ANALYSIS AND DESIGN OF PRESTRESSED

I-GIRDER

Submitted in partial fulfillment of the requirements

for the degree of

Bachelor of Engineering

By

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CERTIFICATE

This is to certify that the project entitled **"***Analysis and Design of Prestressed I-Girder***"** is a bonafide work of *Bhat Akashdeep Aravind (11CE12), Bincy Babu (11CE03), Shaikh Mohd. Tazir Asif (11CE45) and Siddique Tanveer Irshad (11CE55)* submitted to the University of Mumbai in partial fulfilment of the requirement for the award of the degree of "Bachelor of Engineering" in Department of Civil Engineering.

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Project Report Approval for B. E.

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Declaration

We declare that this written submission represents our ideas in our own words and where others ideas or words have been included, we have adequately cited and referenced the original sources. We also declare that we have adhered to all principles of academic honesty and integrity and have not misrepresented or fabricated or falsified any idea/data/fact/source in our submission. We understand that any violation of the above will be cause for disciplinary action by the Institute and can also evoke penal action from the sources which have thus not been properly cited or from whom proper permission has not been taken when needed.

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Abstract

Post-tensioned simply supported pre-stressed concrete (PC) I-girder bridges are widely used bridge system for short to medium span (20m to 50m) highway bridges, due to its moderate self -weight, structural efficiency, ease of fabrication, low maintenance etc. This study is on design and analysis of prestressed I- girder. Design constraints were decided using IRC 6:2010 for loading, IRC 18:2000 for minimum dimension requirement, IRC 21:2000 for concrete stresses. To formulate the entire problem for a couple of span under class A and class 70R loading, obtain shear force and bending moment at regular intervals along the beam. The software STAAD PRO is used for the analysis and design of prestressed concrete girders.

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f_c	Prestress in concrete at the level of steel.
E_s	Modulus of elasticity of steel.
E_c	Modulus of elasticity of concrete.
α_e	Modular ratio.
ε_{cs}	Total residual shrinkage strain
ε_c	Ultimate creep strain for a sustained unit stress
Φ	creep coefficient
ε_e	Elastic strain
μ	Coefficient of friction between cable and duct

Chapter 1

Introduction

1.1 Background

Prestress concrete is ideally suited for the construction of medium and long span bridges. Ever since the development of prestressed concrete by Freyssinet in the early 1930s, the material has found extensive application in the construction of long-span bridges, gradually replacing steel which needs costly maintenance due to the inherent disadvantage of corrosion under aggressive environment conditions. One of the most commonly used forms of superstructure in concrete bridges is precast girders with cast-in-situ slab. This type of superstructure is generally used for spans between 20 to 40 m. T or I-girder bridges are the most common example under this category and are very popular because of their simple geometry, low fabrication cost, easy erection or casting and smaller dead loads. An I-beam girder is described by having the cross section of the girder taking the shape of the capital letter I. The vertical plate in the middle is known as the web, and the top and bottom plates are referred to as flanges.

1.2 Aim and Objective

1. Analysis of loads and stresses acting on Prestressed I-Girder in accordance with IRC 6:2010.
2. Design of the Prestressed I-Girder as per IRC 18:2000.
3. Determination of Losses as per IRC 1343:2000.
4. To prepare a model of I-Girder and analyse it in Staad pro software.

1.3 I-Girder

A girder bridge, in general, is a bridge that utilizes girders as the means of supporting the deck. A bridge consists of three parts: the foundation (abutments and piers), the superstructure (girder, truss, or arch), and the deck. A girder bridge is very likely the most commonly built and utilized bridge in the world. Its basic design, in the most simplified form, can be compared to a log ranging from one side to the other across a river or creek. The term "girder" is often used interchangeably with "beam" in reference to bridge design. However, some authors define beam bridges slightly differently from girder bridges.

When a beam bends the top of the beam is in compression and the bottom is in tension. These forces are greatest at the very top and the very bottom. So to make the stiffest beam with the least amount of material you would want the material to be only at the top and the bottom sides. However, you still need to connect them together or they would just be two separate plates and would not be stiff at all. So you put a web in the middle to connect them and make them work together. The resulting shape is the traditional “I-beam” or a wide flange beam. The I-shape is an ideal shape for beams that is for resisting flexure. It’s an extremely efficient shape for resisting bending; which is another way of saying it has a lot of strength for a small amount of material and expense. The shape of an I-Girder is shown in Fig: 1.1

Fig 1.1: I-Girder

1.4 Prestressed Concrete

Prestressed concrete is basically concrete in which internal stresses of a suitable magnitude and distribution are introduced so that the stresses resulting from external loads are counteracted to desired degree. In reinforced concrete members, the prestress is commonly introduced by tensioning the steel reinforcement.

The earliest example of wooden barrel construction by free force-fitting of metal bands and shrink-fitting of metal tyres on wooden wheels indicate that the art of prestressing has been practised from ancient times. The tensile strength of plain concrete is only a fraction of its compressive strength and the problem of it being deficient in tensile strength appears to have been the driving factor in the development of the composite material known as "reinforced concrete".

The development of early cracks in reinforced concrete due to incompatibility in the strains of steel and concrete was perhaps the starting point in the development of the a new material like "Prestressed concrete". The application of permanent compressive stress to a material like concrete, which is strong in compression but weak in tension, increases the apparent tensile strength of that material, because the subsequent application of tensile stress must first nullify the compressive prestress. In 1904, Freyssinet attempted to introduce permanently acting forces in concrete to resist the elastic forces developed under loads and this idea was later developed under the name of "prestressing".

1.5 Need for High Strength Steel and Concrete

The significant observations which resulted from the pioneering research on prestressed concrete were:

1. Necessity of using high-strength steel and concrete.
2. Recognition of losses of prestress due to various causes.

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 apparent 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 2000N/mm^2. These high stress ranges are possible only with the use of high-strength steel.

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 invariably preferred to minimise costs. High strength concrete is less liable to shrinkage cracks, and has a higher modulus of elasticity and smaller ultimate creep strength, resulting in a smaller loss of prestress in steel. The use of high-strength concrete results in a reduction in the cross-sectional dimensions of prestressed structural elements. With a reduced dead-weight of the material, longer spans become technically and economically practicable.

1.6 Advantages of Prestressed Concrete.

Prestressed concrete offers great technical advantages in comparison with other forms of construction, such as reinforced concrete and steel. Prestressed concrete members possess improved resistance to shearing forces, due to the effect of compressive prestress, which reduces the principal tensile stress. The use of curved cables, particularly in long-span members, helps to reduce the shear forces developed at the support sections.

The use of high-strength concrete and steel in prestressed members results in lighter and slender members than is possible with reinforced concrete. The two structural features of prestressed concrete, namely high-strength concrete and freedom from cracks, contribute to the improved durability of the structure and the aggressive environmental conditions. Prestressing of concrete improves the ability of the material for energy absorption under impact loads. The ability to resist repeated working loads has been proved to be as good in prestressed as in reinforced concrete.

1.7 Application of Prestressing

Prestressed concrete is the main material for floors in high-rise buildings and the entire containment vessels of nuclear reactors.

Unbonded post-tensioning tendons are commonly used in parking garages as barrier cable.

Also, due to its ability to be stressed and then de-stressed, it can be used to temporarily repair a damaged building by holding up a damaged wall or floor until permanent repairs can be made.

The advantages of prestressed concrete include crack control and lower construction costs; thinner slabs—especially important in high rise buildings in which floor thickness savings can translate into additional floors for the same (or lower) cost and fewer joints, since the distance that can be spanned by post-tensioned slabs exceeds that of reinforced constructions with the same thickness. Increasing span lengths increases the usable unencumbered floor space in buildings; diminishing the number of joints leads to lower maintenance costs over the design life of a building, since joints are the major focus of weakness in concrete buildings.

The first prestressed concrete bridge in North America was the Walnut Lane Memorial Bridge in Philadelphia, Pennsylvania. It was completed and opened to traffic in 1951. Prestressing can also be accomplished on circular concrete pipes used for water transmission. High tensile strength steel wire is helically-wrapped around the outside of the pipe under controlled tension and spacing which induces a circumferential compressive stress in the core concrete. This enables the pipe to handle high internal pressures and the effects of external earth and traffic loads. The world’s tallest prestressed concrete C.N tower, Toronto, 553 m overall height is shown in Fig: 1.2

Fig 1.2: CN Tower, Old Toronto, Canada

1.8 Materials for Prestressed Concrete.

1.8.1 High-Strength Concrete Mixes

Prestressed concrete requires concrete which has a high compressive strength at a reasonably early age, with comparatively higher tensile strength than ordinary concrete. Low shrinkage, minimum creep characteristic and a high value of Young's modulus are generally deemed necessary for concrete used for prestressed members. Many desirable properties, such as durability, impermeability and abrasion resistance, are highly influenced by the strength of concrete. With the development of vibration techniques in 1930, it became possible to produce, without much difficulty, high-strength concrete having 28-day cube compressive strength in the range of 30-70 N/mm^2. Recent developments in the field of concrete-mix design have indicated that it is now possible to produce even ultra-high strength concrete, of any desired 28-day cube compressive strength ranging from 70 to 100 N/mm^2, without taking recourse to unusual materials or processing and without facing any significant technical difficulties,

