\ f_{yv} / b) = 0.4$ Mpa
If V > Vc,	Shear reinforcement is provided such that,

$$(A_{sv} / S_v) = (V – Vc) / (0.87\ f_{yv}\ d_t)$$

Where,

f_{yv} = yield strength of the link/shear reinforcement <= 415 Mpa

A_{sv} = cross sectional area of two legs of a link

S_v = link spacing along the member

d_t = depth from extreme compression fiber either to the longitudinal bars or to the centroid of the tendons whichever is greater.

3.3.3 Check for Moment resistance of section

Moment due to ultimate loads is given by:-

M_{ult} = 1.25 G + 2.0 SG + 2.5 Q (cl.12 of IRC:18-2000)

The moment of resistance of section can be calculated as under

1) When failure of section is by failure of steel (under reinforced section):

M_{rult} = 0.9 db As fp

Where,

As = the area of the high tensile steel

fp = the ultimate tensile strength for steel

db = depth of girder from the max. Compression edge to the center of gravity of the steel tendons.

2) When failure of section is by crushing of concrete:

$M_{rult} = 0.176\ b\ d_b^2\ f_{ck} + 2/3 * 0.8\ (B_f – b)\ (d_b – t/2)\ t\ f_{ck}$

Where,

b = width of web

B_f = width of top flange

t = thickness of top flange

For safety, always $M_{urlt} > M_{ult}$

3.3.4 Check for serviceability – maximum deflection

There is no limit state mentioned in the present IRC Code for deflection. But as per IS:1343-1980, for type 1 structure, deflection due to the prestressing force, dead load and any sustained imposed load may be calculated using elastic analysis.

3.3.5 Check for cracking

As per cl. 19.3.1 of IS:1343-1980, the deflection is limited to:-

1. Span/250 for all loads including the effect of temp, creep and shrinkage.

2. Span/300 or 20 mm whichever is less. Including the effects of temp, creep and shrinkage after erection of partitions.

3. Span/300 upward deflection of prestressed concrete member.

Limit state of serviceability-cracking gives three types of prestressed concrete member.

Type 1. No tensile stress allow
Type 2. Tensile stress allow but no visible cracking.
Type 3. Cracking is allowed.

There is no tensile stress permissible therefore it is a Type 1 design.

CHAPTER:-4

LONGITUDINAL AND TRANSVERSE ANALYSIS OF I GIRDER SECTION

4.1 GENERAL

This part pertains to longitudinal analysis and design of PSC I girder section having effective span of 40 m. longitudinal analysis contains of analysis for flexure and shear covering following steps,

Step I : The calculation for sectional properties like area, moment of inertia, sectional modulas etc for external and internal girders for end span and mid span section.

Step II: The detailed load calculations for dead load and live load are covered. Live load bending moment and shear force for one internal girder is calculated using Matlab 7 program for IRC Class AA, Class A and Class 70R loading at 1/8th, 1/4th, 3/8th, and at mid span section

Step III: For load distribution between outer and inner girders, courben's method is used. The loading considered is IRC Class A, Class AA, and Class 70R.

Step IV: Cross section of the girder and number of cables are assumed. The cable profile assumed is parabolic with number of cables at mid span and at end span is 8. Each cable has 13T12 strands. The prestressing force of each cable is 140 kN. The prestressing is done in two stages. At each stage, It is checked that the stresses are within permissible limits of IRC. More over the check for shear and check for M.R of section is also verified as per IRC.

Step V: The calculation of losses are covered under this step, the losses covered are:

1. The loss of prestress due to Elastic shortening
2. Friction and slip loss
3. Creep and Shrinkage loss
4. Loss due to Relaxation of steel

Step VI: This step covers:

1. Design of deck slab
2. Design of diaphragm
3. Design of Anchor (end) block

4.2 DESIGN DATA

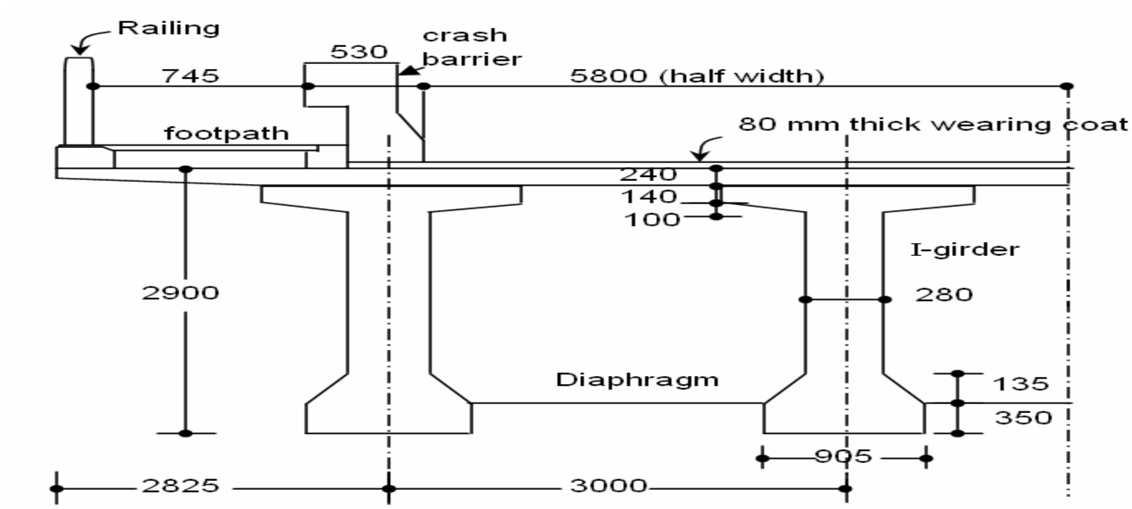


Fig 4.1 Half cross sectional view of I-type superstructure (all dimensions are in mm)

1 Carriage way width : 11.6 m (Three Lane)

2 Walk way width : 0.745 m

3. Total carriage way width : 14.65 m

4. Depth of I girder : 2.66 m

6. Wearing coat thickness : 0.08 m

7. Carriage way Live Loading

Case 1. Class A (Wheeled Vehicle) – Two lanes

Case 2. Class AA (Tracked Vehicle) – Single train

Case 3. Class 70R (Wheeled) – Single train

8. Grade of concrete : M40

9. Grade of steel : Fe415

10. Prestressing system : FREYSINET (12T13)

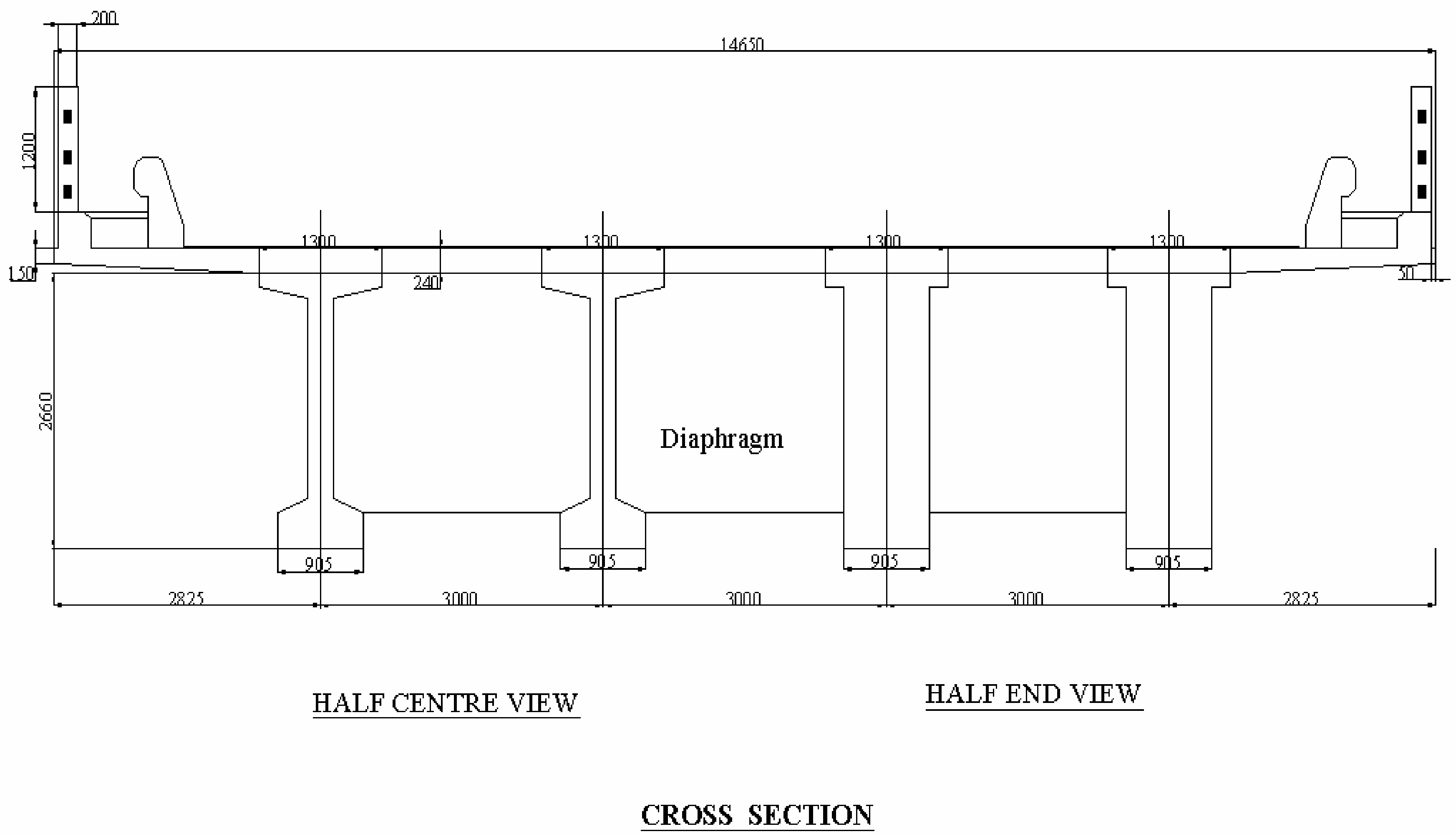


Fig 4.2 Cross sectional view of I-type superstructure (all dimensions are in mm)

4.3 CALCULATION FOR SECTIONAL PROPERTIES

Calculation for the sectional properties has been done with the help of Matlab 7.0 program. This is shown in Appendix C. Dimensions of different elements for the different sections are shown in sketches in following pages. From that centre of gravity, cross-sectional area and moment of inertia for different section are calculated. The results are presented in a tabular form as under:-

