SOFTWARE ANALYSIS AND DESIGN OF PRESTRESSED CONCRETE T-GIRDER (37.0m) ON MIDAS

A DISSERTATION

SUBMITTED IN THE PARTIAL FULFILLMENT OF THE REQUIREMENTS

FOR THE AWARD OF THE DEGREE OF

MASTER OF TECHNOLOGY

IN

STRUCTURAL ENGINEERING

Submitted by:

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SEPTEMBER, 2020

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CERTIFICATE

This is to certify that Ms. Reenu Verma Studying VIth Semester, of Part-Time M.Tech with specialization in Structural Engineering at Delhi Technological University (DTU) has completed the project on **"SOFTWARE ANALYSIS AND DESIGN OF PRESTRESSED CONCRETE T-GIRDER (37.0m) ON MIDAS"** as partial fulfillment of the requirement for the award of degree of Masters of Technology (2K17/STE/501) for the year 2019-2020.

To the best of my knowledge, the matter embodied in this report has not been submitted to any other university/institute for the award of any degree or diploma. The above statement made is correct to the best of our knowledge.

Place:- Delhi
Date: 12.09.2020

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CANDIDATE'S DECLARATION

I, Reenu Verma, 2K17/STE/501 student of M.Tech Structural engineering , hereby declare that the project Dissertation entitled **"SOFTWARE ANALYSIS AND DESIGN OF PRESTRESSED CONCRETE T-GIRDER (37.0m) ON MIDAS"** submitted at **Department of Civil Engineering, DTU, Delhi** is an authentic record of my work carried out under the supervision of **Prof. Alok Verma.** I have not submitted this work elsewhere for any other degree or diploma.

Date:- Delhi
Place:- 12.09.2020

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A project titled **"SOFTWARE ANALYSIS AND DESIGN OF PRESTRESSED CONCRETE T-GIRDER (37.0m) ON MIDAS"** was successfully accomplished as the part of second year M.Tech project, Department of Civil Engineering, DTU, Delhi. I take this opportunity to express my deep regards and profound gratitude to the faculty members who have helped me with their immense knowledge, inspiration and support to complete the project successfully. I am obliged to **Prof. Alok Verma**, Department of Civil Engineering for their timely assistance and valuable inputs to enhance the quality of my project.

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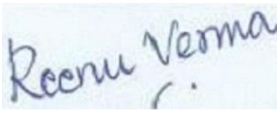
Thanking You,



Reenu Verma

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ABSTRACT

The concept of pre-stressed concrete appeared in the year 1888. In this present engineering technology, durable and sustainable bridges play an important role for the socio-economic development of the nation. Owners and designers have long recognized the low initial cost, low maintenance needs and long life expectancy of pre-stressed concrete bridges. This is reflected in the increasing market share of pre-stressed concrete, which has grown from zero in 1950 to more than 55 percent today. This growth continues very rapidly, not only for bridges in the short span range, but also for long spans with excessive length which, here therefore, has been nearly the exclusive domain of structural steel. Many bridge designers are surprised to learn that precast, pre-stressed concrete bridges are usually lower in first cost than all other types of bridges coupled with savings in maintenance, precast bridges offer maximum economy. The precast pre-stressed bridge system has offered two principal advantages: it is economical and it provides minimum downtime for construction.

Pre-stressing is the application of an initial load on the structure so as to enable the structure to counteract the stresses arising during its service period. In the present project, the behavior of pre-stressed concrete beams, how they will be stressed, the percentage of elongation, and the pressure applied to make beams pre-stressed will be thoroughly examined. This work presents a longitudinal and transverse design and analysis of PSC T-Girder which is 37.0m in span. The study focuses on PSC Beams, where the beam post-tensioning values, rate of elongation and behavior can be defined after stressing. The software MIDAS is used to analyze the T-girder.

PSC T-beam, have gained wide acceptance in freeway and bridge systems due to their structural efficiency, better stability, serviceability, economy of construction and pleasing aesthetics. PSC beam design is more complicated as structure is more complex as well as needed sophisticated from work. In the place of PSC T- beam if we talk about RCC T-beam geometry is simple and does not have sophisticated in construction.

The main code followed in this course is IS: 1343 – 2012 entitled Code of Practice for Pre-stressed Concrete. It is published by the Bureau of Indian Standards. Some provisions of Code IS: 456 - 2000 entitled Code of Practice for Structural Concrete are also applicable to Pre-stressed Concrete.

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CHAPTER 1

INTRODUCTION

__GENERAL

Bridge design is an important as well as complex approach of structural engineer. As in case of bridge design, span length and live load are always important factor. These factors affect the conceptualization stage of design. The effect of live load for various span are varied. In shorter spans track load govern whereas on larger span wheel load govern. Selection of structural system for span is always a scope for research. Structure systems adopted are influence by factor like economy and complexity in construction. Code strategy engages us to pick structural system i.e. T- Beam Girder of 37.0 m span as selected for this study. In 37.0 m span, code provisions allow as to choose a structural system i.e. PSC T- beam. This study investigates the structural systems for span 37 m and detail design has been carried out with IRC loadings and IS code books. The choice of economical and constructible structural system is depending on the result.

Bridge design is a goal and what's more personalities boggling approach for the structural design. Bridge is life line of road network, both in urban and rural areas. With rapid technology growth the conventional bridge has been replaced by innovative cost effective structural system. One of these solutions presents a structural PSC system that is T-Beam.

PSC T-beam, have gained wide acceptance in freeway and bridge systems due to their structural efficiency, better stability, serviceability, economy of construction and pleasing aesthetics. PSC beam design is more complicated as structure is more complex as well as needed sophisticated from work. In the place of PSC T- beam if we talk about RCC T- beam geometry is simple and does not have sophisticated in construction.