1.9 Methods of Prestressing

1.9.1 Pretensioning

In pretensioned members, the tendons are tensioned even before casting the concrete. One end of the reinforcement is secured to an abutment while the other end of the reinforcement is pulled by using a jack and this end is then fixed to another abutment. The concrete is then poured. After the concrete has cured and hardened the ends of the reinforcement are released from the abutment. The reinforcement which tends to resume its original length will compress the concrete surrounding it by bond action. The prestress is thus transmitted to concrete entirely by the action of bond between reinforcement and surrounding concrete. A typical pretensioning bed is as shown in the Fig 1.3

Fig 1.3: Pretensioning Bed

1.8.2 Post-tensioning

A post-tensioned member is one in which the reinforcement is tensioned after the concrete has fully hardened. The beam is first cast leaving ducts for placing the tendons. When concrete has hardened and developed its strength. The tendon is passed through the duct. One end is provided with an anchor and is fixed to the one end of the member. Now other end of the tendon is pulled by a jack that is butting against the end of the member. The jack simultaneously pulls the tendon and compresses the concrete. After the tendon is subjected to the desired stresses the end of the tendon is also properly anchored to the concrete. The stages in post-tensioning are as shown in Fig 1.3

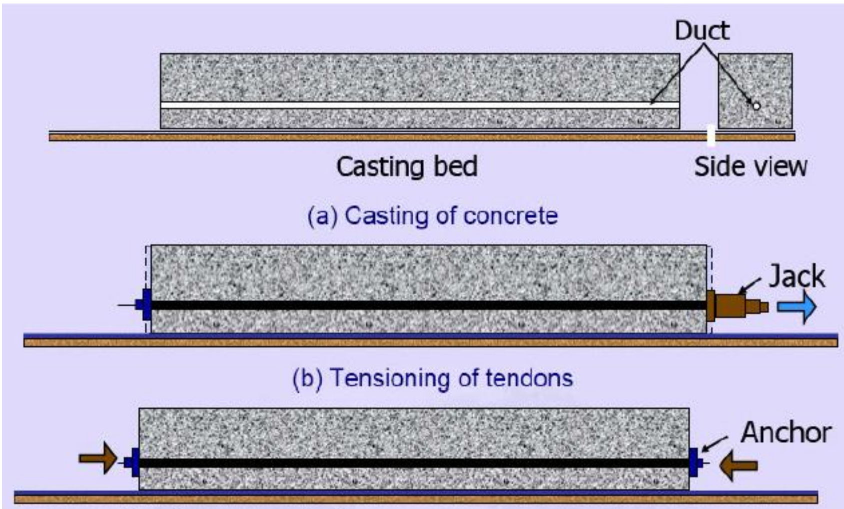


Fig 1.4: Stages in Post-tensioning

1.8.3 Pretensioned member Vs Post-tensioned member

Table 1.1: Pretensioned member Vs Post-tensioned member

Pretensioned member	**Post-tensioned member**
1. In pretensioned prestressed concrete, steel is tensioned prior to that of concrete. It is released once the concrete is placed and hardened. The stresses are transferred all along the wire by means of bond.	1. Concreting is done first then wires are tensioned and anchored at ends. The stress transfer is by end bearing not by bond.
2. Suitable for short span and precast products like sleepers, electric poles on mass production.	2. Suitable for long span bridges
3. In pretensioning the cables are basically straight and horizontal. Placing them in curved or inclined position is difficult. However the wires can be kept with eccentrically. Since cables cannot be aligned similar to B.M.D. structural advantages are less compare to that of post-tensioned.	3. The post tensioning cables can be aligned in any manner to suit the B.M.D due to external load system. Therefore it is more economical particularly for long span bridges. The curved or inclined cables can have vertical component at ends. These components will reduce the design shear force. Hence post-tensioned beams are superior to pretensioned beams both from flexural and shear resistances point.
4. Prestress losses are more as compared to that of post-tensioned concrete.	4. Losses are less compare to pre-tensioned concrete

1.10 System of Prestressing

1. Freyssinet system
2. Magnel Blaton system
3. Gifford-Udall system
4. Lee-McCall system

1.10.1 Freyssinet System

Freyssinet system was introduced by the French Engineer Freyssinet and it was the first method to be introduced. High strength steel wires of 5mm or 7mm diameter, numbering 8 or 12 or 16 or 24 are grouped into a cable with a helical spring inside. Spring keeps proper spacing for the wire. Cable is inserted in the duct. Anchorage device consists of a concrete cylinder with a concentric conical hole and corrugations on its surface, and a conical plug carrying grooves on its surface (Fig. 3). Steel wires are carried along these grooves at the ends. Concrete cylinder is heavily reinforced. Members are fabricated with the cylinder placed in position. Wires are pulled by Freyssinet double acting jacks which can pull through the suitable grooves all the wires in the cable at a time. One end of the wires is anchored and the other end is pulled till the wires are stretched to the required length. An inner piston in the jack then pushes the plug into the cylinder to grip the wires.

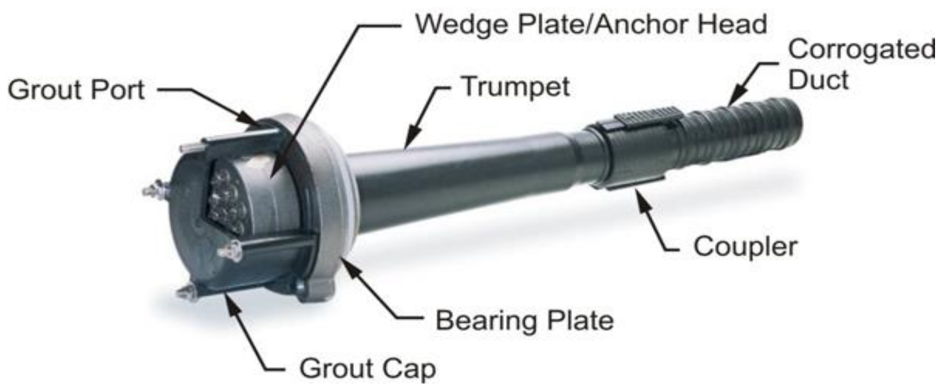


Fig 1.5: Freyssinet System

1.10.2 Magnel Blaton system

In Freyssinet system several wires are stretched at a time. In Magnel Blaton system, two wires are stretched at a time. This method was introduced by a famous engineer, Prof. Magnel of Belgium. In this system, the anchorage device consists of sandwich plate having grooves to hold the wires and wedges which are also grooved. Each plate carries eight wires. Between the two ends the spacing of the wires is maintained by spacers. Wires of 5mm or 7mm are adopted. Cables consist of wires in multiples of 8 wires. Cables with as much as 64 wires are also used under special conditions. A specially devised jack pulls two wires at a time and anchors them.

1.10.3 Gifford Udall System

This system originated in Great Britain, is widely used in India. This is a single wire system. Each wire is stressed independently using a double acting jack. Any number of wires can be grouped together to form a cable in this system. There are two types of anchorage device in this

a) Tube anchorages

b) Plate anchorages

Tube anchorage consists of a bearing plate, anchor wedges and anchor grips. Anchor plate may be square or circular and have 8 or 12 tapered holes to accommodate the individual prestressing wires. These wires are locked into the tapered holes by means of anchor wedges. In addition, grout entry hole is also provided in the bearing plate for grouting. Anchor wedges are split cone wedges carrying serrations on its flat surface. There is a tube unit which is a fabricated steel component incorporating a thrust plate, a steel tube with a surrounding helix. This unit is attached to the end shutters and forms an efficient cast-in component of the anchorage.

1.10.4 Lee McCall System

This method is used to prestress steel bars. The diameter of the bar is between 12 and 28mm. bars provided with threads at the ends are inserted in the performed ducts. After stretching the bars to the required length, they are tightened using nuts against bearing plates provided at the end sections of the member.

1.10.5 Other Methods of Prestressing:

1.10.5.1 Electrical Prestressing:

In this method, reinforcing bars is coated with thermoplastic material such as sulphur or low melting alloy and buried in the concrete. After the concrete is set, electric current of low voltage but high amperage is passed through the bar. Electric current heats the bar and the bar elongates. Bars provided with threads at the other end are tightened against heavy washers, after required elongation is obtained. When the bar cools, prestress develops and the bond is restored by re-solidification of the coating.

1.10.5.2 Chemical Prestressing:

Chemical prestressing is done using expanding cement. Prestressing can be applied embedding steel in concrete made of expanding cement. Steel is elongated by the expansion of the concrete and thus gets prestressed. Steel in turn produces compressive stress in concrete.

Chapter 2

Losses of Prestress

The initial prestress in concrete undergoes a gradual reduction with time from the stage of transfer due to various causes. This is generally referred to as loss of prestress. The following are the different types of losses encountered.