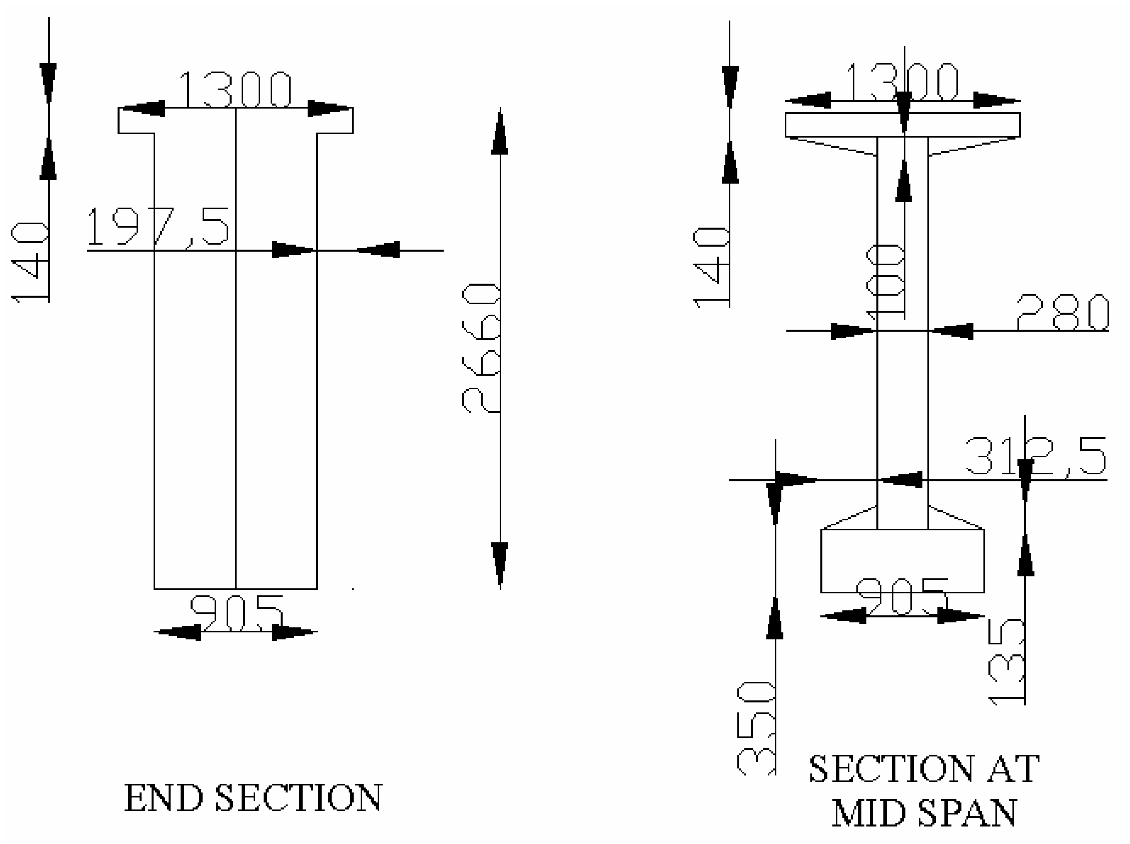


Fig 4.3 Cross section of I girder in transverse direction
(all dimensions are in mm)

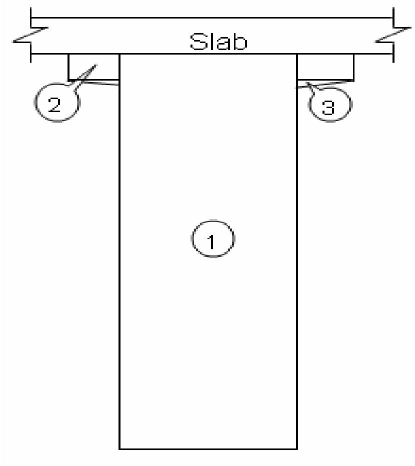


Table 4.1 Sectional properties calculations of I girder at support

Properties of Girder without Deck slab

AREA m^2	I m^4	Y_{top} m	Y_{bottom} m	Z_{top} m	Z_{bottom} m
1.74	1.00	1.19	1.47	0.84	0.68

Composite section (External girder)

AREA m^2	I m^4	$Y_{topslab}$ m	$Y_{topgirder}$ m	Y_{bottom} m	$Z_{topslab}$ m	$Z_{topgirder}$ m	Z_{bottom} m
2.71	2.07	0.96	0.80	1.93	2.15	1.95	1.07

Composite section (Internal girder)

AREA m^2	I m^4	$Y_{topslab}$ m	$Y_{topgirder}$ m	Y_{bottom} m	$Z_{topslab}$ m	$Z_{topgirder}$ m	Z_{bottom} m
2.46	1.88	1.04	0.98	1.61	1.79	1.68	1.16

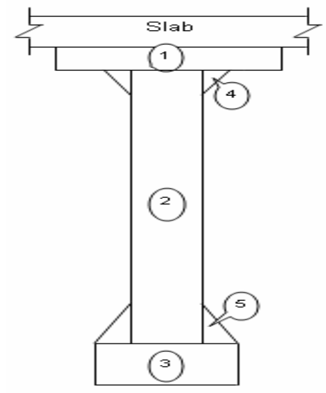


Table 4.2 Sectional properties calculations of I girder at mid span

Properties of Girder without Deck slab

AREA m^2	I m^4	Y_{top} m	Y_{bottom} m	Z_{top} m	Z_{bottom} m
1.00	0.89	1.27	1.39	0.63	0.57

Composite section (External girder)

AREA m^2	I m^4	$Y_{topslab}$ m	$Y_{topgirder}$ m	Y_{bottom} m	$Z_{topslab}$ m	$Z_{topgirder}$ m	Z_{bottom} m
1.72	1.61	0.92	0.90	1.97	1.74	1.46	0.81

Composite section (Internal girder)

AREA m^2	I m^4	$Y_{topslab}$ m	$Y_{topgirder}$ m	Y_{bottom} m	$Z_{topslab}$ m	$Z_{topgirder}$ m	Z_{bottom} m
1.96	1.75	0.82	0.80	2.07	2.12	1.98	0.84

4.5.1 Calculation of Dead loads Bending moment and Shear force

1. Self weight of girder

u.d.l. = area of uniform section x density of concrete

Here area of section at mid span = 1.0031 m^2

u.d.l. = 1.0031 x 25 = 25.08 kN/m

area of section at support = 1.7444 m^2

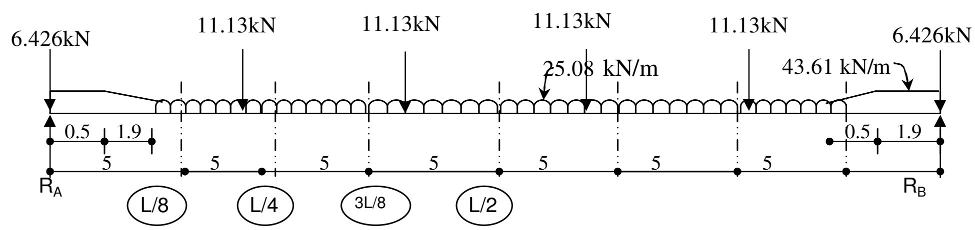
web widening at support = 1.7444 x 25 = 43.61 kN/m

intermediate stiffner = $\left[\frac{1.3+0.650}{2} \times 2.66 - 1.0031 \right] \times 0.280 \times 25$

= 11.13 kN

end stiffner = [1.3 x 2.66 - 1.7444] x 0.3 x 25

= 12.852 kN



B.M. due to self weight of girder

R_A = 516.109 kN

B.M. $_{\text{at } L/2}$ = 3914.027 kN.m

2. Cast in situ diaphragms and slab load

cast in situ intermediate diaphragm = $\left[\frac{1.775+2.1}{2} \times 2.26 + (1.775 \text{x} .15) \right] \times 0.275 \times 25$

= 31.93438 kN

Load on internal girder = 31.93438 kN

Load on external girder = 15.96719 kN

cast in situ end diaphragm = 1.775 x 2.41 x 0.3 x 25

= 32.08313 kN

Load on internal girder = 32.08313 kN

Load on external girder = 16.04156 kN

Cast in situ deck slab

Load on internal girder = 3 x 0.24 x 25 = 18 kN/m

Load on external girder = 4.025 x 0.24 x 25 = 24.15 kN/m

3. Super imposed Dead load

Description	Calculation	Load (kN)
Railing	1.2 x 0.2 x 24	5.80
Footpath slab	0.745 x 0.07 x 24	1.25
Crash barrier	area = 0.27 m^2	6.50
Wearing coat	11.6 x 0.08 x 22	23.00
Total SIDL (kN/m)		76.00

Total SIDL 76.00 kN/m

Total SIDL per Girder = 76/4 19.00 kN

Bending moment and shear force are calculated at four sections viz. mid span, 3/8th span quarter span and 1/8th span section. Considering the girder is simply supported and the values are given in table 4.4 and 4.5.

Table 4.4 B.M. @ various sections due to dead load for I - girder (kN.m)

	B.M. @ mid span		B.M. @ 3/8th section		B.M. @ 1/4th section		B.M. @ 1/8th section	
	External	Internal	External	Internal	External	Internal	External	Internal
Due to Weight of girder	3914.03	3914.03	3663.88	3663.88	2960.76	2960.76	1757.35	1757.35
Due to Cast in situ diaphragm	271.44	542.88	237.51	475.02	203.58	407.16	101.79	203.58
Due to Cast in situ slab	3489.68	2601.00	3271.57	2438.44	2617.26	1950.75	1526.73	1137.94
Due to SIDL	2745.50	2745.50	2573.91	2573.91	2059.13	2059.13	1201.16	1201.16

Table 4.5 S.F @ various sections due to dead load for I - girder (kN)

	S.F @ support		S.F @ 1/8th section		S.F @ 1/4th section		S.F @ 3/8th section	
	External	Internal	External	Internal	External	Internal	External	Internal
Due to Weight of girder	516.11	516.11	336.45	336.45	229.86	229.86	112.15	112.15
Due to Cast in situ diaphragm	39.99	79.98	23.95	47.90	23.95	47.90	7.98	15.97
Due to Cast in situ slab	410.55	306.00	307.91	229.50	205.28	153.00	102.64	76.50
Due to SIDL	323.00	323.00	242.25	242.25	161.50	161.50	80.75	80.75

4.5 LOAD CALCULATION

Dead Load

The dead load carried out by a bridge consists of its own weight, weight of diaphragm, wearing coat, footpath, railing and crash barrier. After considering all the weights Bending moments and Shear force at different sections has been calculated.

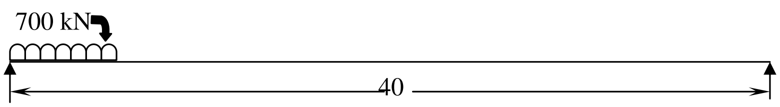
Live Load

Live load is taken as per cl. 207 of IRC: 6-2000. Live load Bending moment and Shear force has been calculated by using Matlab 7.0 program. The program finds out the position of axles for Class 70R, Class AA and Class A for maximum Bending moment and shear at different sections.