T-BEAM

T-beam utilized as a part of construction, is a load bearing structure of reinforced cement concrete, wood or metal, with a t-formed cross area. The highest point of the t-molded cross segment fills in as a flange or pressure part in opposing compressive stress. The web (vertical area) of the beam beneath the compression flange serves to oppose shear stress and to give more noteworthy detachment to the coupled strengths of bending

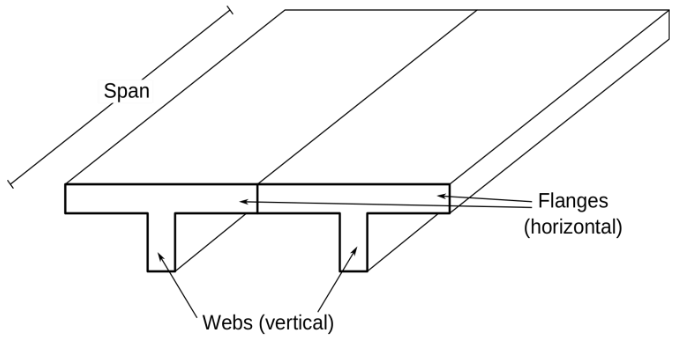


Fig 1: T-Beam

T-beam, used in construction, is a load-bearing structure of reinforced concrete, wood or metal, with a t-shaped cross section. The top of the T-shaped cross section serves as a flange or compression member in resisting compressive stresses. The web of the beam below the compression flange serves to resist shear stress and to provide greater separation for the coupled forces of bending.

A beam and slab bridge or T- beam bridge is constructed when the span is between 10 -25 m. The bridge deck essentially consists of a concrete slab monolithically cast over longitudinal girders so that the T-beam effect prevails. To impart transverse stiffness to the deck, cross girders or diaphragms are provided at regular intervals. The number of longitudinal girders depends on the width of the road. Three girders are normally provided for a two lane road bridge. T-beam bridges are composed of deck slab 20 to 25cm thick and longitudinal girders spaced from 1.9 to 2.5m and cross beams are provided at 4 to 5m interval.

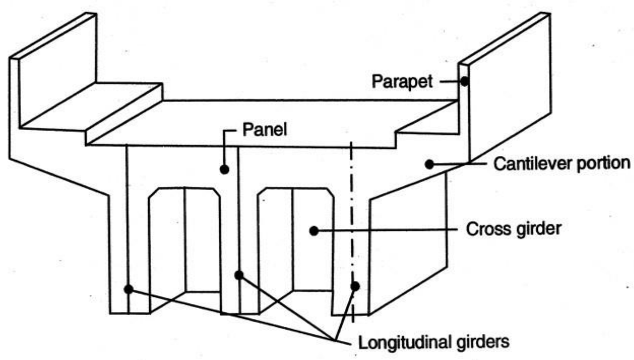


Fig 2: Components of T-Beam Bridge

ADVANTAGES

- ✓ Beam bridges are helpful for short spans.
- ✓ Long distances are normally covered by placing the beams on piers.
- ✓ It has simply geometry.
- ✓ Easy to cast in construction.
- ✓ It is mostly adopted Bridge.
- ✓ Slab act monolithically with beam

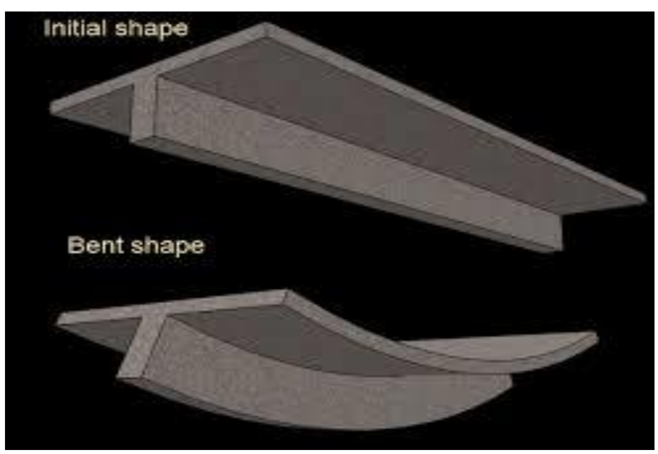


Fig 3: T-Girder

PRESTRESSED CONCRETE

History and background

A prestressed concrete structure is different from a conventional reinforced concrete structure due to the application of an initial load on the structure prior to its use. The initial load or prestress is applied to enable the structure to counteract the stresses arising during its service period. The prestressing of a structure is not the only instance of prestressing. The concept of prestressing existed before the applications in concrete.

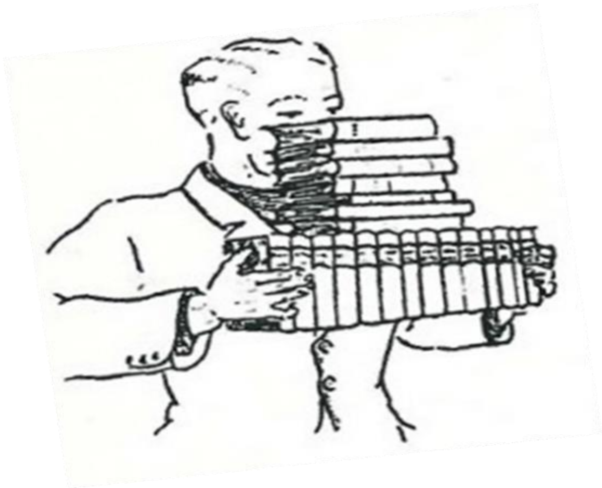


Fig 4 - Here is Belgian engineer Gustave Magnel's drawing that explains prestressing by showing how a row of books, pressed tightly together end to end, becomes a beam capable of supporting more books.

The following two examples of prestressing before the development of prestresses concrete are provided.

Force fitting of metal bands on wooden barrel is an example in which the metal bands induce a state of initial hoop compression, to counteract the hoop tension caused by filling of liquid in barrels.

Pre tensioning the spokes in a bicycle wheel is also an example here tension is applied to such an extent that there will always be a residual tension in the spoke.