2.1 Losses in Pretensioning

1. Elastic deformation of concrete
2. Relaxation of stress in steel
3. Shrinkage of concrete
4. Creep of concrete

2.2 Losses in Post-tensioning

1. No loss due to elastic deformation if all wires are simultaneously tensioned. If the wires are successively tensioned, there will be loss of prestress due to elastic deformation of concrete.
2. Relaxation of stress in steel
3. Shrinkage of concrete
4. Creep of concrete
5. Friction
6. Anchorage slip

2.3 Loss due to elastic deformation of concrete

When the prestress is transmitted to the concrete member, there is contraction due to prestress. This contraction causes a loss of stretch in the wire. When some of the stretch is lost, prestress gets reduced. This comes under immediate loss. In pretensioned concrete, when the prestress is transferred to the concrete the member shortens and prestressing steel also shortens in it. Hence there is a loss of prestress. In case of post-tensioning if all the cables are tensioned simultaneously there is no loss since the applied stress is recorded after the elastic shortening has completely occurred. If the cables are tensioned sequentially there is loss in the tendon during subsequent stretching of other tendons. The loss of prestress due to deformation of concrete depends on the modular ratio & the average stress in concrete at the level of steel.

Strain in concrete at level of steel= f_c/E_c

Stress in steel corresponding to this strain= $f_c\, E_s/E_c$

Therefore, loss of stress in steel= $\alpha_e\, f_c$

2.4 Loss due to shrinkage of concrete

Shrinkage of concrete is defined as the contraction due to loss of moisture. Due to the shrinkage of concrete, the prestress in the tendon is reduced with time. Curing the concrete adequately and delaying the application of load provide long term benefits with regards to durability and loss of prestress. In special situations detailed calculations may be necessary to monitor shrinkage strain with time. Specialised literature or international codes can provide guidelines for such calculations.

Factors affecting the shrinkage in concrete

1. The loss due to shrinkage of concrete results in shortening of tensioned wires & hence contributes to the loss of stress.
2. The shrinkage of concrete is influenced by the type of cement, aggregate & the method of curing used.
3. Use of high strength concrete with low water cement ratio results in reduction in shrinkage and consequent loss of prestress.
4. The primary cause of drying shrinkage is the progressive loss of water from concrete.
5. The rate of shrinkage is higher at the surface of the member.

6. The differential shrinkage between the interior surfaces of large member may result in strain gradients leading to surface cracking. Hence, proper curing is essential to prevent cracks due to shrinkage in prestress members. In the case of pretensioned members, generally moist curing is restored in order to prevent shrinkage until the time of transfer. Consequently, the total residual shrinkage strain will be larger in pretensioned members after transfer of prestress in comparison with post-tensioned members, where a portion of shrinkage will have already taken place by the time of transfer of stress. This aspect has been considered in the recommendation made by the code (IS:1343) for the loss of prestress due to shrinkage of concrete and is obtained below:

   If,

   ε_{cs}= Total residual shrinkage strain= 300×10^{-6} for Pretensioning and

   = $[200 \times 10^{-6}/\log_{10}(t+2)]$ for Post-tensioning

   t= Age of concrete at transfer in days.

   Then, the loss of stress= $\varepsilon_{cs} E_s$

2.5 Loss due to relaxation of stress in steel

Most of the codes provide for the loss of stress due to relaxation of steel as a percentage of initial stress in steel. The Indian standard code recommends a value varying from 0 to 90 N/mm^2 for stress in wires varying from 0.5 f_{pu} to 0.8 f_{pu}

Where, f_{pu}= characteristic strength of prestressing tendon.

Table 2.1: Relaxation losses for prestressing losses at 1000h at 27±2° C (IS: 1343)

Sr. No	Initial Stresses	Relaxation Loss (%)	
		Normal	Low
1	0.7 f_{pu}	0	0
2	0.6 f_{pu}	0.3	1.0
3	0.7 f_{pu}	5.0	2.5
4	0.8 f_{pu}	8.0	4.5

2.6 Loss due to creep

Creep of concrete is defined as the increase in deformation with time under constant load. Creep is time dependent. Due to the creep of concrete, the prestress in the tendon is reduced with time. For stresses in concrete less than one third of the characteristic strength the ultimate strain is found to be proportional to elastic strain. The ratio of the ultimate creep strain to the elastic strain is defined as the ultimate creep coefficient or simply creep coefficient Φ.

The following considerations are applicable for calculating the loss of prestress due to creep.

1) The creep is due to the sustained (permanently applied) loads only. Temporary loads are not considered in the calculation of creep.

2) Since the prestress may vary along the length of the member, an average value of the prestress can be considered.

3) The prestress changes due to creep and the creep is related to the instantaneous prestress. To consider this interaction, the calculation of creep can be iterated over small time steps.

Formula:

Ultimate creep strain

The loss of stress due to creep of concrete = $\varepsilon_c f_c E_s$

Creep coefficient method

Creep coefficient = (creep strain/ elastic strain)

$\Phi = \varepsilon_c / \varepsilon_e$

$\varepsilon_c = \Phi \varepsilon_e = \Phi \varepsilon_c E_c E_s = \Phi f_c \alpha_e$

2.7 Loss due to anchorage slip

In most post tensioning system when the tendon force is transferred from the jack to the anchoring ends, the friction wedges slip over a small distance. Anchorage blocks also moves before it settles on concrete. Loss of prestress is due to the consequent reduction in the length of the tendon. Certain quantity of prestress is released due to this slip of wire through the anchorages .The amount of slip depends on type of wedge and stress in the wire.

Anchorage loss can be accounted for at the site by overextending the tendon during prestressing operation by the amount of draw- in before anchorage

Δ= slip anchorage in mm

L=Length of the cable, in mm

A=Cross-sectional area of the cable in mm^2

E_s =Modulus of elasticity of steel in N/mm^2

P=Prestressing force in the cable, in N

Then,

$(PL/AE)=\Delta$

Hence, Loss of stress due to anchorage slip = $P/A= E_s \Delta/L$

2.8 Loss due to friction

Frictional loss occurs only in post tensioned beams. When the cable is stressed, friction between the sides of the duct and the cable does not permit full tension to be transmitted. Therefore at a point away from the jacking end prestress is less.

In post tensioned members, tensions are housed in ducts or sheaths. If the profile of cable is linear, loss will be due to straightening or stretching of cables called wobble effect .If the profile is curved, there will be loss in stress due to friction between tendon and the duct or between the tendons themselves \.

$P_x = P_0 e^{-(\mu\alpha +kx)}$

Where,

P_0= The Prestressing force at the jacking end.

μ = Coefficient of friction between cable and duct

α= the cumulative angle in radians through the tangent to the cable profile has turned between any two points under consideration

k = Friction coefficient for wave effect

$e=2.7183$

The IS code recommends the following value for k

k = 0.15 per 100 m for normal condition

k = 1.5 per 100 m for thin walled ducts where heavy vibration are encountered and in other adverse conditions.

Values for coefficient of friction μ

0.55 for steel moving on smooth concrete

0.35 for steel moving on steel fixed to duct

0.25 for steel moving on steel fixed to concrete

0.25 for steel moving on lead

0.18-0.30 for multilayer wire rope cables in rigid rectangular steel sheaths

0.15-0.25 for multilayer wire rope cables with spacer plates providing lateral separation.

Chapter 3

Loads and Stresses

3.1 Types of loads and stresses

The following are the loads and stresses to be considered in the design of the prestressed I-Girder.

1. Dead load
2. Live load
3. Impact or dynamic effect of live load
4. Temperature stresses

3.1.1 Dead load

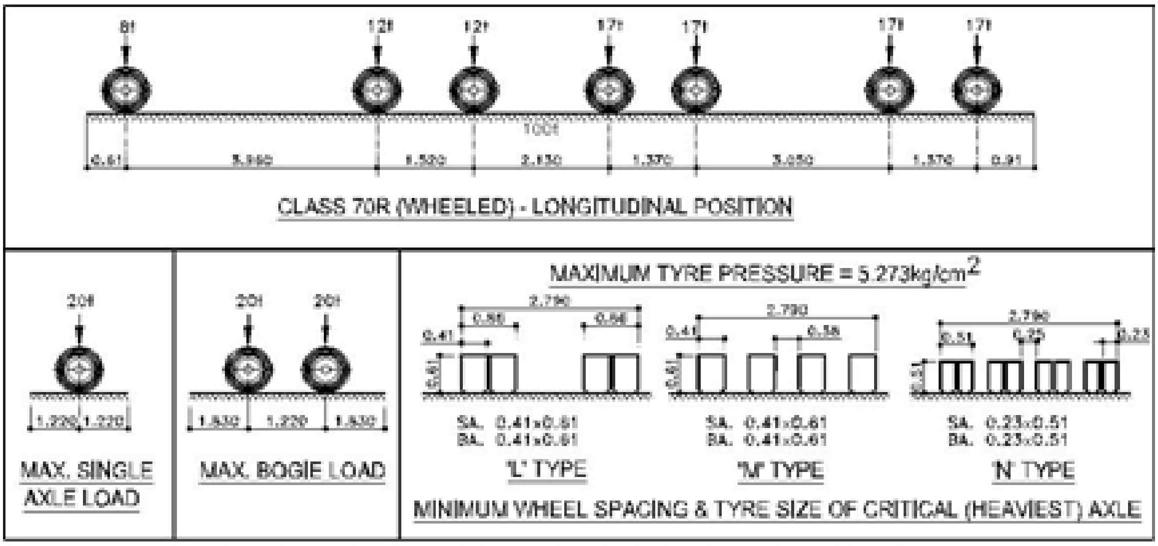
The dead load carried by a girder or member shall consist of the portion of the weight of the superstructure (and the fixed load carried thereon) which is supported wholly or in part by the girder or member including its own weight.