Shear force due to Live load

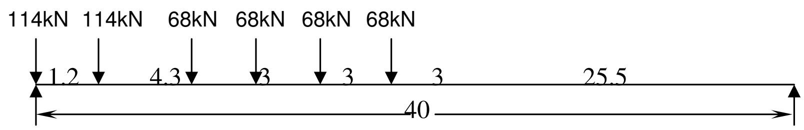
At support section

a) CLASS AA (TRACKED VEHICLE)



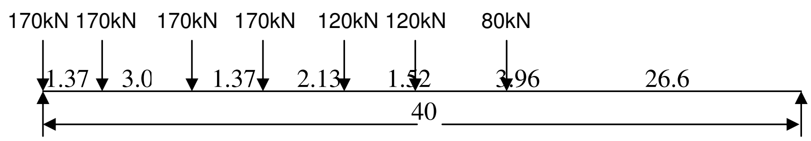
S.F = 733.5 KN

b) CLASS A TWO LANE (WHEELED VEHICLE)



S.F = 472.9 kN

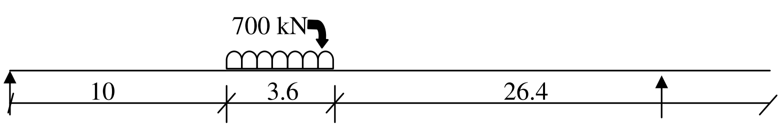
c) CLASS 70R (WHEELED VEHICLE)



S.F = 953.7 kN

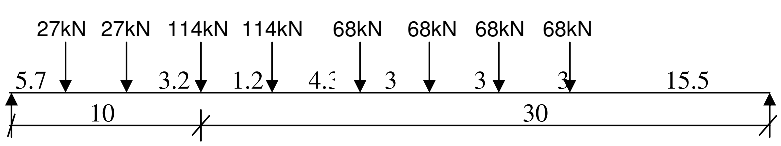
At quarter span

a) CLASS AA (TRACKED VEHICLE)



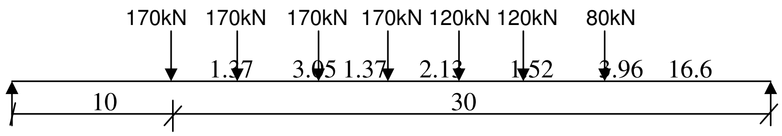
S.F = 541 kN

b) CLASS A TWO LANE (WHEELED VEHICLE)



S.F = 381 kN

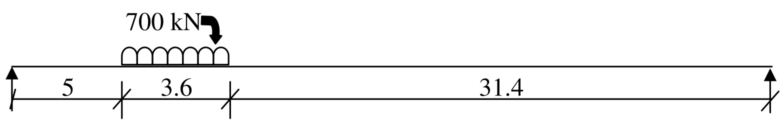
c) CLASS 70R (WHEELED VEHICLE)



S.F = 678.1 kN

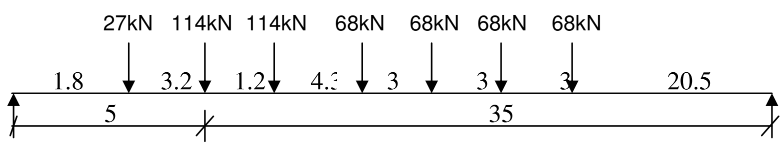
At section 1/8 span

a) CLASS AA (TRACKED VEHICLE)



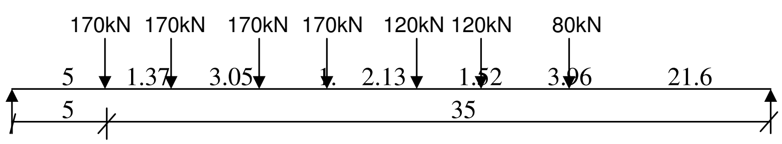
S.F = 637.3 kN

b) CLASS A TWO LANE (WHEELED VEHICLE)



S.F = 397.8 kN

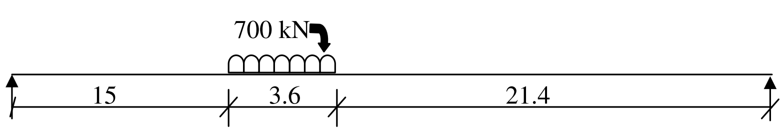
c) CLASS 70R (WHEELED VEHICLE)



S.F = 815.9 kN

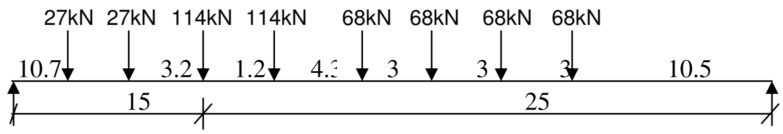
At section 3/8 span

a) CLASS AA (TRACKED VEHICLE)



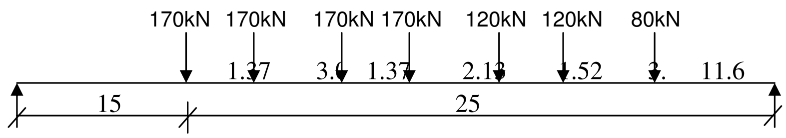
S.F = 421.2 kN

b) CLASS A TWO LANE (WHEELED VEHICLE)



S.F = 315.2 kN

c) CLASS 70R (WHEELED VEHICLE)



S.F = 554.9 kN

Table 4.8 S.F @ various sections due to live loads with impact (kN)

	S.F. @ supprt	S.F. @ /8th sectio	S.F. @ /4th sectio	S.F. @ /8th section
class AA (tracked)	733.5	637.3	541	421.2
class A (two lane)	472.9	397.8	381	315.2
class 70R (wheeled)	953.7	815.9	678.1	554.9
Footway load	127.50	31.88	63.75	95.63

Values of S.F in TABLE 4.8 are multiplied by the load distribution factors of TABLE 4.3 to get S.F on individual girder. Which is given in Table 4.9.

Table 4.9 S.F @ various sections due to live loads on individual girder (kN)

	S.F @ mid span		S.F @ 3/8th section		S.F @ 1/4th section		S.F @ 1/8th section	
	External	Internal	External	Internal	External	Internal	External	Internal
class AA (tracked)	376.77	243.08	227.58	146.83	277.31	178.91	327.04	211.00
class A (two lane)	416.50	308.52	214.45	158.85	283.79	210.21	353.08	261.54
class 70R (wheeled)	495.72	314.24	276.84	175.49	349.80	221.74	422.76	267.99
Footway load	138.13	42.50	34.53	10.63	69.06	21.25	103.59	31.88

4.6 CABLE NUMBERS, THEIR ARRANGEMENT, FRICTION AND SLIP FORCES

Assuming total cable numbers and their position at mid span section and in end span section in I girder with parabolic cable profile and two stages in which cables are prestressed, then final stresses at top and bottom at transfer and service condition are checked. If these stresses are within the permissible limit. Detailed cable forces and losses calculation is carried out and section is further checked for shear and M.R of the section. The details of prestressing cables with their eccentricities given in Appendix B of Fig no 2/3.

All the cables are having parabolic, so vertical component of prestressing force will help to counteract to some amount of shear force. The position of cables at end and at midspan is shown in Fig no. 4.4. There are totally eight cables used. Each cable transfers prestressing force of 140 kN.

The minimum spacing of cables has been kept 150 mm.

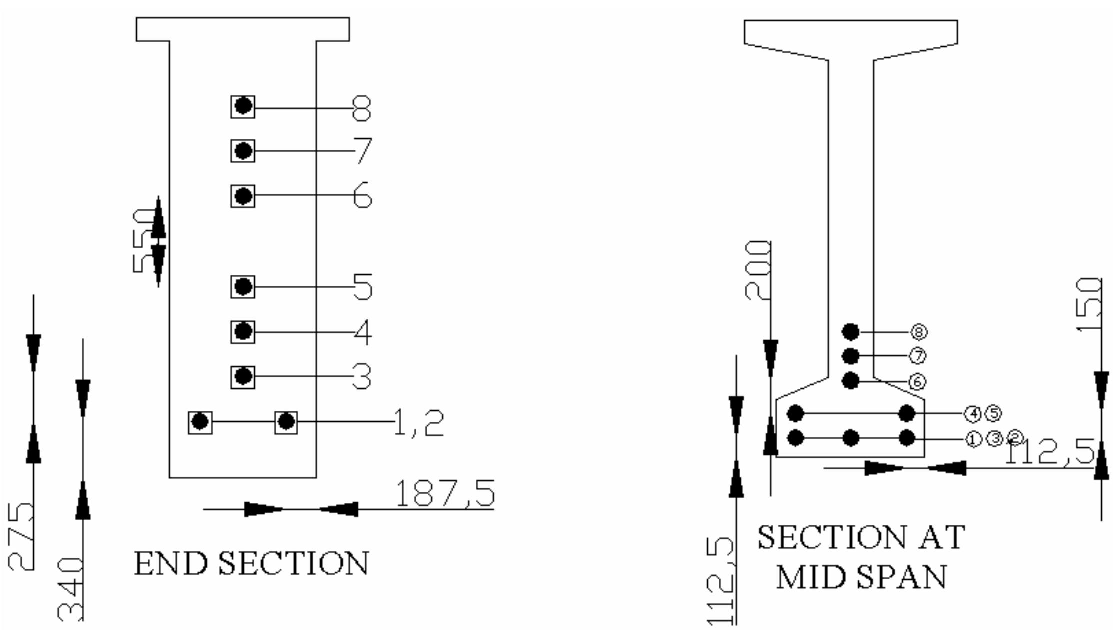


Fig 4.4 Arrangement of cables in I girder

(all diamentions are in mm)

4.6.1 Cable numbers and their arrangement

Assuming 8 cables of 12 strands. The diameter of each strands is 12.7 mm. Prestressing will be carried out in two stages. In the first stage first 6 cables will be prestressed and in the second stage remaning 2 cables will prestressed.

Assumed section & cable profile is then checked at mid span and at end span for
(i) Stresses at top and bottom (at transfer condition)
(ii) Stresses at top and bottom (at service condition)

4.6.2 Permissible stresses at top and bottom at transfer and service condition (mid and end span)

4.6.2.1 Permissible stress at transfer condition at top and bottom (mid span)

Concrete strength on 14th day is 35040 kN/m^2
As per cl. 7 of IRC : 18 -2000 permissible compression for
M_{40} grade concrete :

$$= 0.50 \text{ X } 35040 = 17520 \text{ kN/m}^2$$

4.6.2.2 Permissible stress at transfer condition at top (mid and end span)

Concrete strength on 14th day is 35040 kN/m^2
As per cl. 7 of IRC : 18 -2000 permissible compression for
M_{40} grade concrete :

$$= 0.50 \text{ X } 35040 = 17520 \text{ kN/m}^2$$

4.6.2.3 Permissible stress at service condition at top and bottom (mid and end span)

As per cl. 7 of IRC : 18 - 2000 permissible compression for
M_{40} grade concrete :

$$= 0.33 \text{ X } 40000 = 13200 \text{ kN/m}^2$$

4.6.3 Sectional properties of girder at STAGE I and STAGE II

STAGE I	**Section at Support**	**Section at L/2**
I moment of inertia (m^4)	1.51	1.01
Area (m^2)	2.46	1.16
Y_{top} (m)	1.30	1.41
Y_{bottom} (m)	1.22	1.25
Z_{top} (m^3)	1.16	0.71
Z_{bottom} (m^3)	1.24	0.81
Eccentricity of cables I^{st} stage (m)	0.37	1.03
Moment due to Self wt. of girder (kN.m) (M_d)	0.00	5846.20
No. of cables STAGE I	6	6