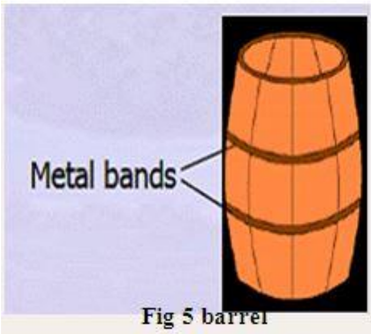


Fig 5 barrel

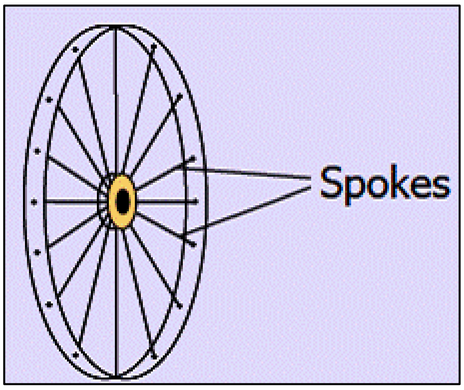


Fig 6: Wheel spokes

Before the development of prestressed concrete, two significant developments of reinforced concrete are the invention of Portland cement and introduction of steel in concrete. These are also mentioned as the part of the history. The key developments are mentioned next to the corresponding year.

1. 1824-Aspdin.J. (England) Obtained a patent for the manufacture of Portland cement.
2. 1857-Monier.J. (France) Introduced steel wires in concrete to make flower pots, pipes, arches and slabs.
3. 1886-Jackson.P.H. (USA) Introduced the concept of tightening steel tie rods in artificial stone and concrete arches.
4. 1888-Doehring.C.E.W. (Germany) Manufactured concrete slabs and small beams with embedded tensioned steel.
5. 1908-Stainer.C.R. (USA) Recognised losses due to shrinkage and creep, and suggested retightening the rods to recover lost prestress.
6. 1923-Emperger.F. (Austria) Developed a method of winding and pre- tensioning high tensile steel wires around concrete pipes.
7. 1924-Hewett.W.H. (USA) Introduced hoop-stressed horizontal reinforcement around walls of concrete tanks through the use of turnbuckles. Thousands of liquid storage tanks and concrete pipes were built in the two decades to follow.
8. 1925-Dill.R.H.(USA) Used high strength unbonded steel rods. The rods were tensioned and anchored after hardening of the concrete. 1926-Eugene Freyssinet (France) Used high tensile steel wires, with ultimate strength as high as 1725 MPa and yield stress over 1240 MPa. In 1939, he developed conical wedges for end anchorages for post-tensioning and developed double-acting jacks. He is often referred to as the **Father of Prestressed concrete.**
9. 1938-Hoyer.E. (Germany) Developed „long line" pre-tensioning method.

Fig 7: Portrait of Eugene Freyssinet

10. 1940-Magnel.G. (Belgium) Developed an anchoring system for post-tensioning, using flat wedges.

11. During the Second World War, applications of prestressed and precast concrete increased rapidly. The names of a few persons involved in developing prestressed concrete are mentioned. Guyon, Y., (France) built numerous prestressed concrete bridges in western and central Europe. Abeles, P. W., (England) introduced the concept of partial prestressing. Leonhardt, F., (Germany), Mikhailor, V., (Russia) and Lin, T. Y., (USA) are famous in the field of prestressed concrete.

12. The International Federation for Prestressing (FIP), a professional organisation in Europe was established in 1952. The Precast/Prestressed Concrete Institute (PCI) was established in USA in 1954. Prestressed concrete was started to be used in building frames, parking structures, stadiums, railway sleepers, transmission line poles and other types of structures and elements.

13. In India, the applications of prestressed concrete diversified over the years. The first prestressed concrete bridge was built in 1948 under the Assam Rail Link Project. Among bridges, the Pamban Road Bridge at Rameshwaram, Tamil nadu, remains a classic example of the use of prestressed concrete girders.

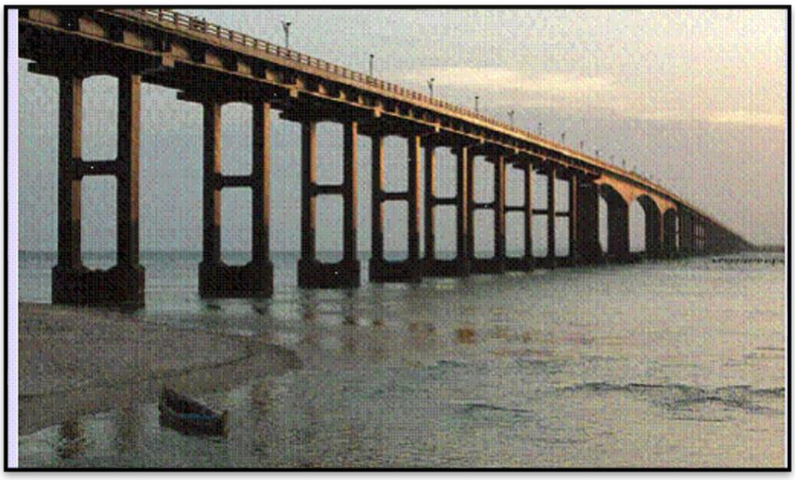


Fig 8: Pamban bridge, Rameshwaram, Tamil Nadu.

The development of prestressed concrete can be studied in the perspective of traditional building materials. In the ancient period, stones and bricks were extensively used. These materials are strong in compression, but weak in tension. For tension, bamboos and coir ropes were used in bridges. Subsequently iron and steel bars were used to resist tension. These members tend to buckle under compression. Wood and structural steel members were effective both in tension and compression.

In reinforced concrete, concrete and steel are combined such that concrete resists compression and steel resists tension. This is a passive combination of the two materials. In prestressed concrete high strength concrete and high strength steel are combined such that the full section is effective in resisting tension and compression. This is an active combination of the two materials. The following sketch shows the use of the different materials with the progress of time.