3.1.2 Live load

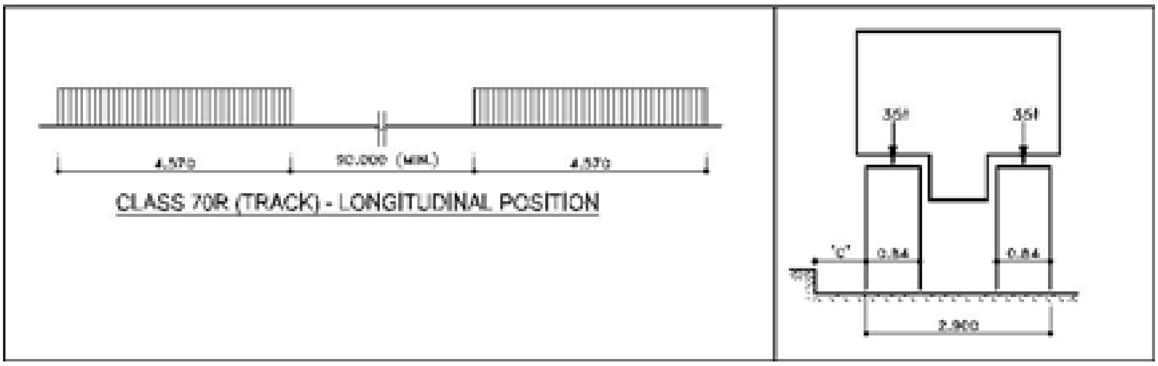
- Class AA Loading: This loading is to be adopted within certain municipal limits, in certain existing or contemplated industrial areas, in other specified areas, and along certain specified highways. Bridges designed for Class AA loading should be checked for Class A loading also, as under certain conditions, heavier stresses may be obtained under Class A loading.

- Class A Loading: This loading is to be normally adopted on all roads on which permanent bridges or culverts are to be constructed.
- Class B Loading: This loading is to be normally adopted for temporary structures and for bridges in specified areas. Structures with timber span are to be regarded as temporary structures for the purpose of this class.

3.2 Details of I.R.C Loading



WHEEL ARRANGEMENT FOR 70R (WHEELED VEHICLE)



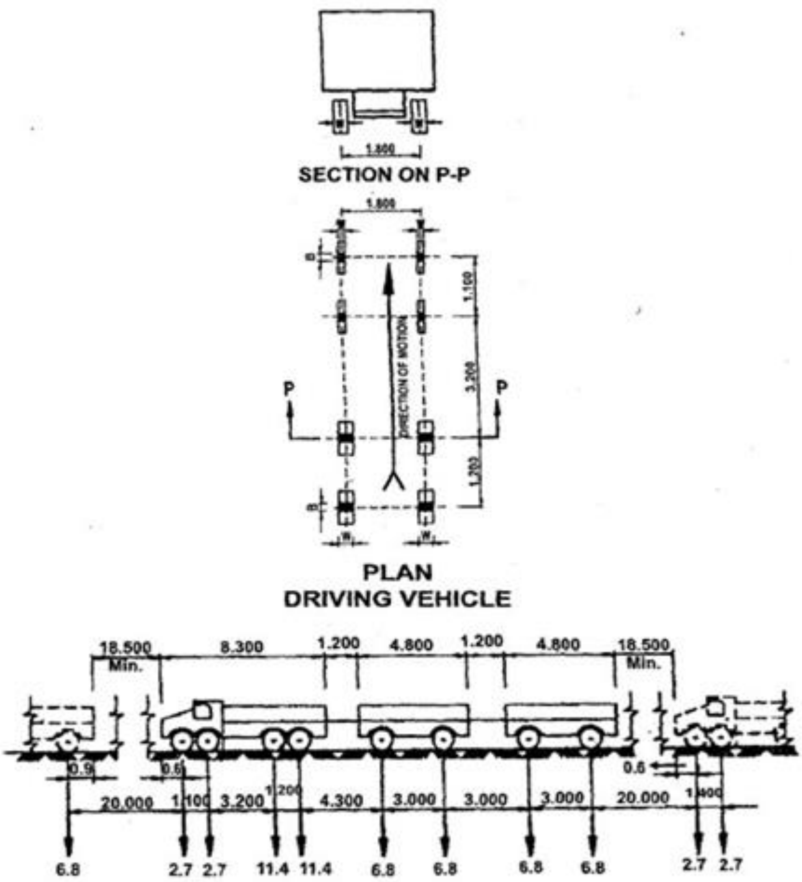
Wheel arrangement for Class 70R (Tracked vehicle)

Fig 3.1: Class 70R Tracked & Wheeled Vehicles

Notes under Fig 3.1

1) The nose to tail spacing between two successive vehicles shall not be less than 90m for tracked vehicle and 30m for wheeled vehicle.

2) For multi-lane bridges and culverts, each Class 70R loading shall be considered to occupy two lanes and no other vehicle shall be allowed in these two lanes. The passing/crossing vehicle can only be allowed on lanes other than these two lanes. Load combination is as shown in Table 3.1
3) The maximum loads for the wheeled vehicle shall be 20 tonne for a single axle or 40 tonne for a bogie of two axles spaced not more than 1.22m centres.
4) Class 70R loading is applicable only for bridges having carriageway width of 5.3m and above (i.e. 1.2 x 2 + 2.9 = 5.3). The minimum clearance between the road face of the kerb and the outer edge of the wheel or track, 'C', shall be 1.2m.
5) The minimum clearance between the outer edge of wheel or track of passing or crossing vehicles for multilane bridge shall be 1.2m. Vehicles passing or crossing can be either same class or different class, Tracked or Wheeled.
6) Axle load in tonnes, linear dimension in meters.



Class 'A' Train of Vehicle

Fig 3.2: Class 'A' Train of Vehicles

NOTES UNDER FIG 3.2

1) The nose to tail distance between successive trains shall not be less than 18.5m.
2) For single lane bridges having carriageway width less than 5.3m, one lane of Class A shall be considered to occupy 2.3m. Remaining width of carriageway shall be loaded with 500 Kg/m^2, as shown in Table 3.1
3) For multi-lane bridges each Class A loading shall be considered to occupy single lane for design purpose. Live load combinations as shown in Table 3.1 shall be followed.
4) Axle loads in tonne. Linear dimensions in metre.

3.3 Live Load Combinations

The carriageway live load combinations shall be considered for the design as shown below in Table: 3.1.

Table3.1: Live load combination

Sr.No.	Carriageway Width.	Number of lanes for design purpose.	Load combination.
1	Less than 5.3m	1	One lane of Class A considered to occupy 2.3m .The remaining width of carriageway shall be loaded with 500KGg/m^2.
2	5.3m to 9.6m	2	One lane of Class 70R or two lane of Class A.
3	9.6m to 13.1m	3	One lane of class 70R with one lane of Class A or three lanes of Class A
4	13.1m to 16.6m	4	One lane of Class 70R for every two lanes with one lane of Class A for the remaining lanes, if any, or one lane of Class A of each lane.
5	16.6m to 20.1m	5	
6	20.1m to 23.6m	6	

Chapter 4

Analysis and Design of Prestressed I-Girder

4.1 Brief description of the proposal

4.1.1 Concrete

Controlled concrete of grade M45 giving cube strength of 45N/mm^2 on 150mm cubes tested at 28 days is proposed for PSC girders, diaphragms and RCC deck slab.

4.1.2 High Tensile Steel

High tensile steel shall be the standard coated, stress relived confirming to IS: 6006-class II having a minimum ultimate tensile strength of 18.74t per strand. The wires shall be stress relived plain, cold drawn wires confirming to IS: 1785.

4.1.3 Design Aspect

In accordance with the sequence of construction and depending on the imposition of loads, forces and moments are encountered. Beams sections are checked for bending moment and shear force effects at various sections.

Design of PSC girder is in conformity with the IRC code of practice for plain reinforced and prestressed concrete. The end block design is in conformity with IS: 1343 code of practice for design of prestressed concrete structures and IRC: 18. Deflection of the PSC girder is checked accounting for all long term and short term losses in prestress.

4.2 Design data

For the design purpose following preliminary data was selected based on IRC recommendations.

The length of the girder is 21.1m and the centre to centre distance between the bearings is 20.3m. The deck slab width is 10.5m. It is simply supported by four numbers of prestressed post tensioned concrete girders spaced at 2.5m.

After considering the dimensions of the anti-crash barriers and the footpath the carriage way width was determined as 7.5m.