STAGE II Composite section	**Section at Support**	**Section at L/2**
I moment of inertia (m^4)	3.00	2.04
Area (m^2)	3.16	1.86
Y_{top} (m)	1.56	1.07
Y_{bottom} (m)	1.33	1.82
$Z_{top\ of\ slab}$ (m^3)	1.90	1.90
$Z_{top\ of\ girder}$ (m^3)	2.25	2.45
$Z_{bottom\ of\ girder}$ (m^3)	1.53	1.11
Eccentricity of cables II^{nd} stage (m)	-0.80	1.14
Eccentricity of cables all cables (m)	0.17	1.49
Moment due to Live Load (kN.m) (M_l)	0.00	9419.00
No. of cables STAGE II	2.00	2.00

Check for stresses at transfer condition at top (mid span)

For 12T13 strands as per cl. 8 of IRC : 18-2000
minimum breaking force/strand = 183.75 kN

0.75 X braking force = 137 kN (as IS : 6006-1965)

P_1=137 X 12 X 6 = 9864 kN

$\sigma_{tt} = P_1/A \ - \ (P_1 \text{ X } (e/Z_{top})) + \ M_d /Z_{top}$

$\sigma_{tt}= (9864 / 1.16) - ((9864 \text{ X } (1.03/0.71))) + (5846.20/0.71)$

$\sigma_{tt} = \ 8500 - 14309 + 8234 = \ 2425 \text{ kN/m}^2 < 17520 \text{ kN/m}^2$

Check for stresses at transfer condition at bottom (mid span)

$\sigma_{tb} = P_1/A \ + \ (P_1 \text{ X } (e/Z_{bottom})) \ - \ M_d/Z_{bottom}$

$\sigma_{tb}= (9864 / 1.16) + ((9864 \text{ X } (1.03/0.81))) - (5846.20/0.81)$

$\sigma_{tb} = 8500 + 12540 - 8234 = 12800 \text{ kN/m}^2 < 17520 \text{ kN/m}^2$

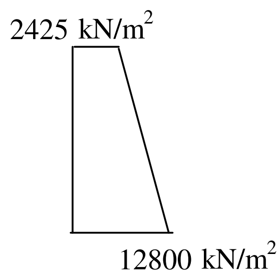


Fig 4.5 Stress diagram at transfer at mid span

Check for stresses at transfer condition at top (end span)

$\sigma_{tt} = P_1/A \ - \ P_1 \, e/Z_{top}$

$\sigma_{tt} = 4009 - 3146 = 863 \text{ kN/m}^2 < 17520 \text{ kN/m}^2$

Check for stresses at transfer condition at bottom (end span)

$\sigma_{tb}P_1/A \ + \ P_1 e/Z_{bottom}$

$\sigma_{tb} = 4009 + 2943 = 6952 \text{ kN/m}^2 < 17520 \text{ kN/m}^2$

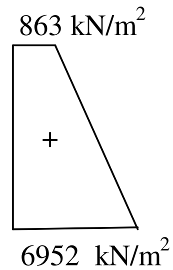


Fig 4.6 Stress diagram at transfer at end span

Check for stresses at service condition at top (mid span)

P_2 = 137 X 12 X 2 = 3288 kN

Assuming 20% loss in posttensioning for both Ist and IInd stage prestressing.

$\sigma_{st} = 0.8P_1/A + P_2/A - (0.8P_1 \text{ X} (e/Z_{top})) - P_2e/Z_{top} + M_d/Z_{top} + M_l/Z_{top}$

σ_{st}= (0.8 X 9864 / 1.16) + 3288 / 1.86 - ((0.8 X 9864 X (1.03 / 1.9)) -

(3288 X 1.14 / 1.9) + (5846.20 / 1.9) + (9419 / 1.9)

σ_{st} = 6802 + 1767 - 4277 - 1972 + 3076 + 4957 = 10353 kN/m^2<13200kN/m^2

Check for stresses at service condition at bottom (mid span)

$\sigma_{sb} = 0.8 P_1/A + P_2/A + 0.8P_1e/Z_b + P_2e/Z_b - M_d/Z_b - M_l/Z_b$

σ_{sb}= (0.8 X 9864 / 1.16) + 3288 / 1.86 + ((0.8 X 9864 X (1.03 / 1.11)) +

(3288 X 1.14 / 1.11) - (5846.20 / 1.11) - (9419 / 1.11)

σ_{sb} =6802 +1767 + 7322 + 3376 - 5266 - 8485 =5515 kN/m^2<13200kN/m^2

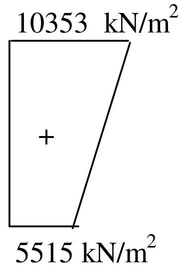


Fig 4.7 Stress diagram at service at mid span

Check for stresses at service condition at top (end span)

σ_{st}= 0.8 P_1/A + P_2/A - 0.8P_1e/Z_t - P_2e/Z_t

σ_{st} = 3207 + 1040 - 4277 - 1972 = 2320 kN/m^2 < 13200 kN/m^2

Check for stresses at service condition at bottom (end span)

σ_{sb} = 0.8 P_1/A + P_2/A + 0.8P_1e/Z_b + P_2e/Z_b

σ_{sb} = 3207 + 1040 + 5312 + 2449 = 12008 kN/m^2 < 13200 kN/m^2

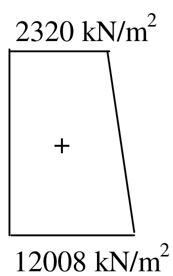


Fig 4.8 Stress diagram at service condition at end span

4.7 CALCULATION OF LOSSES

In this section Losses due to elastic shortening, Creep, Shrinkage and relaxation are calculated and from that stresses at different sections are also calculated. Creep and Shrinkage co-efficient are selected from IRC:18-2000.

Prestressing stages:

Stage 1: Nos. 1, 2, 3, 4, 5, 6 will be stressed at 14^{th} day after casting of girder.

Stage 2: Nos. 7 and 8 will be stressed at 56^{th} day after casting of girder or 21^{th} days of casting of deck slab and cross girder.

4.7.1 Calculation Friction and Slip losses for each cable (as per cl. 11.6 of IRC : 18-200

Table 4.10 Geometry of cable (m)

Cable	Distance from bottom at end span (A)	Distance from bottom at mid span (B)	hz. len of parabolic profile	h=A-B	$\frac{2 \times h}{\text{hz. len}}$	change in angle t (radian)
8	2.27	0.76	20.30	1.51	0.14828	0.14720
7	1.99	0.61	20.30	1.38	0.13596	0.13513
6	1.72	0.46	20.30	1.26	0.12365	0.12302
5	1.17	0.26	20.30	0.91	0.08916	0.08893
4	0.89	0.26	20.30	0.63	0.06207	0.06199
3	0.62	0.11	20.30	0.51	0.04975	0.04971
2	0.34	0.11	20.30	0.23	0.02266	0.02266
1	0.34	0.11	20.30	0.23	0.02266	0.02266

Table 4.11 Ordinates of cable at different sections (m)

Cable	DISTANCE FROM BOTTOM AT DIFFERENT SECTIONS					
	end	c/l bearing	L/8	L/4	3L/8	L/2
	0	0.3	5.3	10.3	15.3	20.3
8	2.27	2.22	1.58	1.13	0.85	0.76
7	1.99	1.95	1.36	0.94	0.69	0.61
6	1.72	1.68	1.15	0.76	0.54	0.46
5	1.17	1.14	0.75	0.48	0.31	0.26
4	0.89	0.87	0.60	0.41	0.30	0.26
3	0.62	0.60	0.39	0.23	0.14	0.11
2	0.34	0.33	0.24	0.17	0.12	0.11
1	0.34	0.33	0.24	0.17	0.00	0.11

Table 4.12 Angle made by the tangent (Radian)

Cable	Section at					
	end	c/l bearing	L/8	L/4	3L/8	L/2
	0	0.3	5.3	10.3	15.3	20.3
8	0.15	0.15	0.11	0.07	0.04	0
7	0.14	0.13	0.10	0.07	0.03	0
6	0.12	0.12	0.09	0.06	0.03	0
5	0.09	0.09	0.07	0.04	0.02	0
4	0.06	0.06	0.05	0.03	0.02	0
3	0.05	0.05	0.04	0.02	0.01	0
2	0.02	0.02	0.02	0.01	0.01	0
1	0.02	0.02	0.02	0.01	0.01	0

Table 4.13 Calculation of stresses after friction loss (as per cl. 11.6 of IRC : 18-2000)

Cable 8 (The detailed calculations are shown on Page 57)

Section	Distance from End(m)	kL	θ_v	$\mu\theta_v$	$x = kL+\mu\theta_v$	$F=e^x$
c/cbearing	0.3	0.001	0.15	0.03652	0.038	1.04
L/8	5.3	0.024	0.11	0.02739	0.052	1.05
L/4	10.3	0.047	0.07	0.01826	0.066	1.07
3L/8	15.3	0.070	0.04	0.00913	0.080	1.08
L/2	20.3	0.093	0.00	0.00000	0.093	1.10

Cable 7

Section	Distance from End(m)	kL	θ_v	$\mu\theta_v$	$x = kL+\mu\theta_v$	$F=e^x$
bearing	0.3	0.001	0.13	0.03	0.03	1.04
L/8	5.3	0.024	0.10	0.03	0.05	1.05
L/4	10.3	0.047	0.07	0.02	0.06	1.07
3L/8	15.3	0.070	0.03	0.01	0.08	1.08
L/2	20.3	0.093	0.00	0.00	0.09	1.10

Cable 6

Section	Distance from End(m)	kL	θ_v	$\mu\theta_v$	$x = kL+\mu\theta_v$	$F=e^x$
bearing	0.3	0.001	0.12	0.03	0.03	1.03
L/8	5.3	0.024	0.09	0.01	0.04	1.04
L/4	10.3	0.047	0.06	0.00	0.05	1.05
3L/8	15.3	0.070	0.03	0.00	0.07	1.07
L/2	20.3	0.093	0.00	0.00	0.09	1.10

Sample Calculation
For section at L/2 (Cable 8)

Taking wobble coefficient per m length of steel (k) = 0.0046
Coefficient of friction (μ) = 0.25

Distance form end (L) = 20.3 m
kL = 0.0046 X 20.3 = 0.09338
θ_v = 0 (at mid span cumulative angle change will zero)
$\mu\theta v = 0$

$x = kL + \mu\theta v = 0.09338 + 0 = 0.09338$

$F = e^x = 1.10$

stress = (183.75 X 0.75 / 98.7) = 1.38 kN/mm^2

Now after friction loss

Stress = 1.38 / 1.10 = 1.26 kN/mm^2 (per strand)

End section Mid section

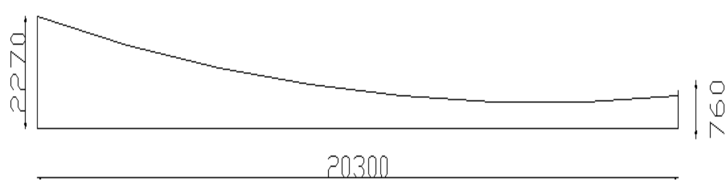


Fig 4.9 Profile of cable number 8

Cable 5

Section	Distance from End(m)	kL	θ_v	$\mu\theta_v$	$x = kL+\mu\theta_v$	$F=e^x$
bearing	0.3	0.001	0.09	0.02	0.02	1.02
L/8	5.3	0.023	0.07	0.02	0.04	1.04
L/4	10.3	0.046	0.04	0.01	0.06	1.06
3L/8	15.3	0.069	0.02	0.01	0.07	1.08
L/2	20.3	0.092	0.00	0.00	0.09	1.10