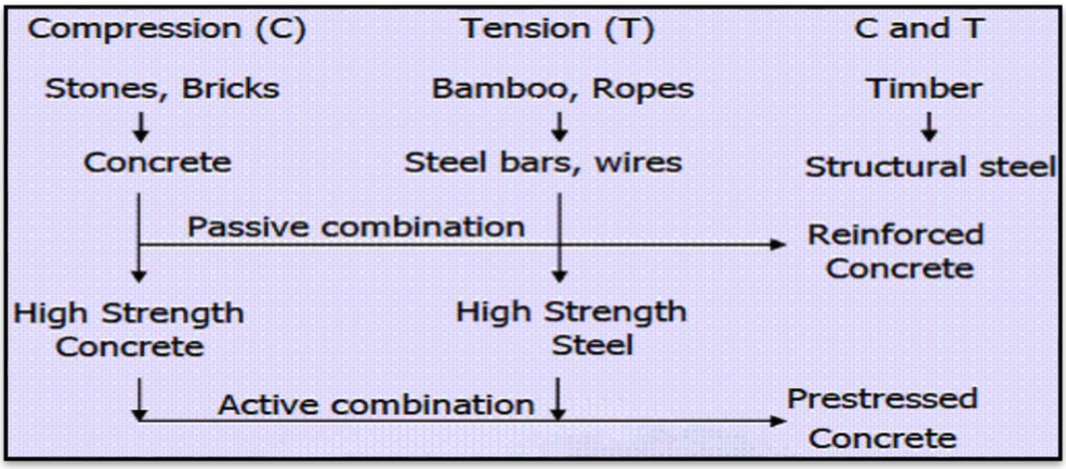


Fig 9: Development of building material

Types of Prestressing

Prestressing can be accomplished in three ways: pre-tensioned concrete, and bonded or unbonded post-tensioned concrete.

Pre-tensioned concrete is cast around already tensioned tendons. This method produces a good bond between the tendon and concrete, which both protects the tendon from corrosion and allows for direct transfer of tension. The cured concrete adheres and bonds to the bars and when the tension is released it is transferred to the concrete as compression by static friction. However, it requires stout anchoring points between which the tendon is to be stretched and the tendons are usually in a straight line. Thus, most pre-tensioned concrete elements are prefabricated in a factory and must be transported to the construction site, which limits their size. Pre-tensioned element

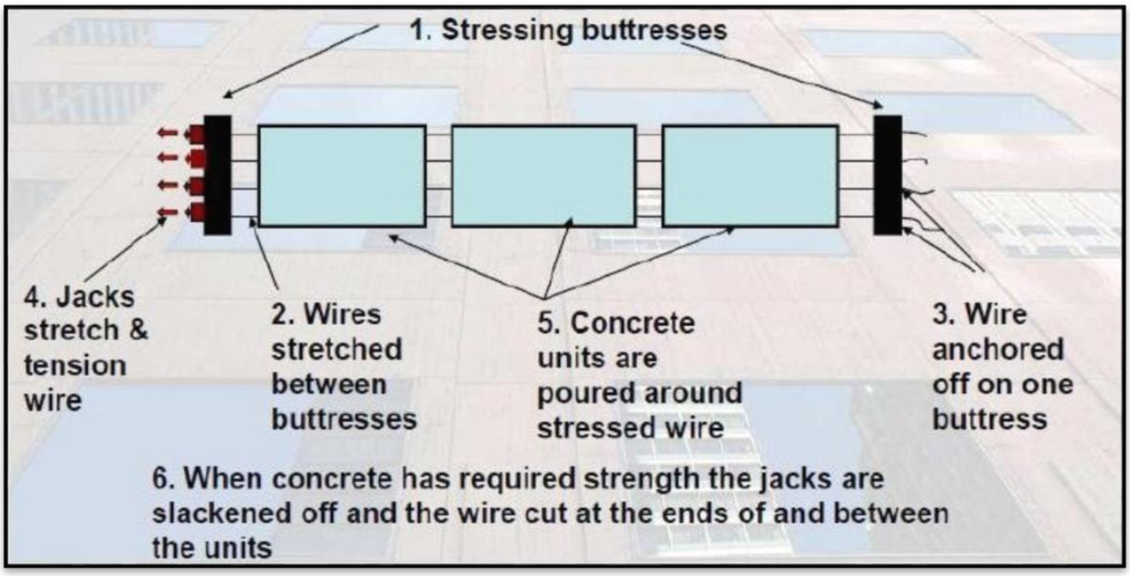


Fig 10: Pre-tensioning of beams

may be balcony elements, lintels, floor slabs, beams or foundation piles. An innovative bridge construction method using pre-stressing is the stressed ribbon bridge design.

1. Bonded post-tensioned concrete is the descriptive term for a method of applying compression after pouring concrete and the curing process (*in situ*). The concrete is cast around a plastic or steel or aluminum curved duct, to follow the area where otherwise tension would occur in the concrete element. A set of tendons are fished through the duct and the concrete is poured. Once the concrete is hardened, the tendons are tensioned by hydraulic jacks that react (push) against the concrete member itself. When the tendons are stretched sufficiently, according to the design specifications (see Hooke's law), they are wedged in position and maintain tension after the

jacks are removed, transferring pressure to the concrete. The duct is then grouted to protect the tendons from corrosion. This method is commonly used to create monolithic slabs for house construction in locations where expansive soils (such as adobe clay) create problems for the typical perimeter foundation. All stresses from seasonal expansion and contraction of the underlying soil are taken into the entire tensioned slab, which supports the building without significant flexure. Post-tensioning is also used in the construction of various bridges; both after concrete is cured after support by false work and by the assembly of prefabricated sections, as in the segmental bridge.

2. Unbounded post-tensioned concrete differs from bonded post-tensioning by providing each individual cable permanent freedom of movement relative to the concrete. To achieve this, each individual tendon is coated with grease (generally lithium based) and covered by a plastic sheathing formed in an extrusion process. The transfer of tension to the concrete is achieved by the steel cable acting against steel anchors embedded in the perimeter of the slab. The main disadvantage over bonded post-tensioning is the fact that a cable can destress itself and burst out of the slab if damaged (such as during repair on the slab).