Depending on the size of the carriage way the type of the loading combination was determined from the Table No: 3.2. The load combination was found out to be as Two lane of Class A or One lane of Class 70R. Referring the IRC 18:2000 the minimum dimensions for the girders where fixed as:

Web Thickness: 300mm

Top flange width: 900mm

Bottom flange width: 800mm

Overall Depth of Girder: 1900mm

Minimum thickness of deck slab: 250mm

Minimum cover to steel: 40mm

Minimum cover to duct: 75mm

The various section properties for the running section and end section where calculated. For the computation of the bending moment and shear force a model was prepared in the STAAD PRO software as shown in Fig: 4.1

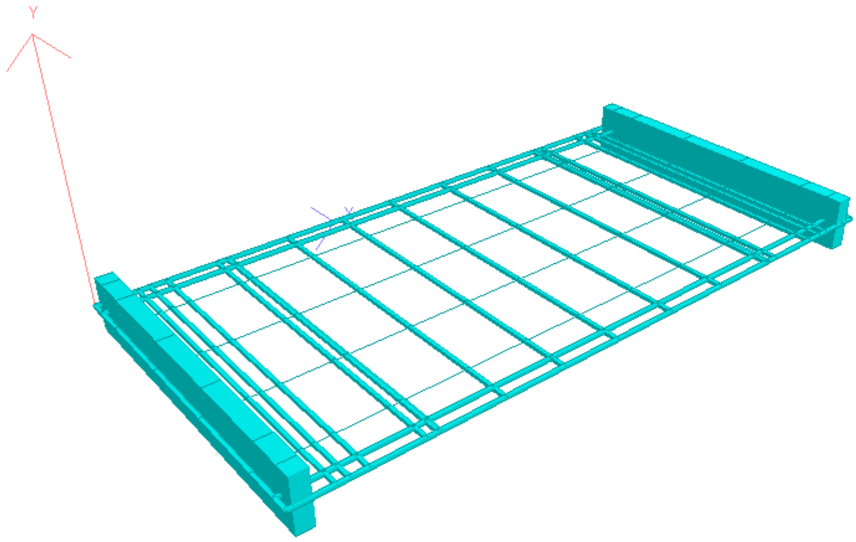


Fig 4.1: Staad Model

4.2.1 Staad Editor File

The editor file prepared in staad pro software after graphical modelling and inputs is shown below

UNIT METER KN

JOINT COORDINATES

4 0.000338018 0 -0.000200272; 5 20.6743 0 -0.000200272; 6 20.6743 0 10.4998;

11 0.000338018 0 10.4998; 14 0.400338 0 10.4998; 15 0.400338 0 -0.000200272;

31 0.400338 0 6.49979; 33 0.400338 0 8.99979; 35 0.400338 0 3.99979;

37 0.400338 0 1.49979; 74 0.000338018 0 6.49979; 75 0.000338018 0 8.99979;

76 0.000338018 0 3.99979; 77 0.000338018 0 1.49979; 78 20.6743 0 6.49979;

79 20.6743 0 8.99979; 80 20.6743 0 3.99979; 81 20.6743 0 1.49979;

83 0.400338 0 9.99979; 93 0.000338018 0 9.99979; 94 20.6743 0 9.99979;

96 0.400338 0 0.49979; 106 0.000338018 0 0.49979; 107 20.6743 0 0.49979;

108 9.9703 0 -0.000200272; 109 9.9703 0 10.4998; 110 9.9703 0 6.49979;

111 9.9703 0 8.99979; 112 9.9703 0 3.99979; 113 9.9703 0 1.49979;

114 9.9703 0 9.99979; 115 9.9703 0 0.49979; 116 8.06034 0 -0.000200272;

117 8.06034 0 10.4998; 118 8.06034 0 6.49979; 119 8.06034 0 8.99979;

120 8.06034 0 3.99979; 121 8.06034 0 1.49979; 122 8.06034 0 9.99979;

123 8.06034 0 0.49979; 124 6.10034 0 -0.000200272; 125 6.10034 0 10.4998;

126 6.10034 0 6.49979; 127 6.10034 0 8.99979; 128 6.10034 0 3.99979;

129 6.10034 0 1.49979; 130 6.10034 0 9.99979; 131 6.10034 0 0.49979;

132 4.09034 0 -0.000200272; 133 4.09034 0 10.4998; 134 4.09034 0 6.49979;

135 4.09034 0 8.99979; 136 4.09034 0 3.99979; 137 4.09034 0 1.49979;

138 4.09034 0 9.99979; 139 4.09034 0 0.49979; 140 2.11034 0 -0.000200272;

141 2.11034 0 10.4998; 142 2.11034 0 6.49979; 143 2.11034 0 8.99979;

144 2.11034 0 3.99979; 145 2.11034 0 1.49979; 146 2.11034 0 9.99979;

147 2.11034 0 0.49979; 148 3.70034 0 -0.000200272; 149 3.70034 0 10.4998;

150 3.70034 0 6.49979; 151 3.70034 0 8.99979; 152 3.70034 0 3.99979;

153 3.70034 0 1.49979; 154 3.70034 0 9.99979; 155 3.70034 0 0.49979;

156 1.25034 0 -0.000200272; 157 1.25034 0 10.4998; 158 1.25034 0 6.49979;

159 1.25034 0 8.99979; 160 1.25034 0 3.99979; 161 1.25034 0 1.49979;

162 1.25034 0 9.99979; 163 1.25034 0 0.49979; 164 12.0843 0 -0.000200272;

165 12.0843 0 10.4998; 166 12.0843 0 6.49979; 167 12.0843 0 8.99979;

168 12.0843 0 3.99979; 169 12.0843 0 1.49979; 170 12.0843 0 9.99979;

171 12.0843 0 0.49979; 172 14.1943 0 -0.000200272; 173 14.1943 0 10.4998;

174 14.1943 0 6.49979; 175 14.1943 0 8.99979; 176 14.1943 0 3.99979;

177 14.1943 0 1.49979; 178 14.1943 0 9.99979; 179 14.1943 0 0.49979;

180 16.3043 0 -0.000200272; 181 16.3043 0 10.4998; 182 16.3043 0 6.49979;

183 16.3043 0 8.99979; 184 16.3043 0 3.99979; 185 16.3043 0 1.49979;

186 16.3043 0 9.99979; 187 16.3043 0 0.49979; 188 18.4143 0 -0.000200272;

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207 19.2743 0 8.99979; 208 19.2743 0 3.99979; 209 19.2743 0 1.49979;

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213 20.1243 0 10.4998; 214 20.1243 0 6.49979; 215 20.1243 0 8.99979;

216 20.1243 0 3.99979; 217 20.1243 0 1.49979; 218 20.1243 0 9.99979;

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225 1.60034 0 1.49979; 226 1.60034 0 9.99979; 227 1.60034 0 0.49979;

228 4.05034 0 -0.000200272; 229 4.05034 0 10.4998; 230 4.05034 0 6.49979;

231 4.05034 0 8.99979; 232 4.05034 0 3.99979; 233 4.05034 0 1.49979;

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240 18.9243 0 3.99979; 241 18.9243 0 1.49979; 242 18.9243 0 9.99979;

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246 16.3943 0 6.49979; 247 16.3943 0 8.99979; 248 16.3943 0 3.99979;

249 16.3943 0 1.49979; 250 16.3943 0 9.99979; 251 16.3943 0 0.49979;

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495 232 136; 496 233 137; 497 234 138; 498 235 139;

DEFINE MATERIAL START

ISOTROPIC CONCRETE

E 2.17185e+007

POISSON 0.17

DENSITY 23.5616

ALPHA 1e-005

DAMP 0.05

ISOTROPIC CONCRETE0

E 2.17185e+007

POISSON 0.17

DENSITY 0.01

ALPHA 1e-005

DAMP 0.05

END DEFINE MATERIAL

MEMBER PROPERTY AMERICAN

3 7 8 22 41 124 126 128 130 151 163 164 176 188 TO 198 203 TO 213 218 TO 228 -

233 TO 243 248 TO 258 263 TO 273 278 TO 288 293 TO 303 308 TO 318 -

323 TO 333 338 TO 348 353 TO 363 368 TO 378 383 TO 393 398 399 407 408 413 -

414 TO 423 428 TO 438 443 TO 453 458 TO 469 474 TO 476 481 TO 485 490 TO 492 -

497 498 PRIS YD 0.1 ZD 0.1

10 56 TO 59 141 166 400 TO 406 PRIS YD 1.7 ZD 0.5

MEMBER PROPERTY AMERICAN

125 127 129 131 133 135 137 139 289 TO 292 379 TO 382 394 TO 397 439 TO 441 -

442 PRIS AX 2.168 AY 1.89656 AZ 1.89656 IX 2.694 IY 0.41 IZ 2.662

259 TO 262 274 TO 277 349 TO 352 364 TO 367 409 TO 412 454 TO 457 470 TO 473 -

477 TO 480 486 TO 489 493 TO 495 -

496 PRIS AX 1.8465 AY 1.61547 AZ 1.61567 IX 2.183 IY 0.384 IZ 2.148

199 TO 202 214 TO 217 229 TO 232 244 TO 247 304 TO 307 319 TO 322 334 TO 337 -

424 TO 427 PRIS AX 1.525 AY 1.33438 AZ 1.33438 IX 1.673 IY 0.357 IZ 1.635

CONSTANTS

MATERIAL CONCRETE MEMB 10 56 TO 59 125 127 129 131 133 135 137 139 141 166 -

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289 TO 292 304 TO 307 319 TO 322 334 TO 337 349 TO 352 364 TO 367 -