Cable 4

Section	Distance from End(m)	kL	θ_v	$\mu\theta_v$	$x = kL+\mu\theta_v$	$F=e^x$
bearing	0.3	0.001	0.06	0.02	0.02	1.02
L/8	5.3	0.024	0.05	0.01	0.04	1.04
L/4	10.3	0.047	0.03	0.01	0.06	1.06
3L/8	15.3	0.070	0.02	0.00	0.07	1.08
L/2	20.3	0.093	0.00	0.00	0.09	1.10

Cable 3

Section	Distance from End(m)	kL	θ_v	$\mu\theta_v$	$x = kL+\mu\theta_v$	$F=e^x$
bearing	0.3	0.001	0.05	0.01	0.01363	1.01
L/8	5.3	0.024	0.04	0.01	0.03357	1.03
L/4	10.3	0.047	0.02	0.01	0.05351	1.05
3L/8	15.3	0.070	0.01	0.00	0.07344	1.08
L/2	20.3	0.093	0.00	0.00	0.09	1.10

Cable 1 & 2

Section	Distance from End(m)	kL	θ_v	$\mu\theta_v$	$x = kL+\mu\theta_v$	$F=e^x$
bearing	0.3	0.00	0.02	0.01	0.01	1.01
L/8	5.3	0.02	0.02	0.00	0.03	1.03
L/4	10.3	0.05	0.01	0.00	0.05	1.05
3L/8	15.3	0.07	0.01	0.00	0.07	1.07
L/2	20.3	0.09	0.00	0.00	0.09	1.10

4.7.2 Summary of Force at different section after Friction and Slip loss

	Section at			
	support	L/8	L/4	3L/8
Cable 8				
Prestressing force (kN)	1428.20	1449.90	1471.40	1492.50
Horz.component (kN)	1412.89	1441.15	1467.45	1491.50
Vert.component (kN)	208.58	159.07	107.74	54.68
Cable 7				
Prestressing force (kN)	1428.60	1451.70	1474.40	1496.80
Horz.component (kN)	1415.80	1444.38	1471.09	1495.96
Vert.component (kN)	189.08	144.86	98.45	50.09
Cable 6				
Prestressing force (kn)	1429.10	1453.40	1477.30	1500.80
Horz.component (kN)	1418.43	1447.29	1474.54	1500.10
Vert.component (kN)	174.34	133.13	90.28	45.88
Cable 5				
Prestressing force (kN)	1431.20	1459.00	1486.30	1513.20
Horz.component (kN)	1425.62	1455.80	1484.85	1512.83
Vert.component (kN)	126.25	96.59	65.62	33.41
Cable 4				
Prestressing force (kN)	1433.30	1464.00	1494.10	1523.70
Horz.component (kN)	1430.62	1462.46	1493.40	1523.52
Vert.component (kN)	87.59	67.12	45.68	23.29
Cable 3				
Prestressing force (kN)	1434.30	1466.30	1497.60	1527.50
Horz.component (kN)	1432.54	1465.29	1497.14	1527.38
Vert.component (kN)	70.97	54.43	37.06	18.90
Cable 1 & 2				
Prestressing force (kN)	1437.00	1471.90	1506.00	1530.10
Horz.component (kN)	1436.64	1471.69	1505.91	1530.08
Vert.component (kN)	32.55	24.64	16.81	8.54

First stage prestressing	12T13	1,2,3,4,5,6	n_1 = 6 nos
Second stage prestressing	12T13	7,8	n_2 = 2 nos

Total force after friction and slip STAGE I				
Prestressing force (kN)	8601.90	8786.50	8967.30	9125.40
Horz.component (kN)	8580.49	8774.23	8961.74	9123.99
Vert.component (kN)	524.27	400.55	272.27	138.57
Total force after friction and slip STAGE 2				
Prestressing force (kN)	2856.80	2901.60	2945.80	2989.30
Horz.component (kN)	2828.69	2885.53	2938.54	2987.46
Vert.component (kN)	397.66	303.93	206.19	104.76
Eccentricity of group of cables from bottom of Girder				
First stage (m)	0.84	0.05	0.03	0.02
Second stage (m)	2.13	1.47	1.04	0.77
For all cables (m)	1.17	0.79	0.54	0.37

)0)

Length of parabolic cables upto mid span
20.34
20.34
20.34
20.36
20.39
20.41
20.54
20.54

Length of parabola
40.6
40.6
40.6
40.6
40.6
40.6
40.6
40.6

hz. Lengthof parabola upto mid span
20.3
20.3
20.3
20.3
20.3
20.3
20.3
20.3

Stress kN/mm^2
1.34
1.32
1.30
1.28
1.26

Stress kN/mm^2
1.33
1.31
1.29
1.28
1.26

Stress kN/mm^2
1.34
1.33
1.31
1.29
1.26

Stress kN/mm^2
1.35
1.33
1.30
1.28
1.26

Stress kN/mm^2
1.36
1.33
1.31
1.28
1.26

Stress kN/mm^2
1.36
1.33
1.31
1.28
1.26

Stress kN/mm^2
1.37
1.34
1.31
1.29
1.26

L/2
1497.40 1497.40 0.00
1497.40 1497.40 0.00
1497.40 1489.56 0.00
1497.40 1443.51 0.00
1497.40 1395.54 0.00
1497.40 1456.34 0.00
1497.40 1448.72 0.00

8984.40 8682.39 0.00
2994.8 2994.8 0
0.22 0.69 0.34

CHAPTER:-5

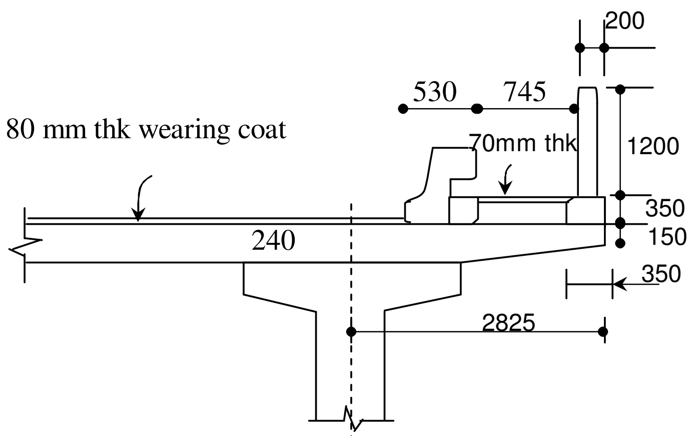
DESIGN OF SLAB, DIAPHRAGM AND END BLOCK

5.1 LOAD CALCULATION FOR TRANSVERSE ANALYSIS OF DECK SLAB

The design of deck slab is carried out for heaviest loading of IRC 70R, 40t bogi. For maximum positive moment Class 70R, 40t bogi placed at centre of span and for maximum negative moment Class 70R, 40t bogi placed at centre of support. Analysis is carried out using STAAD Pro 2003. Input file for the same is shown in Appendix A.

5.1.1 Dead load calculation

1 Calculation of loads & moment at cantilever throat



Dead load of slab

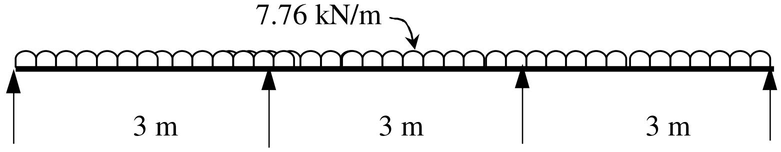
Description	Calculation	Load(kN)	L.A.(m)	Moment (kN.m)
Deck slab (Rect 1)	2.685 x 0.15 x 25	10.07	1.34	13.52
Deck slab (Tri)	0.5(2.175x0.09x25)	2.44	0.73	1.77
Deck slab (Rect 2)	0.510 x 0.24 x 25	3.06	0.26	0.78
Total		15.57	1.03	16.07

Super imposed dead load

Description	Calculation	Load(kN)	L.A.(m)	Moment (kN.m)
Railing	1.2 x 0.2 x 24	5.80	2.51	14.56
Outer kerb	0.35 x 0.35 x 24	2.94	2.51	7.38
Inner kerb	0.35 x 0.225 x 24	1.90	1.57	2.97
Footpath slab	0.745 x 0.07 x 24	1.26	2.11	2.66
Wearing coat	0.08 x 1.210 x 22	2.13	0.61	1.29
	Total	14.03		28.86

2 Calculation of load and moment for internal slab.

weight of slab	=	0.24 x 25	6 kN/m
weight of wearing coat	=	0.08 x 22	1.76 kN/m
Total weight	=		7.76 kN/m

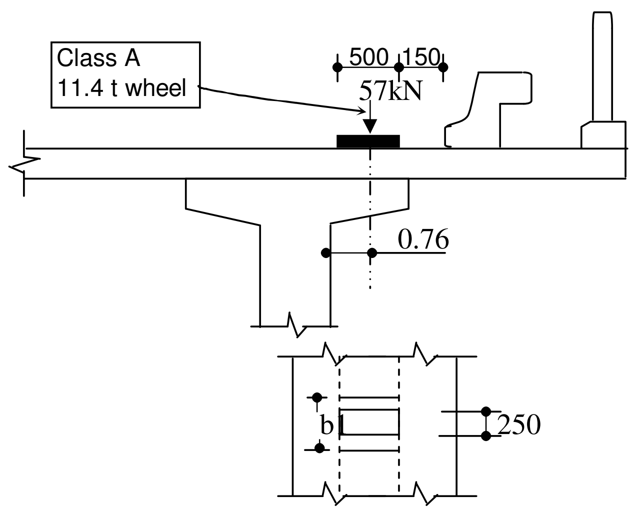


From STAAD.Pro 2003 output

Max +ve moment(span moment)	5.52 kN-m
Max -ve moment (support moment)	6.97 kN-m

5.1.2 Live load calculation for I girder deck slab

1. Calculation of loads & moment at cantilever throat



plan of wheel loading

As per cl. 305.16.2 of IRC:21-2000 for cantilever solid slab

$b_{ef} = 1.2a + b_1$

here, $a =$ 0.76

$b_1 = 0.25 + (2 \times 0.08) =$ 0.41

$b_{ef} =$ 1.322 m

Load/m. longitudinal run, w = 57/1.322

= 43.12 kN

Impact factor = 1.5 (As per fig 5 of IRC:6-2000)

w = 43.12 x 1.5

= 64.68 kN

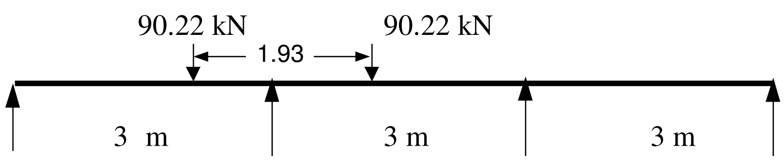
Moment at cantilever throat = 64.68 x 0.76