Fig 11: Steel tendons being stretched by jacks in post tensioned members

METHODS FOR POST TENSIONING (FREYSSINET SYSTEM)

There are various methods of pre-stressing in our project for our project we adopted post tensioned member for the following reason,

1. Post-tensioning allows longer clear spans, thinner slabs, fewer beams and more slender, dramatic elements.
2. Thinner slabs mean less concrete is required.
3. Post-tensioning can thus allow a significant reduction in weight versus a conventional concrete building with the same number of floors reducing the foundation load and can be a major advantage in seismic areas.
4. A lower structure weight and size can also translate to considerable savings in mechanical systems and façade costs.
5. Another advantage of post-tensioning is that beams and slabs can be continuous,i.e. a single beam can run continuously from one end of the building to the other.
6. Reduces occurrence of tension cracks.
7. Freezing & thawing durability is higher than non pre stressed concrete.
8. Post-tensioning allows bridges to be built to very demanding geometry requirements, including complex curves, and significant grade changes.
9. Post-tensioning also allows extremely long span bridges to be constructed without the use of temporary intermediate supports. This limits the effect on nature and evades interruption to water or street traffic underneath. Consequently for receiving post strain framework we use Freyssinet framework which is a simple and practical technique accordingly making it the most generally utilized strategy. As post tensioning is appropriate for bend links of various link profile, subsequent to projecting of the solid the pressure are acquainted with the wires either from one end or from both the finishes. The chief depends on wet activity. It comprises of a chamber with a tapered inside through cylinders. This permits high pressure of wires to skillet against the mass of the wire and is wedge by a tapered attachment. These wedges will have number of wires in the abandoned structure and these wires are bent to take the torsional obstruction of the structures. to evade loss of prestress because of versatile shortening of cement these abandoned links are tensioned at the same time to the ideal estimation of beginning pressure. At times to diminish the heap bearing limit just as to adjust various sorts of burden following up on the part links of various profiles gave in the wedge tube. In such cases links are tensioned and moored progressively.

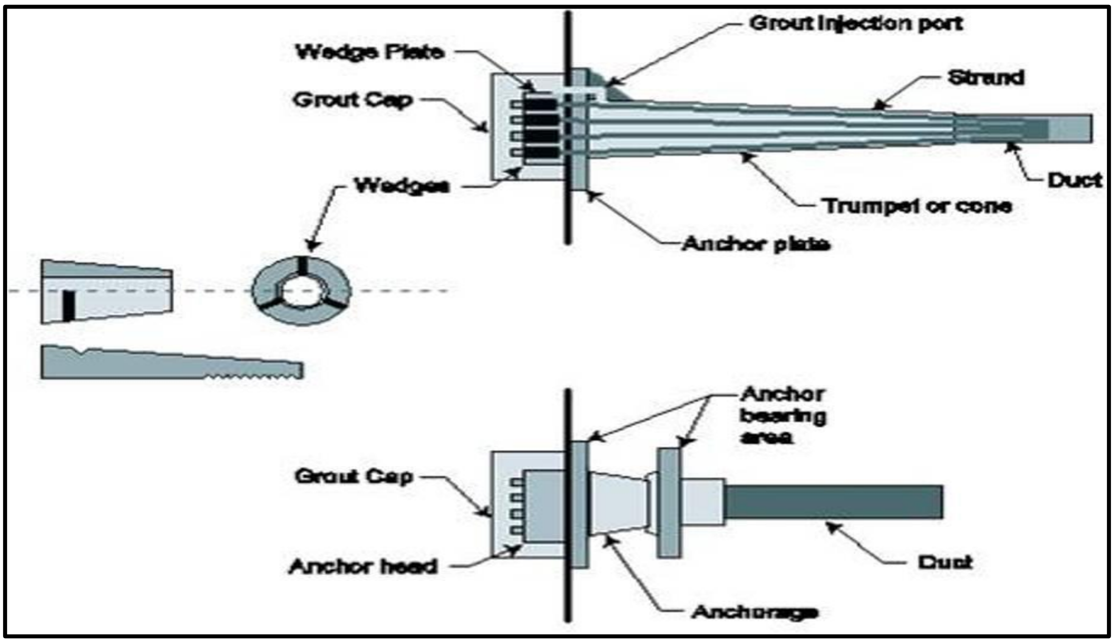


Fig 12: Shows all the equipment and the method of post tensioning

RESULTANT LONGITUDINAL STRESS DEVELOPED IN PSC SECTION

The analysis of stress developed in PSC section is based on the following assumption,

1. Concrete is homogeneous and elastic material. With the range of working stress both concrete and steel behave elastically not withstanding small amount of creep which occurs in both materials under sustained load.
2. A plane section before bending will remain plain even after bending. Here the analysis of stress are done in two steps which are as follows,
   i. Unloaded condition.
   ii. Working load condition.

The general formula f or finding out the stresses in longitudinal section is,

$$f= (F/A) + (F*e*y/I) + (M*y/I)$$

Where,

f= stress developed at the required longitudinal section

F= is the prestressing force induced in the wires.

e=eccentricity of the centroidal axis of the steel wire.

y=distance of longitudinal fibre from centroidal axis.

A= Area of cross section

M= bending moment due to self-weight and working load as per the required condition.

I=moment of inertia of the section about centroidal axis of bending.

η=loss ratio is defined as the ratio of effective Pestressing force to the initial pre stressing force.

Here, the above stresses at both the conditions must be satisfied and tension is not permitted means there is no shear reinforcement required.

The values of e and y are taken positive if measured above centroidal axis and if they are measured below the centroidal axis the values are taken as negative.

Here for moment values o M for unloaded condition only girder moment or dead load moment is considered and for working load condition overall moment that is the sum of dead load and live load moment is considered.

The maximum permissible compressive stress of concrete is taken as $0.47f_{ck}$ for M30 concrete and $0.35f_{ck}$ for M60 concrete. Here f_{ck} is the grade of concrete as per the IS 1343-2012 (code of practice for prestressed concrete).

CALCULATION OF PRESTRESSING FORCE

After selecting the cross section of the members all the parameters such as centroid, area, moment of inertia, section modulus and the inferior and superior stresses are calculated. Then from inferior and superior stresses the prestressing force is calculated as follows,

$$\mathbf{P= (A*f_{inf}*Z_b)/(Z_b+A*e)}$$

Where,

P= prestressing force A= area of section

f_{inf}= inferior stress at the section

Z_b=section modulus at bottom of centroidal axis e=eccentricity of the cable

After selecting the system and type of anchorage number of cables are calculated depending on the ultimate breaking load of steel strands.