379 TO 382 394 TO 397 400 TO 406 409 TO 412 424 TO 427 439 TO 442 -

454 TO 457 470 TO 473 477 TO 480 486 TO 489 493 TO 496

MATERIAL CONCRETE0 MEMB 3 7 8 22 41 124 126 128 130 151 163 164 176 -

188 TO 198 203 TO 213 218 TO 228 233 TO 243 248 TO 258 263 TO 273 -

278 TO 288 293 TO 303 308 TO 318 323 TO 333 338 TO 348 353 TO 363 -

368 TO 378 383 TO 393 398 399 407 408 413 TO 423 428 TO 438 443 TO 453 458 -

459 TO 469 474 TO 476 481 TO 485 490 TO 492 497 498

SUPPORTS

31 33 35 37 214 TO 217 PINNED

DEFINE MOVING LOAD

Class A without impact

TYPE 100 LOAD 34 34 34 34 57.5 57.5 13.5 13.5

DIST 3 3 3 4.3 1.2 3.2 1.1 WID 2.3

LOAD GENERATION 205

TYPE 100 -18.8 0 3.95 XINC 0.2

TYPE 100 -18.8 0 7.45 XINC 0.2

The bending moment diagram obtained is as shown in Fig 4.2

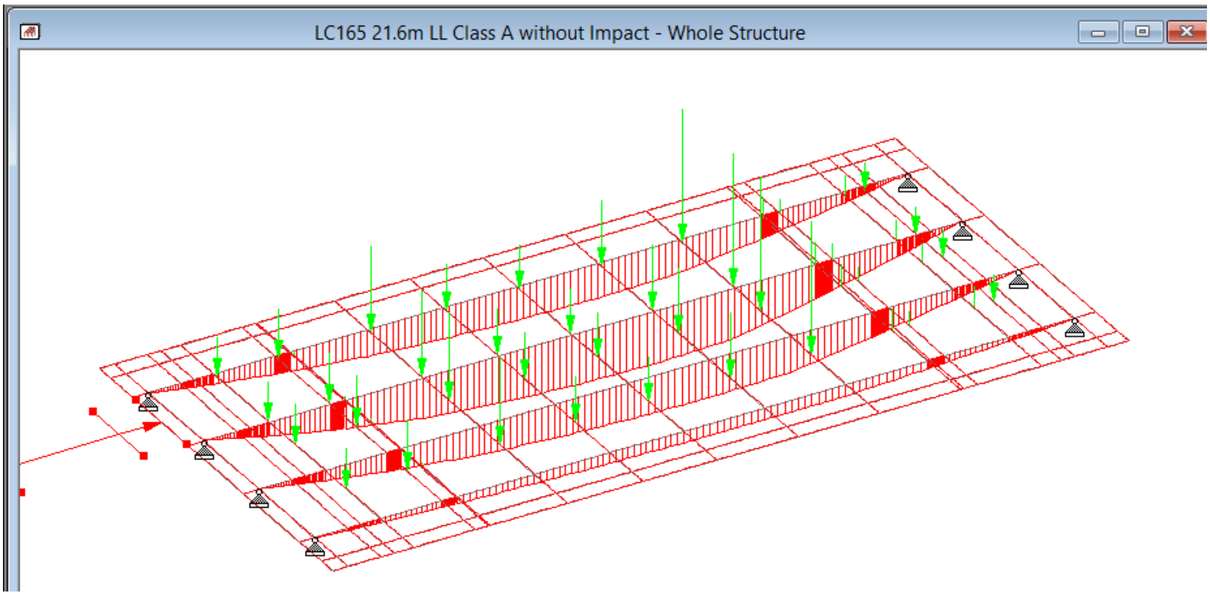


Fig 4.2: Class A without impact

Class A with impact

TYPE 110 LOAD 39.5284 39.5284 39.5284 39.5284 66.8495 66.8495 15.6951 15.6951

DIST 3 3 3 4.3 1.2 3.2 1.1 WID 2.3

LOAD GENERATION 205

TYPE 110 -18.8 0 3.95 XINC 0.2

TYPE 110 -18.8 0 7.45 XINC 0.2

The bending moment diagram obtained is as shown in Fig: 4.3

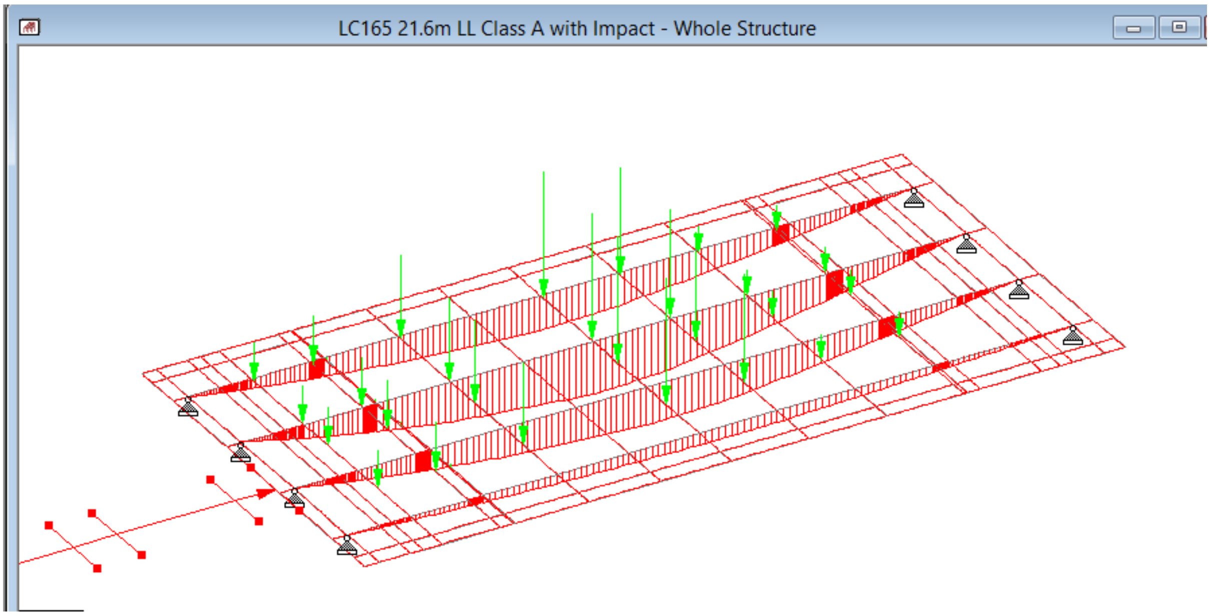


Fig 4.3: Class A with impact

70R without impact

TYPE 70 LOAD 85 85 85 85 60 60 40

DIST 1.37 3.05 1.37 2.13 1.52 3.96 WID 2.79

LOAD GENERATION 177

TYPE 70 -13.4 0 5.49 XINC 0.2

The bending moment diagram obtained is as shown in Fig: 4.4

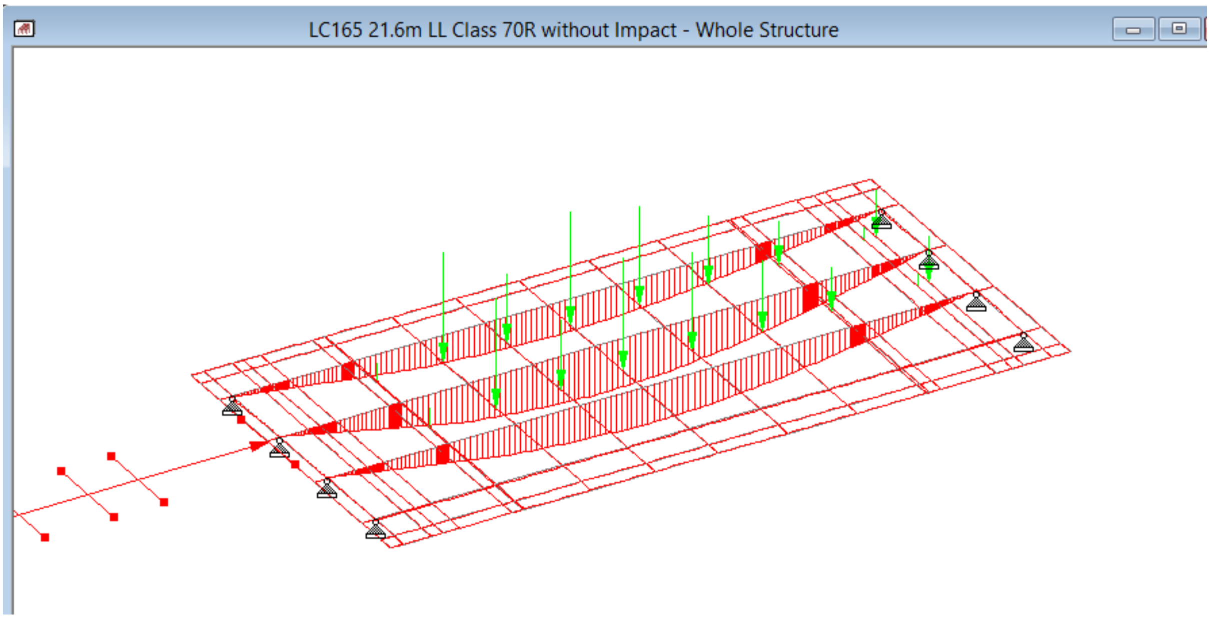


Fig 4.4: Class 70R without impact

70R with impact

TYPE 77 LOAD 98.821 98.821 98.821 98.821 69.756 69.756 46.504

DIST 1.37 3.05 1.37 2.13 1.52 3.96 WID 2.79

LOAD GENERATION 177

TYPE 77 -13.4 0 5.49 XINC 0.2

The bending moment diagram is as shown in Fig: 4.5

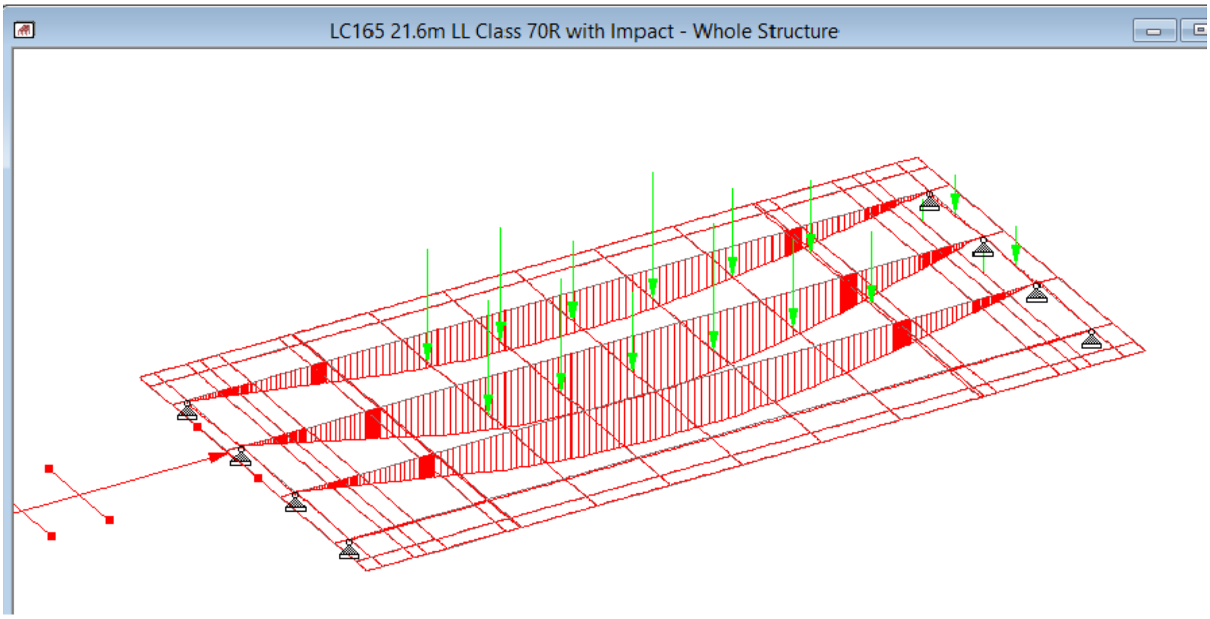


Fig4.5: Class 70R with impact

LOAD 206 LOADTYPE Dead TITLE FOOTPATH

FLOOR LOAD

YRANGE 0 0 FLOAD -5 ZRANGE 8.9 10.1 GY

YRANGE 0 0 FLOAD -5 ZRANGE 0.4 1.6 GY

PERFORM ANALYSIS

FINISH.

4.2.2 Output

The total bending moment and shear force obtained from the Staad Pro is as shown in Table 4.1

Table 4.1: Staad output

			Bending Moment		Shear Force		Footpath		Total Bending Moment	Total Shear Force
Location	From end (m)	From supp. End (m)	70R	Class A	70R	Class A	Moment	Shear		
L/2	10.55	10.15	156	131	13	9.45	23.3	0.45	179.30	13.45
0.4L	8.44	8.04	150	132	17	13.73	23	1.4	173.00	18.4
0.3L	6.33	5.93	129	117	21	18.25	19.6	2.4	148.60	24
0.2L	4.22	3.82	92.5	88.9	27	17.26	14.3	2.7	106.80	30
0.1L	2.11	1.71	47	32	31	22	6.7	3.8	53.70	35.1
Cha. End	4.10	3.70	85	46	29	25.40	12.9	3.5	97.90	32.5
X	1.60	1.20	24	80.8	32	8.83	3.1	0.1	83.90	32.1
0	0.40	0.00	0.5	5.82	34	8.83	0.75	0.05	6.57	34.05