= 49.157 kN.m

2. Calculation of load for deck slab.

Case 1 : Analysis of deck slab with Class 70R, 40t bogi placed at central support

Load per wheel		100 kN
Impact factor = 4.5/(6+L)		1.50
Distance between two wheel		1.93 m
Distance bet. Cg of concen. Load from nearest support		0.965
Value of a for continous slab from IRC-21 clause 305.16.2		2.6
bw	0.263 + (2 x 0.08) =	0.423 m
Bef = (a*a*(1-a/l_0)) + bw		2.1249 m
Bef > 1.2 m then overlapping		
Dispersion width in transverse direction	(eff width+1.2)/2	1.6625
Intensity of load/m length with impact		902.2724 kN/m

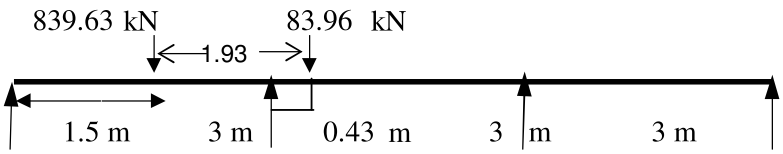


From STAAD.Pro 2003 output

Max +ve moment(span moment)	25.72 kN-m
Max -ve moment (support moment)	47.55 kN-m

Case 2 : Analysis of deck slab with Class 70R, 40t bogi placed at center of span

Load per wheel		100 kN
Impact factor = 4.5/(6+L)		1.50
Distance between two wheel		1.93 m
Distance bet. Cg of concen. Load from nearest support		1.5
Value of a for continous slab from IRC-21 clause 305.16.2		2.6
bw	0.263 + (2 x 0.08) =	0.423 m
Bef =(a*a*(1-a/l_0)) + bw		2.373 m
Bef > 1.2 m then overlapping		
Dispersion width in transverse direction	(eff width+1.2)/2	1.7865
Intensity of load/m length with impact		839.63 kN/m



From STAAD.Pro 2003 output

Max +ve moment(span moment)	43.97 kN-m
Max -ve moment (support moment)	38 kN-m

5.2 DESIGN OF DECK SLAB REINFORCEMENT

5.2.1 Design of cantilever slab (Crash barrier side)

Design moment

Dead load moment	=	16.07	kN.m
Moment due to SIDL	=	28.86	kN.m
Live load moment	=	49.1568	kN.m
Total moment	=	94.08	kN.m

Depth required $= \sqrt{\dfrac{Mc}{b \text{ x } Q}}$

$= \sqrt{\dfrac{94.08 \text{ x } 10^6}{1000 \text{ x } 2.3075}}$

$=$ 201.919 mm

d_{prod} = 240 - 30 - 5

= 205 mm (o.k.)

Ast required $= \dfrac{94.08 \text{ x } 10^6}{200 \text{ x } 0.8668 \text{ x } 205}$

$=$ 2647.25 mm^2

Provide 12 f @ 150 c/c + 20 f @ 150 c/c Ast prov 2848.0 mm^2

5.2.2 Design of deck slab

Max +ve moment (**span moment**) = 5.52 + 43.97 = 49.49 kN.m

Area of steel = Ast = M/sst*j*d 1392.562 mm^2

Provide 10 f @ 200 c/c + 16 f @ 200 c/c Ast prov 1398.0 mm^2

Max -ve moment (**support moment**) = 6.97 + 47.55 = 54.52 kN.m

Area of steel = Ast = M/sst*j*d 1534.098 mm^2

Provide 12 f @ 150 c/c + 16 f @ 150 c/c Ast prov 2094.0 mm^2

5.2.3 Distribution Reinforcement

1. For cantilever slab

Design moment = 0.3*BM due to LL + 0.2*BM due to DL = 23.73199 kN.m
Area of steel = Ast = M/sst*j*d 667.7769 mm^2
Provide half at top and half at bottom
Provide 10 f @ 200 c/c Ast,prod= 785 mm^2

2. Deck slab
a.) Top reinforcement:-

Design moment = 0.3*BM due to LL + 0.2*BM due to DL = 15.659 kN.m
Area of steel = Ast = M/sst*j*d 440.617 mm^2
Provide 10 f @ 150 c/c Ast,prod= 523.3333 mm^2

b.) Bottom reinforcement:-

Design moment = 0.3*BM due to LL + 0.2*BM due to DL = 14.295 kN.m
Area of steel = Ast = M/sst*j*d 402.2364 mm^2
Provide 10 f @ 190 c/c Ast,prod= 413.1579 mm^2

Deck slab reinforcement sketch is given on Appendix B.

5.3 DESIGN OF DIAPHRAGM OF GIRDER

Diaphragms are provided to strengthen the deck. Due to this no flexure of transverse deck is possible. According to Courbon's theory a concentric load, instead of pushing down only nearby girders, causes equal deflection of all the girders. It is designed as the simple RCC unit. Analysis is carried out using STAAD Pro 2003. Input file for the same is shown in Appendix A.

Thickness of End diaphragm = 0.3 m
Thickness of Intermediate diaphragm = 0.28 m

5.3.1 Intermediate diapharm

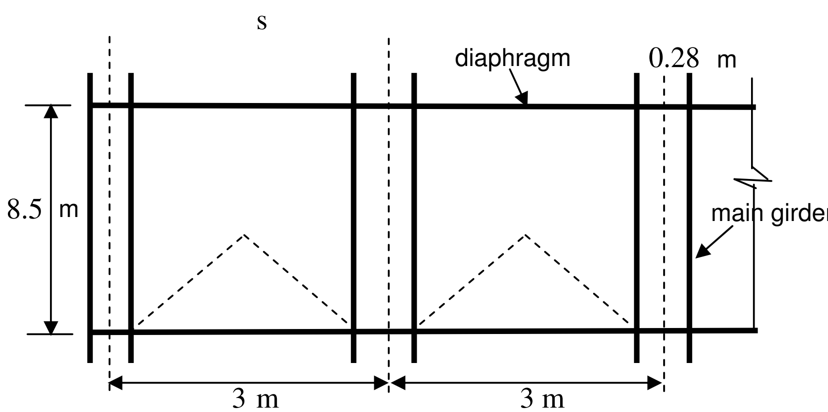


Fig 5.1 Dead Load distribution on diaphragm

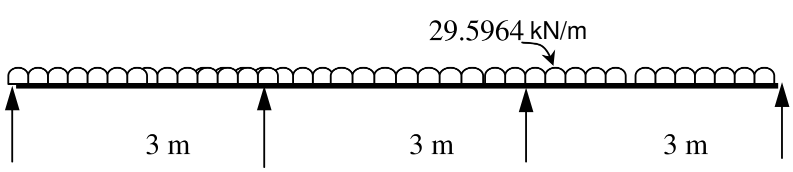
Dead Load calculation.

Clear span of diaphragm = 3-0.28
= 2.72 m

D.L. of deck slab + wearing coat /m run
= [(0.24 x 25) + (0.080 x 22)] x [(2.72/2)+0.28]
= 12.7264 kN/m

Self weight of diaphragm = 2.41 x 0.28 x 25
= 16.87 kN/m

Total D.L. = 29.5964 kN/m



From STAAD.Pro 2003 output

Max +ve moment(span moment)	21.3 kN-m
Max -ve moment (support moment)	26.59 kNn-m
Max S.F. due to D.L. at internal support	51.51 kN

Live load calculation

Case 1 : Class AA placed for middle support moment

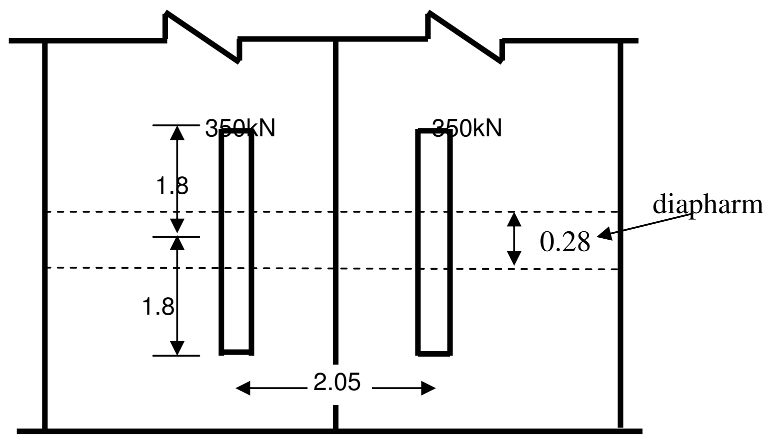
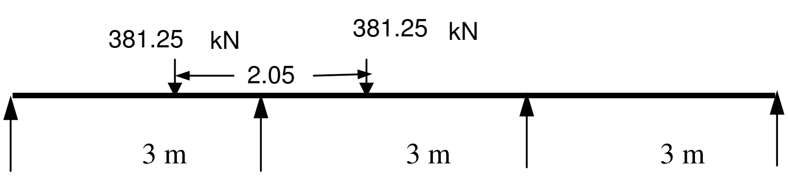


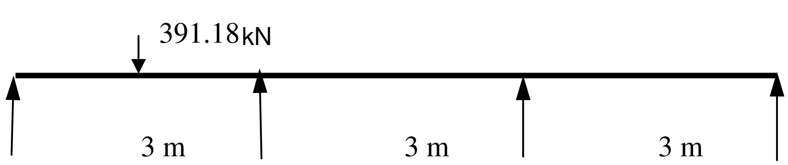
Fig 5.2 Live Load distribution on intermediate diaphragm

Load on cross beam	=	350 x (8.5 - 0.9) / 8.5	=	312.941 kN
With impact (25 % higher)		1.25	=	391.18 kN



Max +ve moment(span moment)	122.56 kN-m
Max -ve moment (support moment)	209.32 kN-m

Case 2 : Class AA placed at centre of span



Max +ve moment(span moment)	229.06 kN-m
Max -ve moment (support moment)	113.65 kN-m

Total design Moment.(D.L.+ L.L.)