END BLOCKS

Unlike in a pre-tensioned member without anchorage, the stress in the tendon of a posttensioned member attains the prestress at the anchorage block. There is no requirement of transmission length or development length. The end zone (or end block) of a post-tensioned member is a flared region which is subjected to high stress from the bearing plate next to the anchorage block. It needs special design of transverse reinforcement. The design considerations are bursting force and bearing stress. The stress field in the end zone of a post-tensioned member is complicated. The compressive stress trajectories are not parallel at the ends. The trajectories diverge from the anchorage block till they become parallel. Based on Saint Venant's principle, it is assumed that the trajectories become parallel after a length equal to the larger transverse dimension of the end zone. The following figure shows the external forces and the trajectories of tensile and compressive stresses in the end zone.

Stress trajectories in the end zone

The larger transverse dimension of the end zone is represented as y0. The corresponding dimension of the bearing plate is represented as yp0. For analysis, the end zone is divided into a local zone and a general zone as shown in the following sketch.

The transverse tensile stress is known as splitting tensile stress. The resultant of the tensile stress in a transverse direction is known as the bursting force (Fbst). Compared to pre-tensioned members, the transverse tensile stress in post-tensioned members is much higher. Besides the bursting force there is spalling forces in the general zone. Spalling force Bursting force

IS:1343 - 2012, Clause 18.6.2.2, provides an expression of the bursting force (Fbst) for an individual square end zone loaded by a symmetrically placed square bearing plate. The formula is

$$\mathbf{Fbst = Pk*(0.32-0.3*\ y_{po}/y_0)}$$

Here,

Pk = prestress in the tendon

Y_{P0} = length of a side of bearing plate

y_0 = transverse dimension of the end zone.

The following sketch shows the variation of the bursting force with the parameter y_{P0}

/ y_0.

The parameter represents the fraction of the transverse dimension covered by the Bearing plate. It can be observed that with the increase in size of the bearing plate the bursting force (Fbst) reduces. The following sketch explains the relative size of the bearing plate with respect to the end zone.

END ZONE REINFORCEMENT

Transverse reinforcement is provided in each principle direction based on the value of Fbst. This reinforcement is called end zone reinforcement or anchorage zone reinforcement or bursting links. The reinforcement is distributed within a length from 0.1y0 to y0 from an end of the member. The amount of end zone reinforcement in each direction (Ast) can be calculated from the following equation.

$$\mathbf{A_{st} = F_{bst}/fs}$$

The stress in the transverse reinforcement (fs) is limited to 0.87fy. When the cover is less than 50 mm, fs is limited to a value corresponding to a strain of 0.001.

The end zone reinforcement is provided in several forms, some of which are proprietary of the construction firms. The forms are closed stirrups, mats or links with loops. A few types of end zone reinforcement is shown in the following sketches. the local zone is further strengthened by confining the concrete with spiral reinforcement. The performance of the reinforcement is determined by testing end block specimens. The end zone may be made of high strength concrete. The use of dispersed steel fibres in the concrete (fibre reinforced concrete) reduces the cracking due to the bursting force. Proper compaction of concrete is required at the end zone. Any honey-comb of the concrete leads to settlement of the anchorage device. If the concrete in the end zone is different from the rest of the member, then the end zone is cast separately.

CHAPTER 2

LITRATURE REVIEW

N.K Paul,(2011)[1] In this review, it is exhibited that, utilization of super elastic shape memory alloy bars consolidating with steel reinforcement with some rate in T-Beam concrete bridge longitudinal girder works successfully exceptionally well. The load carrying capacity can be increased. The failure mechanism of a reinforced concrete girder is demonstrated great utilizing FEA, and the failure load anticipated is near the failure load measured during trial testing. The whole load distortion reaction of the model created coordinates well with the reaction from trial result. This gave trust in the utilization of ANSYS 11.0 and the model created.

R.Shreedhar Spurti Namadapur,(2012)[2] A straightforward span T-beam extension was analyzed by utilizing I.R.C. determinations and loading (dead load and live load) as a one dimensional structure. Finite Element analysis of a three-dimensional structure was done using Staad pro programming. Both models were subjected to I.R.C. Loadings to convey most outrageous bending moment. The results were broke down and it was found that the results got from the limited component model are lesser than the results got from one dimensional examination, which suggests that the results got from I.R.C. loadings are traditionalist and FEM gives practical design.

Amit Saxena,(2013)[3] Dead load bending moment and Shear forces for T-Beam girder are lesser than two cell Bridge. Which empower designer to have lesser heavier region for T-Bar Support than Box Brace for 25 m span. Moment of resistance of steel for both has been evaluated and conclusions drawn that T-Beam Girder has more noteworthy utmost with respect to 25 m span. Cost of concrete for T-Beam Girder is under two cell as sum required by T-Beam Girder.

Mahesh Pokhrel,(2013)[4] General plan and examination of a common T-Girder RCC Bridge has been finished with Assessment of response and structure speculations according to three overall codes to be explicit IRC, AASHTO and Euro code. Among of all, the Euro code gave most moderate plan. It may be a direct result of the use of characteristics load used with no part. Euro code is made up for broad assortment of relevance and degree so it very well may be alluded for the structure of scaffolds. In which truck stacking is used for response in the superstructure and in which non-direct lead of dock and projection isn't thought of. Considering nonlinearity is one of the proposals for the future work for more reasonable result.