Depending upon the bending moment diagram obtained from STAAD PRO the cable profile was decided to be as parabolic in nature. The various losses of prestress were determined in accordance with IS 1343:2000. The section was checked for ultimate shear and similarly the shear reinforcement was designed. The longitudinal reinforcements were designed in accordance with IRC 18:2000 and the minimum reinforcement was provided to be as 0.18% of the cross sectional area of concrete. Referring IRC 18:2000 clause 7.3 and clause 17 the end block and the bursting reinforcement was designed. The calculations of all the above stated particulars are shown in the following excel sheets.

4.4 Calculation of parabolic Cable profile ordinates.

Calculation of parabolic Cable profile ordinates. *CABLE NO. 1*

G

Y = *(10.4) x (10.4) / 676.00* = 0.16

A = *108 (sq.of parabolic length) / 0.160 cable lift above centre*

= *676.0*

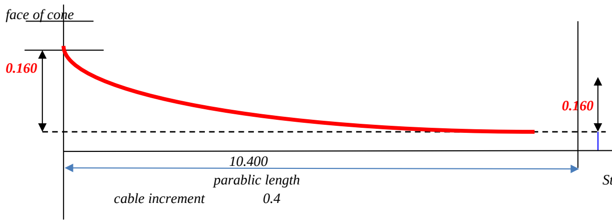
d = *distance of cable above soffit*

Rise of parabolla - f = *140 mm*

C1 = *12.1 m*

Tan F = *2 Y₁ / X₁*

Endblock ordinate = *300.0 mm*



0.64

	Y_1	=	X_1^2 / 676.0			Cable ordinate measured above soffit		Cable Angles				
	meters	section	distance	meters	IN mm	mm	meters	Tan F	Radians	Degrees		
	X_1			Y_1	Y1	Y_2	Y_2	$2 * Y_1 / X_1$				
0	**0.00**	**CENTRE**	**10.400**	**0.000**	**0.000**	**140.000**	**0.14**	**0.000**	**0.000**	**0.000**		0
1	0.400		10.000	0.000	0.237	**140.237**	0.14	0.001	0.001	0.068		1
2	0.800		9.600	0.001	0.947	**140.947**	0.14	0.002	0.002	0.136		2
3	1.200		9.200	0.002	2.130	**142.130**	0.142	0.004	0.004	0.203		3
4	1.600		8.800	0.004	3.787	**143.787**	0.14	0.005	0.005	0.271		4
5	2.000		8.400	0.006	5.917	**145.917**	0.15	0.006	0.006	0.339		5
6	2.400		8.000	0.009	8.521	**148.521**	0.15	0.007	0.007	0.407		6
7	2.800		7.600	0.012	11.60	**151.598**	0.15	0.008	0.008	0.475		7
8	3.200		7.200	0.015	15.15	**155.148**	0.16	0.009	0.009	0.542		8
9	3.600		6.800	0.019	19.17	**159.172**	0.16	0.011	0.011	0.610		9
10	4.000		6.400	0.024	23.67	**163.669**	0.16	0.012	0.012	0.678		10
11	4.400		6.000	0.029	28.64	**168.639**	0.17	0.013	0.013	0.746	REMARK	11
12	4.800		5.600	0.034	34.08	**174.083**	0.17	0.014	0.014	0.814		12
13	5.200		5.200	0.040	40.00	**180.000**	0.18	0.015	0.015	0.881		13
14	5.600		4.800	0.046	46.39	**186.391**	0.19	0.017	0.017	0.949		14
15	6.000		4.400	0.053	53.25	**193.254**	0.19	0.018	0.018	1.017		15
16	6.400		4.000	0.061	60.59	**200.592**	0.20	0.019	0.019	1.085		16
17	6.800		3.600	0.068	68.40	**208.402**	0.21	0.020	0.020	1.153		17
18	7.200		3.200	0.077	76.69	**216.686**	0.22	0.021	0.021	1.220		18
19	7.600		2.800	0.085	85.44	**225.444**	0.23	0.022	0.022	1.288		19
20	8.000		2.400	0.095	94.67	**234.675**	0.23	0.024	0.024	1.356		20
21	8.400		2.000	0.104	104.4	**244.379**	0.24	0.025	0.025	1.424		21
22	8.800		1.600	0.115	114.6	**254.556**	0.25	0.026	0.026	1.491		22
23	9.200		1.200	0.125	125.2	**265.207**	0.27	0.027	0.027	1.559		23
24	9.600		0.800	0.136	136.3	**276.331**	0.28	0.028	0.028	1.627		24
25	10.000		0.400	**0.148**	**147.9**	**287.929**	**0.29**	**0.030**	**0.030**	1.695		25
26	10.400	**END**	0.000	**0.160**	**160.0**	**300.000**	**0.30**	**0.031**	**0.031**	**1.762**		26