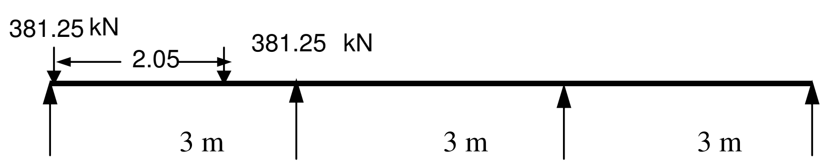
Maximum +ve (span) moment 250.36 kN.m

Maximum -ve (support) moment 235.91 kN.m

Design of intermidiate diapharm

Depth of diapharm	2.65	m
Effective depth	2587.5	mm
Max +ve moment	250.36	kN.m
Area of steel = Ast = M/sst*j*d	558.13	mm^2
Min steel area required = 0.85bd / fy =	1483.92	mm^2
Ast provided =	1483.92	mm^2
bar dia	25	mm
No of bar 3 no Ast,prod	1471.88	mm^2

SF calculation



Shear at internal support due to live load	=	297.21	kN
Total design shear (D.L. + L.L.)	=	348.72	
shear stress = V/(b*d)	=	481.325	kN/m^2
Permissible stress for concrete 100*As/(b*d)	=	0.20316	

From IRC-21 page 37 table 12B

value corrosponding to 0.203157 0.26 kN/m^2

Area of shear reinforcement required V.s/(ss.d) 134.771 mm^2

Provide 2-L 10 f at 200 mm c/c

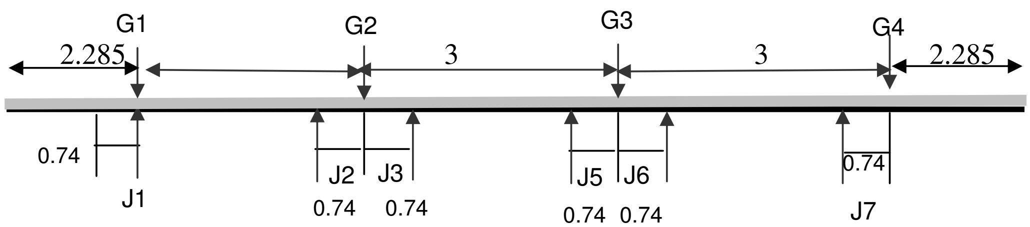
5.3.2 End diapharm

BM calculation

Total dead wt of super structure

Weight due to SIDL	1524.00	kN
Weight due to Slab	565.00	kN
Weight due to Girder	55.00	kN
Weight due to Diapharm	56.00	kN
Total	2250.00	kN

Load @ each bearing joint 375.00 kN



From Staad.pro output,

Maximum +ve (span) moment 138.75 kNm

Maximum -ve (support) moment 947.84 kNm

Design of end diapharm

Depth of diapharm	2.65	m
Effective depth for bottom steel	2587.5	mm
Effective depth for top steel	2535	mm
Max +ve moment	138.75	kN.m

Area of steel = Ast = M/sst*j*d 309.32 mm^2

Min steel area required = 0.85bd / fy = 1589.91 mm^2

Ast provided = 1589.91 mm^2

bar dia 25 mm

No of bar 4 no Ast,prod 1962.5 mm^2

Max -ve moment 947.84 kN.m

Area of steel = Ast = M/sst*j*d 2156.79 mm^2

Ast provided 2156.79 mm^2

Provide 4-20f + 2-25f Ast,prod 2237.25 mm^2

SF calculation

SF result from STAAD Pro
Max. Shear force 1280.86 kN

shear stress = V/(b*d) = 1650.06 kN/m^2

Area of shear reinforcement required V.s/(ss.d) 371.264 mm^2

Provide 4-L 12 f at 150 mm c/c

Distribution steel

0.2% of C/S area IRC-21 Clause-305.19

Ast provided for intermediate diaphragm 1484 mm^2

Provide 10 f at 200mm c/c on both face

Ast provided for end diaphragm 1590 mm^2

Provide 10 f at 200mm c/c on both face

Diaphragm reinforcement sketch is given on Appendix B.

5.4 DESIGN OF ANCHOR BLOCK FOR I - GIRDER

Based on the size of bearing plate, the bearing stress is calculated. Bursting force in the end block is calculated and for that force, the required reinsforcement in the anchorage zone is provided. Checking of bearing stress behind anchorage is also carried out as per IRC : 18-2000.

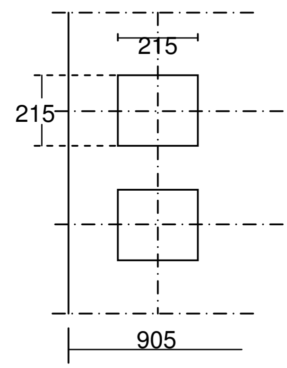
Permissible bearing stress behind Anchorage (Cl 7.3 IRC:18-2000)

$f_b = 0.48$ fcj sqrt(A_2/A_1)

or 0.8 fcj whichever is smaller

Here $A_1 = 0.215 \times 0.215 = 0.046225\ m^2$

$A_2 = 0.350 \times 0.350 = 0.1225\ m^2$



$f_b = 0.48$ fcj sqrt(A_2/A_1)

= 31255.8 kN/m2

and 0.8 fcj = 32000 kN/m2

Therefor f_b = 31255.8 kN/m^2

Maximum cable force after immediate losses

= (0.7 x 183.7 x 12) - (3413.31/8)

= 1116.42 kN

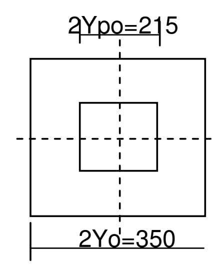
$f_{b,cal}$ = 1116.42/0.046225

= 24151.9 kN/m^2 < fb = 31255.81395348 O.K.

Now, 2Yo = 350

2Ypo = 215

Ypo/Yo = 0.61429



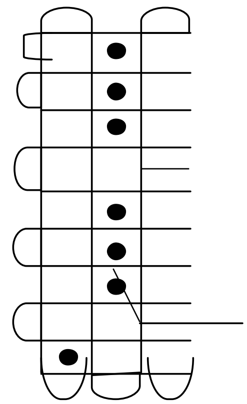
From table 8 of IRC:18-2000

$F_{bst}/P_k =$ 0.17

where F_{bst} = bursting force

P_k = ultimate cable force = 2204.4 kN

F_{bst} = 0.17 x 2204.4 = 374.748 kN



With HYSD bars, fy = 415 Mpa

and allowable stress = 0.87fy

= 361.05 Mpa

Ast required = 374.748 x 1000/361.05

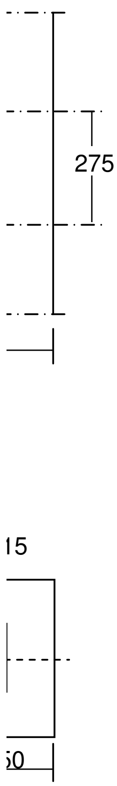
= 1037.94 mm^2

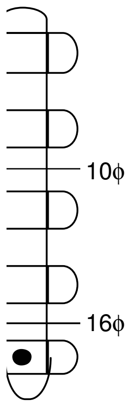
Provide

For web portion

10 ϕ - 4L - 4 nos. Ast,prod = 1256.6 mm^2 Vertically

16 ϕ - 2L - 4 nos. Ast,prod = 1608.448 mm^2 Horizontal


275
15
50



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CHAPTER:-6

COMPUTER AIDED DESIGN OF PRESTRESSED I GIRDER

6.1 INTRODUCTION

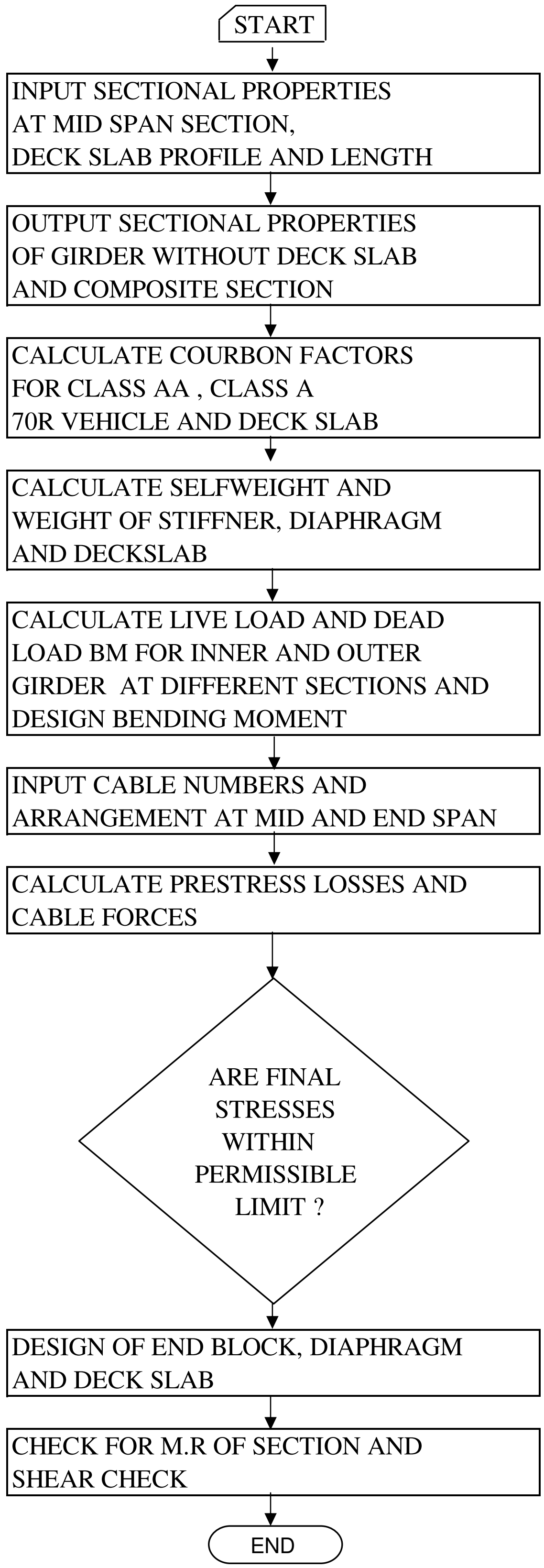
In a conventional approach to the design of prestressed concrete girders, a section of the girder and cable profile is required to be assumed. Thereafter, going through a series of typical and rigorous calculations, stress check is made and it is established whether the section and cable profile are suitable or not. If the design is not suitable, a revised section of girder and/or cable profile is tried. Since the process is repetitive, it can be better achieved by a computer software rather than repetitive hand calculation.

The use of this program is for the stress checking at various sections of the prestressed girder which forms part of a composite bridge deck comprising simply supported I girders and cast in-situ concrete slab. Cross-sectional dimensions of the girder and geometry of the choice of various types of strands, i.e. 7T13, 12T13, 19T13 etc. or any combination of these. The time dependent losses are considered as per IRC-18. At any specified section, checking of final stresses is possible.

The main features of the program are as follows:-

- Program prepared in MATLAB 7 software.
- All the provisions of IRC are taken care of.
- Fully descriptive output is generated.

6.2 FLOW CHART



6.3 ABOUT SOFTWARE

In this age of computers, most of the design processes are now becoming more and more automated with computers. Day by day new invention in field of computer makes use of computer with simple languages and simple application package development tools. The name MATLAB stands for matrix laboratory. The main feature of MATLAB is, it is a high-performance language for technical computing. It integrates computation, visualization, and programming in an easy-to-use environment where problems and solutions are expressed in familiar mathematical notation. Due to inbuilt mathematical function it is easy to programming in this software, compiler interprets line to line and because of this type of debugging facility it is easy for error handling. It coding is also easy. One can import and export data to EXCEL from MATLAB and vice verse.

6.4 PROGRAM STRUCTURE

The program includes following sub-modules:

- Sectional properties
- Input geometrical Data for carriageway
- Live load Analysis
- Dead load Analysis
- Cable profile
- Computation of Friction and Slip losses
- Computation of Final Losses
- Computation of Final Stresses
- Design of End Block, Diaphragm, Deck Slab
- Check for Moment resistance of section
- Check for Shear strength
- Output

Sectional properties

In this, any section will be specified and the cross sectional parameters of the girders will be given so that related cross sectional properties at specified sections can be carried out.

Input geometrical Data for carriageway and footpath

In this, geometrical data of the carriageway can be fed. So related properties can be carried out and can be used further for design.