Vishal U. Misal,(2014)[5] The cost analysis and design of prestressed concrete girder and reinforced concrete girder is presented under a IRC class 70 R loading to formulate the entire problem for a couple of span under the loading mentioned above to obtain shear force and bending moment at regular intervals along the beam. The software STAAD PRO is used for the will be validated by comparing its results with the corresponding classical theory result. To carry out the parametric analysis for prestressed concrete I girder and reinforced concrete girder. To calculate the quantities of concrete and steel required as per the analysis and design carried out for the girders and to carry out the comparative study for the same analysis and design of prestressed concrete girders. Before using the software for analysis it

Rajamoori Arun Kumar,(2014)[7] Bending moment and shear force for PSC T-Beam Girder are lesser then RCC T-Beam girder bridge. Which allow designer to have lesser heavier section for PSC T-Beam Girder then RCC T-Beam Girder for 24m span. Moment of resistance of PSC T-Beam Girder is more as compare to RCC T-Beam Girder for 24 m span. Cost of concrete for PSC T-Beam Girder is less then RCC T-Beam Girder.

Manjeetkumar M Nagarmunnoli,(2014)[8] Focus about on the impacts of deck thickness in RCC T-Pillar Extension. For each decrement in deck section thickness diminishes the bowing solidness by around 40% to half. Stresses acting in the deck under truck wheel load are around multiple times more undeniable than the suitable loads. For each decrement in the deck piece thickness from 280 mm to 150 mm would significantly collect the part incline by around 31% under the wheel stack. The uncracked portrayal of inaction rots by around 45% for each decrement in the deck territory thickness from 280 mm to 150 mm exposed to IRC Class A truck stacking. The Bend power made in the deck piece diminishes by around 0.43% for each decrement in the deck section thickness.

Sandesh Upadhayaya,(2016)[11] To obtain even better working results the T-beam configuration deck slab can be subjected to pre/post tensioning. The pre-stressing force can be applied more conveniently and computation of required jacking force is also simple. This problem can be overcome with greater ease in case of T-Beam deck slab configuration.

Mayur Hingane,(2018)[12] T- girder bridges are commonly used type of bridge. they are easy to construct and maintain because the structural construction of such bridges are easy. Hence mostly they are preferred due to the critical design of other type of bridges as it provides connectivity within shorter and medium distance. The aim of our study was to analysed the t-girder bridge by using staad pro. Software. in this study we have consider span length of 25m. the deck slab has been analyses for IRC class AA loading using carbons method. excel sheet is made to design the maximum Bending Moment, Maximum Shear Force which produced due to dead load and live load of class AA tracked vehicle.

Sanket Patel,(2016)[13] The study includes parametric study on prestressed concrete girder bridge superstructure. After analyzing and Tee Girder with CSI Bridge 2014 it concluded that as the span increases the shows better results for selecting between both girders. By the numbers of prestressing cables required to resist the load, required less cables. Loads are almost similar in both the girders but for 40m span is governing section is governing but is has its own flaws too. It is having a complex shuttering and it's required more skilled labours to carry out that task but overall is preferable.

Abrar Ahmed,(2017)[14] By validating the analytical data with the manual, it can be concluded that the software (CSI Bridge) results can be considered for the design of substructure as the results obtained is showing good agreement. By extracting the results it is seen that for the spans greater than 30m, is economical overall and is suitable type of section. For lower spans the T-beam girder can be adopted which is easy to install and maintain. By having self-developed excel user feels easy to design the sections for different spans in less time. Number of cells in the can be increased to decrease the overall depth of the girder for higher spans.

CHAPTER 3

OBJECTIVE AND RESEARCH METHODOLOGY

3.1 OBJECTIVES

- To concentrate the conduct of basic PSC T-beam beam and bridge under standard IRC loading in MIDAS Bridge software
- To study the deck slab interaction with the loading considered as IRC Codes.
- To evaluate the suitability of the bridges for long span
- To evaluate code expressions for live-load distribution factors for prestressed concrete girder bridges.

3.2. RESEARCH METHODOLOGY

3.2 General Hypotheses

3.2 Model Simulation of T-girder Longitudinally

3.2.1 Principle of Modeling

3.2.2 Description of Midas Software

3.3 Loads Applied in Modeling

3.4 Midas Input

3.5 Prestressing Layout of T-Girder

3.6 Construction Sequence

3.7 Model Simulation of T-Girder Deck Slab Transversely

3.8 External Loads Applied in Modeling (with OHE)

3.9 External Loads Applied in Modeling (without OHE)

3.10 Live Load

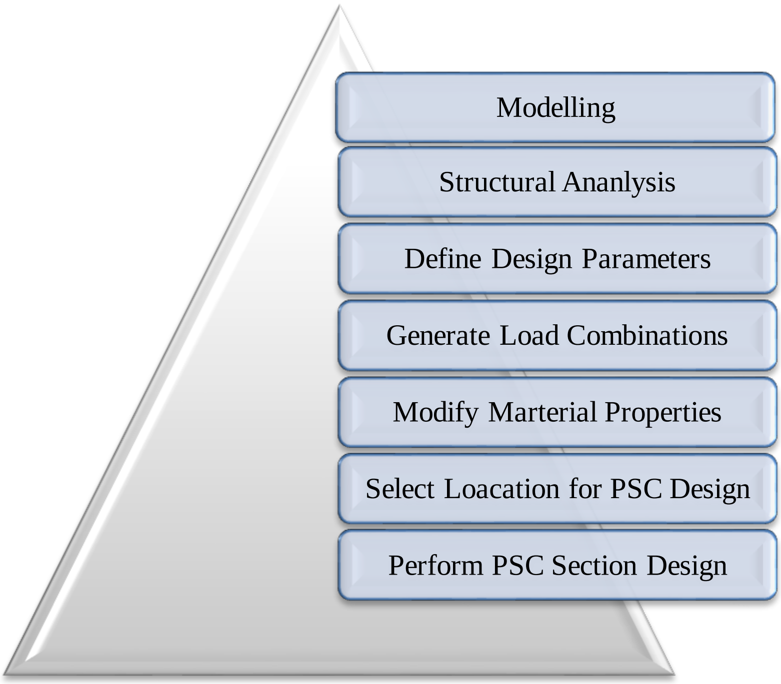
RESEARCH METHODOLOGY

This report presents the longitudinal analysis of Precast Pretension T-Girder of 37.0m span in straight alignment.

This design note includes:

- ✓ Verification of flexural stresses along T-Girder in construction and in service stages.
- ✓ Verification of maximum permissible shear stresses & reinforcement
- ✓ Verification of Shear Connector reinforcement.
- ✓ Verification of Ultimate bending moment capacity.