Calculation of parabolic Cable profile ordinates. *CABLE NO. 2*

G

Y = *(10) x (10) / 193.14* = 0.56

A = *108 (sq.of parabolic length) / 0.560 cable lift above centre*

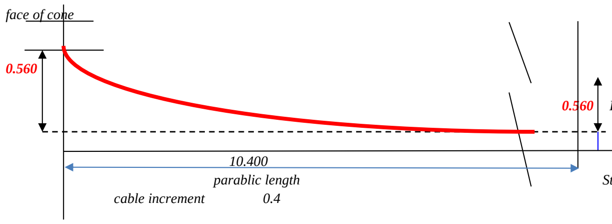
= *193.1*

d = *distance of cable above soffit*

Rise of parabolla - f = *340 mm*

Tan F = *2 Y₁ / X₁*

Endblock ordinate = *900.0 mm*



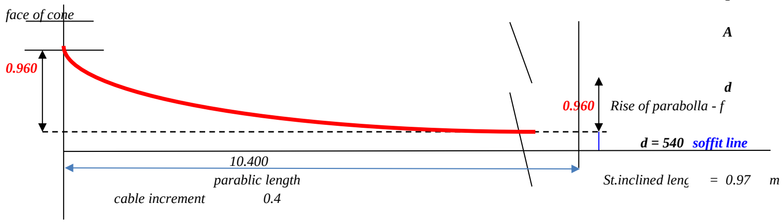
Y_1	=	X_1^2 / 193.1			Cable ordinate measured above soffit		Cable Angles				
meters	section	distance	meters	IN mm	mm	meters	Tan F	Radians	Degrees		
X_1			Y_1	Y1	Y_2	Y_2	$2 * Y_1 / X_1$				
0.00	**CENTRE**	**10.400**	**0.000**	**0.000**	**340.000**	**0.34**	**0.000**	**0.000**	**0.000**		0
0.400		10.000	**0.001**	0.828	**340.828**	0.34	0.004	0.004	0.237		1
0.800		9.600	**0.003**	3.314	**343.314**	0.34	0.008	0.008	0.475		2
1.200		9.200	**0.007**	7.456	**347.456**	0.347	0.012	0.012	0.712		3
1.600		8.800	**0.013**	13.254	**353.254**	0.35	0.017	0.017	0.949		4
2.000		8.400	**0.021**	20.710	**360.710**	0.36	0.021	0.021	1.186		5
2.400		8.000	**0.030**	29.822	**369.822**	0.37	0.025	0.025	1.424		6
2.800		7.600	**0.041**	40.59	**380.592**	0.38	0.029	0.029	1.661		7
3.200		7.200	**0.053**	53.02	**393.018**	0.39	0.033	0.033	1.898		8
3.600		6.800	**0.067**	67.10	**407.101**	0.41	0.037	0.037	2.135		9
4.000		6.400	**0.083**	82.84	**422.840**	0.42	0.041	0.041	2.372		10
4.400		6.000	**0.100**	100.24	**440.237**	0.44	0.046	0.046	2.609	REMARK	11
4.800		5.600	**0.119**	119.29	**459.290**	0.46	0.050	0.050	2.845		12
5.200		5.200	**0.140**	140.00	**480.000**	0.48	0.054	0.054	3.082		13
5.600		4.800	**0.162**	162.37	**502.367**	0.50	0.058	0.058	3.319		14
6.000		4.400	**0.186**	186.39	**526.391**	0.53	0.062	0.062	3.555		15
6.400		4.000	**0.212**	212.07	**552.071**	0.55	0.066	0.066	3.792		16
6.800		3.600	**0.239**	239.4	**579.408**	0.58	0.070	0.070	4.028		17
7.200		3.200	**0.268**	268.4	**608.402**	0.61	0.075	0.074	4.264		18
7.600		2.800	**0.299**	299.1	**639.053**	0.64	0.079	0.079	4.500		19
8.000		2.400	**0.331**	331.4	**671.361**	0.67	0.083	0.083	4.736		20
8.400		2.000	**0.365**	365.3	**705.325**	0.71	0.087	0.087	4.971		21
8.800		1.600	**0.401**	400.9	**740.947**	0.74	0.091	0.091	5.207		22
9.200		1.200	**0.438**	438.2	**778.225**	0.78	0.095	0.095	5.442		23
9.600		0.800	**0.477**	477.2	**817.160**	0.82	0.099	0.099	5.677		24
10.000		0.400	**0.518**	**517.8**	**857.751**	**0.86**	**0.104**	**0.103**	**5.912**		25
10.400	**END**	0.000	**0.560**	**560.0**	**900.000**	**0.90**	**0.108**	**0.107**	**6.147**		26

Calculation of parabolic Cable profile ordinates. **CABLE NO. 3** **G**

$$Y = \frac{(10)\ x \quad (10)}{112.67} = 0.96$$

$$A = \frac{108 \quad \text{(sq.of parabolic length)}}{0.960 \quad \text{cable lift above centre}} = 112.7$$

d = distance of cable above soffit = 540 mm



$$\text{Tan } F = \frac{2\ Y_1}{X_1}$$

Endblock ordinate = 1500.0 mm

Y_1	=	$\frac{X_1^2}{112.7}$			Cable ordinate measured above soffit		Cable Angles			REMARK	
meters	section	distance	meters	IN mm	mm	meters	Tan F	Radians	Degrees		
X_1			Y_1	Y1	Y_2	Y_2	$2 * Y_1 / X_1$				
0.00	**CENTRE**	**10.400**	**0.000**	**0.000**	**540.000**	**0.54**	**0.000**	**0.000**	**0.000**		0
0.400		10.000	**0.001**	1.420	**541.420**	0.54	0.007	0.007	0.407		1
0.800		9.600	**0.006**	5.680	**545.680**	0.55	0.014	0.014	0.814		2
1.200		9.200	**0.013**	12.781	**552.781**	0.553	0.021	0.021	1.220		3
1.600		8.800	**0.023**	22.722	**562.722**	0.56	0.028	0.028	1.627		4
2.000		8.400	**0.036**	35.503	**575.503**	0.58	0.036	0.035	2.033		5
2.400		8.000	**0.051**	51.12	**591.124**	0.59	0.043	0.043	2.440		6
2.800		7.600	**0.070**	69.59	**609.586**	0.61	0.050	0.050	2.845		7
3.200		7.200	**0.091**	90.89	**630.888**	0.63	0.057	0.057	3.251		8
3.600		6.800	**0.115**	115.03	**655.030**	0.66	0.064	0.064	3.657		9
4.000		6.400	**0.142**	142.01	**682.012**	0.68	0.071	0.071	4.062		10
4.400		6.000	**0.172**	171.83	**711.834**	0.71	0.078	0.078	4.466		11
4.800		5.600	**0.204**	204.50	**744.497**	0.74	0.085	0.085	4.870		12
5.200		5.200	**0.240**	240.00	**780.000**	0.78	0.092	0.092	5.274		13
5.600		4.800	**0.278**	278.3	**818.343**	0.82	0.099	0.099	5.677		14
6.000		4.400	**0.320**	319.5	**859.527**	0.86	0.107	0.106	6.080		15
6.400		4.000	**0.364**	363.6	**903.550**	0.90	0.114	0.113	6.482		16
6.800		3.600	**0.410**	410.4	**950.414**	0.95	0.121	0.120	6.883		17
7.200		3.200	**0.460**	460.1	**1000.118**	1.00	0.128	0.127	7.284		18
7.600		2.800	**0.513**	512.7	**1052.663**	1.05	0.135	0.134	7.683		19
8.000		2.400	**0.568**	568.0	**1108.047**	1.11	0.142	0.141	8.083		20
8.400		2.000	**0.626**	626.3	**1166.272**	1.17	0.149	0.148	8.481		21
8.800		1.600	**0.687**	687.3	**1227.337**	1.23	0.156	0.155	8.879		22
9.200		1.200	**0.751**	751.2	**1291.243**	1.29	0.163	0.162	9.275		23
9.600		0.800	**0.818**	818.0	**1357.988**	1.36	0.170	0.169	9.671		24
10.000		0.400	**0.888**	**887.6**	**1427.574**	**1.43**	**0.178**	**0.176**	**10.066**		25
10.400	**END**	0.000	**0.960**	**960.0**	**1500.000**	**1.50**	**0.185**	**0.183**	**10.460**		26

Chapter 5

Conclusion

Some of the conclusions that can be theorized are as follows:

1. Depending upon the bending moment diagram obtained from Staad Pro software a parabolic cable profile is provided
2. The values obtained by manual computation and that of Staad Pro software are found to be in good agreement.
3. The girder designed of dimensions
   Web Thickness: 300mm
   Top flange width: 900mm
   Bottom flange width: 800mm
   Overall Depth of Girder: 1900mm
   Is found to be safe in shear and bending moment.

Chapter 6

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5. IS 1343: 2012, Code of practice for Prestressed Concrete, Bureau of Indian Standards New Delhi, India.