Dead Load Analysis

Bending moments and shear forces at various sections due to dead load i.e. self-weight of girder, deck slab, diaphragms and SIDL are calculated in this. Variation in carriageway is taken care but nos. of kerbs, availability of footpath, crash barrier and parapet is taken as standard.

Live Load Analysis

Bending moments and shear forces at various sections due to live load i.e. Class AA, Class A and Class 70R are calculated in this. Automatically wheel of these IRC train placed on girder to obtain maximum bending moments and maximum shear forces at various sections.

Cable profile

With input of Cable arrangement at end span and at mid span, geometry of the cable in elevation is generated for all the cables. As all the cables are parabolic it takes care of the parabolic cable profile. Co-efficient of wobble friction, curvature friction and jacking stress are also specified. As horizontal deviation is neglected due to very less effect on cable forces from this, the ordinates of each cable in elevation are calculated at various sections.

Computation of friction and Slip Losses

In the calculation of cable forces, half the span of the girder is considered. Forces values at the predetermined intervals are first calculate considering the effect of friction and wobble only. It is presumed that the cable is stressed from both its ends (simultaneously and equally). During lock-off of strands into the anchorages, the jaws move an average distance (slip). On account of the loss of slip, the stress in the tendon at the anchorage drops from P to P_0 . a maximum slip of 6 mm is assumed. Due to reverse friction effects, the length of tendon affected by this loss is limited to some distance. This slip Δ is given by:-

$$\Delta = PL / AE_s$$

and the stress per unit length is given by ΔE_s

Stress after slip is calculated so that the area of slip diagram equals to 1170 kN/mm^2. The stresses after slip and friction loss are obtained at every section. The effective force in each cable, including horizontal and vertical components at predefined sections is calculated.

Computational of Final Losses

After calculation of the effective vertical force in the cables, stresses under the addition of various loads and the losses taken in Stage 1 and stage 2 are calculated at various sections in the following sequences.

- Loss due to elastic shortening of the 1^{st} stage cables. Stresses at the c.g. of 1^{st} stage cables are calculated. Elastic shortening is worked out considering this loss may very linearly from the maximum in the cable stresses first to zero in the cable stresses last. Hence, it may be taken as half of the maximum, later being the product of strain in concrete due to a weighted average compressive stress in concrete at the level of centroid of 1^{st} stage cables under the self weight of girder, initial prestress and modulus of elasticity of the cable steel.

- Loss due to relaxation of 1^{st} stage cables. Losses are calculated as per the cl. 11.4 (Table-4) of IRC-18-2000 based on the U.T.S after elastic shortening of cables.

- Loss due to creep and shrinkage between the age of concrete at 1st stage prestressing and 2nd prestressing. These losses are calculated as per the Table 2 and 3 of IRC-18-2000.

- Loss due to elastic shortening of the 2nd stage cables. Stresses at the c.g. of 2nd stage cables are calculated. Loss due to elastic shortening of 1st stage cables as well as 2nd stage cables is worked out.

- Loss due to relaxation of 2nd stage cables.

- Loss due to creep and shrinkage beyond the age of concrete at 2nd stage prestressing.

Computation of Final Stress

- After 1st stage prestressing
- After immediate loss
- After casting of diaphragm and deck slab
- After gradual loss-I (creep and shrinkage)
- After 2nd stage prestressing
- After immediate loss
- After SIDL
- After gradual loss-II (creep and shrinkage)
- At full live load condition

Design of End Block, Diaphragm and Deck slabs

Based on the size of the bearing plate, the bearing stress is calculated. Bursting force in the end block is calculated and for that force, the required reinforcement in the anchorage zone is provided. Checking of bearing stress behind anchorage is also carried out as per IRC-18 : 2000.

Design of deck slab is carried out for heaviest loading of IRC, Class 70R 40t bogi. For maximum +ve moment Class 70R 40t bogi placed at centre of span and for maximum –ve moment Class 70R 40t bogi placed at centre of support. Diaphragms are provided to strengthen the deck. Due to this no flexure of transverse deck is possible. According to Courbon's theory a concentric load, instead of pushing down only nearby girders, causes equal deflection of all the girders. It is designed as simple RCC unit.

Check for Moment resistance of section

As per the provisions made in IRC: 18, the PSC girder section is checked for the ultimate stress.

Check for Shear Strength

Check for the shear stress will be made at all specified section. Ultimate shear resistance of the section is calculated for uncracked and cracked conditions and the lesser of the two values is compared with the design shear force due to ultimate loads. If the shear force is greater the shear steel is provided. If it is less or shear steel is less than the minimum provision, minimum shear steel is provided.

CHAPTER:-7
SUMMARY AND CONCLUSIONS

7.1 SUMMARY

Prestressed concrete is now a days extensively used in the construction of large span bridge structure. The analysis and design of a prestressed concrete member is not only a tedious task but voluminous too. Its finalization requires repeated trial and error, which requires much time and may lead to mistakes in manual calculations. There are many general purpose packages available for analysis and design of structures, but very few are in the field of design of prestressed concrete structures. Computer Aided Design applications for such types of work will result in a considerable saving in time, in addition to the refinement and economy. With this aim, a computer program was developed in Excel and MATLAB 7.0 which design the girder of a composite bridge deck-girder system and its various components using IRC recommendation. The program is prepared for moving loads for the various IRC loadings. It also finds live loads sharing for moving loads. After giving number of cables, cable profile, losses and the final stresses are checked. As per IRC the shear and M.R check is also provided for the girder design.

7.2 CONCLUSIONS

- Analysis and design of 3-lane, four girder and 40m span bridge superstructure is carried out in this dissertation.

- Courbon's method is great tool in finding out loading on girders which otherwise require plate analysis. When all the girders have same moment of inertia the calculations are simplified a lot. Program finds reaction factors by this method for load sharing between internal and external longitudinal girders.

- This program makes whole analysis and design procedure of bridge superstructure components in a simple way as per IRC provisions and thereby reduces lots of labor of a design engineer.

- The moving load analysis of IRC live loads as incorporated in the program approximates the behavior of imposed live loads much to the perfection.

- Position of live loads near crash barrier induces maximum forces in the outer girder and thus is the worst condition for which the girders are designed.

- The tabular output of moments gives an idea about contributions of dead load, live load and superimposed dead load the check for stresses, is verified for maximum dead load and live load moment at mid span. As there are no tensile stresses, it is a Type 1 design.

- The prestress force and its eccentricity is the most critical part of the girder design and even a slight variation in eccentricity produces big variation in stresses. A compromise between stress requirements and prestressing force compatible with the assumed dimensions of girder is truly a trial and error process requiring engineering ingenuity. The program greatly aids in this process and saves time. Here number of cables is eight and having parabolic cable profile which satisfies all the limit state checks as per IRC rules, therefore it is a safe design.

7.3 FUTURE SCOPE OF WORK

- Rigorous analysis of temperature loss can be included.

- Program further can be extended for more than 3 lane bridges and analysis and design of substructure.

References:-

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The Indian Road Congress

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5. IS:6006- 1983, Indian standard code of Specification for uncoated
stress relieved strand for Prestressed concrete

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Prentice-Hall of India Private Limited, New Delhi.

APPENDIX:-A

STAAD.Pro INPUT FILE FOR ANALYSIS OF DIAPHRAGM AND DECK SLAB

```
STAAD PLANE
START JOB INFORMATION
ENGINEER DATE 16-Jun-05
END JOB INFORMATION
INPUT WIDTH 79
UNIT METER KN
JOINT COORDINATES
1 0 0 0; 2 3 0 0; 3 6 0 0; 4 9 0 0;
MEMBER INCIDENCES
1 1 2; 2 2 3; 3 3 4;
DEFINE MATERIAL START
ISOTROPIC CONCRETE
E 2.17185e+007
POISSON 0.17
DENSITY 23.5616
ALPHA 1e-005
DAMP 0.05
END DEFINE MATERIAL
CONSTANTS
MATERIAL CONCRETE MEMB 1 TO 3
MEMBER PROPERTY AMERICAN
1 TO 3 PRIS YD 0.24 ZD 1
SUPPORTS
LOAD 1 CASE 1
MEMBER LOAD
1 CON GY -90.22 2.035 0.5
2 CON GY -90.22 0.965 0.5
LOAD 2 CASE 2
MEMBER LOAD
1 CON GY -83.96 1.5 0.5
2 CON GY -83.96 0.43 0.5
LOAD 3 SW
MEMBER LOAD
1 TO 3 UNI GY -7.76 0 3 0.5
LOAD 9 SWG
MEMBER LOAD
1 TO 3 UNI GY -29.6 0 3 0.5
LOAD 10 CG2
MEMBER LOAD
1 CON GY -391.18 1.975 0.5
2 CON GY -391.18 1.025 0.5
LOAD 11 CG3
MEMBER LOAD
```

```
1 CON GY -381.18 1.5 0.5
LOAD 12 CG4
MEMBER LOAD
1 CON GY -381.25 0 0.5
1 CON GY -381.25 2.05 0.5
PERFORM ANALYSIS PRINT ALL
PERFORM ANALYSIS PRINT ALL
PERFORM ANALYSIS PRINT ALL
FINISH
```

APPENDIX:-B

DRAWINGS WITH DETAILING

List of Drawings

1/1 GENERAL ARRANGEMENT DRAWING OF PSC I TYPE SUPERSTRUCTURE

2/3 DETAILS OF PRESTRESSING CABLES FOR I GIRDER

3/3 REINFORCEMENT DETAIL OF MAIN GIRDER, CROSS GIRDER, DECK SLAB & END BLOCK FOR I GIRDER

APPENDIX:-C

MATLAB 7.0 PROGRAM

```
% SECTIONAL PROPERTIES

%SECTIONAL PROPERTY CALCULATION OF I-GIRDER AT MID SPAN

% properties input

clear all;
w1(1,1)=input('Enter width of part 1 ');
d1(1,1)=input('Enter depth of part 1 ');

w2(1,1)=input('Enter width of part 2 ');
d2(1,1)=input('Enter depth of part 2 ');

w3(1,1)=input('Enter width of part 3 ');
d3(1,1)=input('Enter depth of part 3 ');

w4(1,1)=input('Enter width of part 4 ');
d4(1,1)=input('Enter depth of part 4 ');

w5(1,1)=input('Enter width of part 5 ');
d5(1,1)=input('Enter depth of part 5 ');

% Area calculation
a1(1,1)=w1(1,1)*d1(1,1);
a2(1,1)=w2(1,1)*d2(1,1);
a3(1,1)=w3(1,1)*d3(1,1);
a4(1,1)=w4(1,1)*d4(1,1);
a5(1,1)=w5(1,1)*d5(1,1);

% Total area

a(1,1)=a1(1,1)+a2(1,1)+a3(1,1)+a4(1,1)+a5(1,1);

% CG from top

cg1(1,1)=d1(1,1)/2;
cg2(1,1)=d1(1,1)+(d2(1,1)/2);
cg3(1,1)=d1(1,1)+d2(1,1)+(d3(1,1)/2);
cg4(1,1)=d1(1,1)+(d4(1,1)/3);
cg5(1,1)=d1(1,1)+d2(1,1)-(d5(1,1)/3);
```