The Design procedure in MIDAS Software for PSC Section (T-Girder) is as follows:



GENERAL HYPOTHESES

3.1.1 Design Basis

The design of the T-Girder is carried out in accordance with the following documents:

Structural Design Basis Report

IRS Concrete Bridge Code 1997

The following software is used:

MIDAS for the structural Analysis

STAAD-PRO for the structural Analysis

<u>**Materials Parameters**</u>

Concrete characteristics for cast in situ slabs:-

Characteristic Concrete Strength : f_{ck} = 45 MPa (on cubic)

Young’s Modulus of concrete : E_i = 32500 MPa

Poisson’s Ratio of concrete : □ = 0.15

Coefficient of Thermal Expansion: □c = 1.17 10^{-5} /°C

Volumetric Weight : □ = 25 KN/m^3

<u>Concrete characteristics for Precast T-Girder</u>

Characteristic Concrete Strength: f_{ck} = 55 MPa (on cubic)

Young's Modulus of concrete: E_i = 35000 MPa

Poisson's Ratio of concrete: □ = 0.15

Coefficient of Thermal Expansion: □c = 1.17 10^{-5} /°C

Volumetric Weight: □ = 25 KN/m^3

<u>Reinforcement</u>

Grade of Reinforcement: σ_s = 500 MPa

Young's Modulus of Reinforcement : E_s = 200000 MPa

<u>Pre-stressing</u>

Pre-stressing steel will be conforming to IS: 14268, class 2 Low Relaxation uncoated stress relieved strands with the following characteristics:

<u>Pre-tensioning (Superstructure):</u>

Nominal Area of Strand : As = 140 mm^2

Nominal ultimate Stress : fpu = 1860 MPa

Maximum Jacking Stress: 0.75fpu = 1395MPa

Modulus of elasticity: Ep = 195000 MPa

Structure Description

The superstructure consists of Precast Pre-Tensioned T-Girder of 36.2m length, for span length of 37.0m. Bearing to bearing length distance is 35.2m. The plan view and cross-sectional view are as shown below.

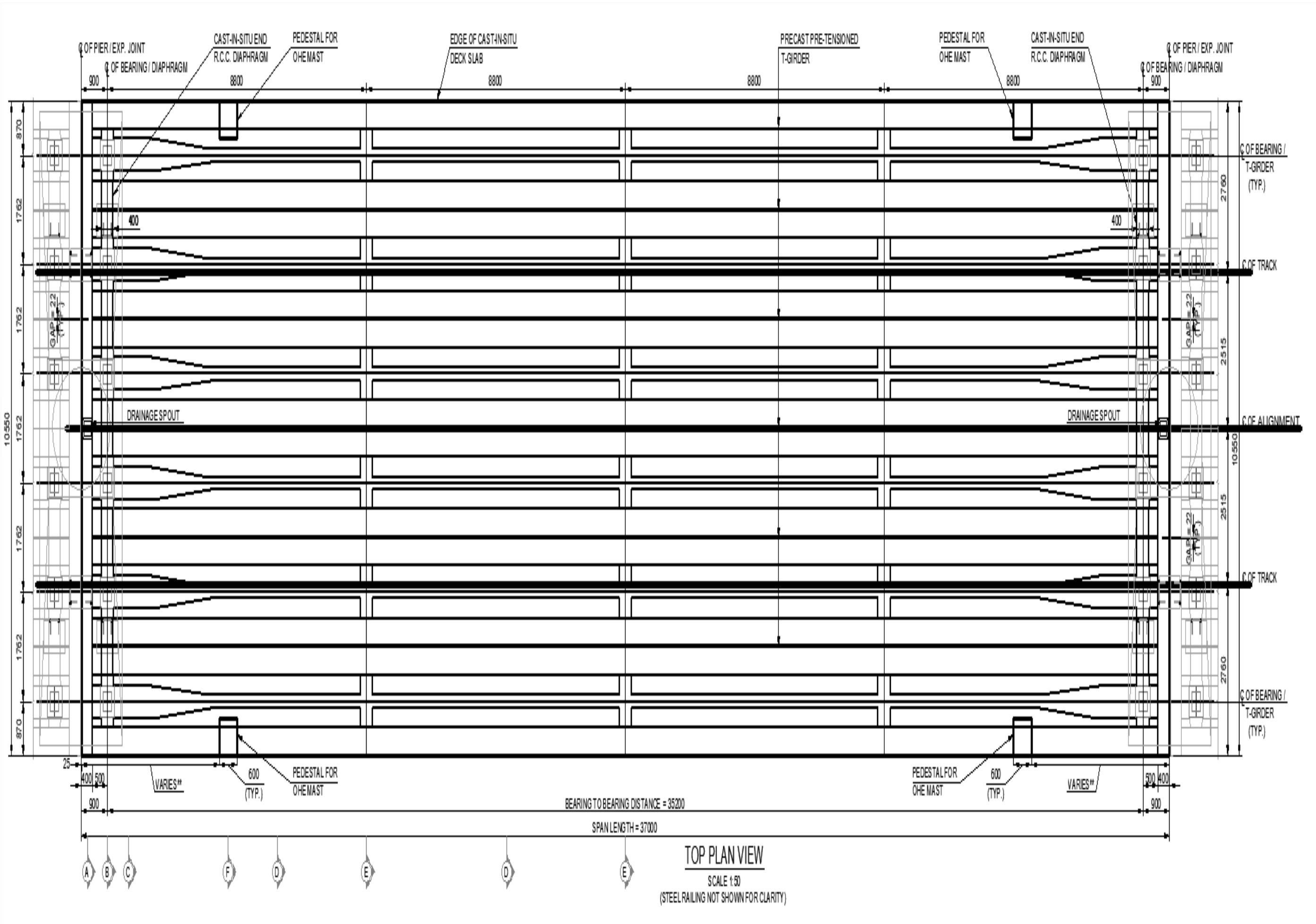


Fig 3.1 Plan view of 37m Span (6 – T Girder Straight Span)

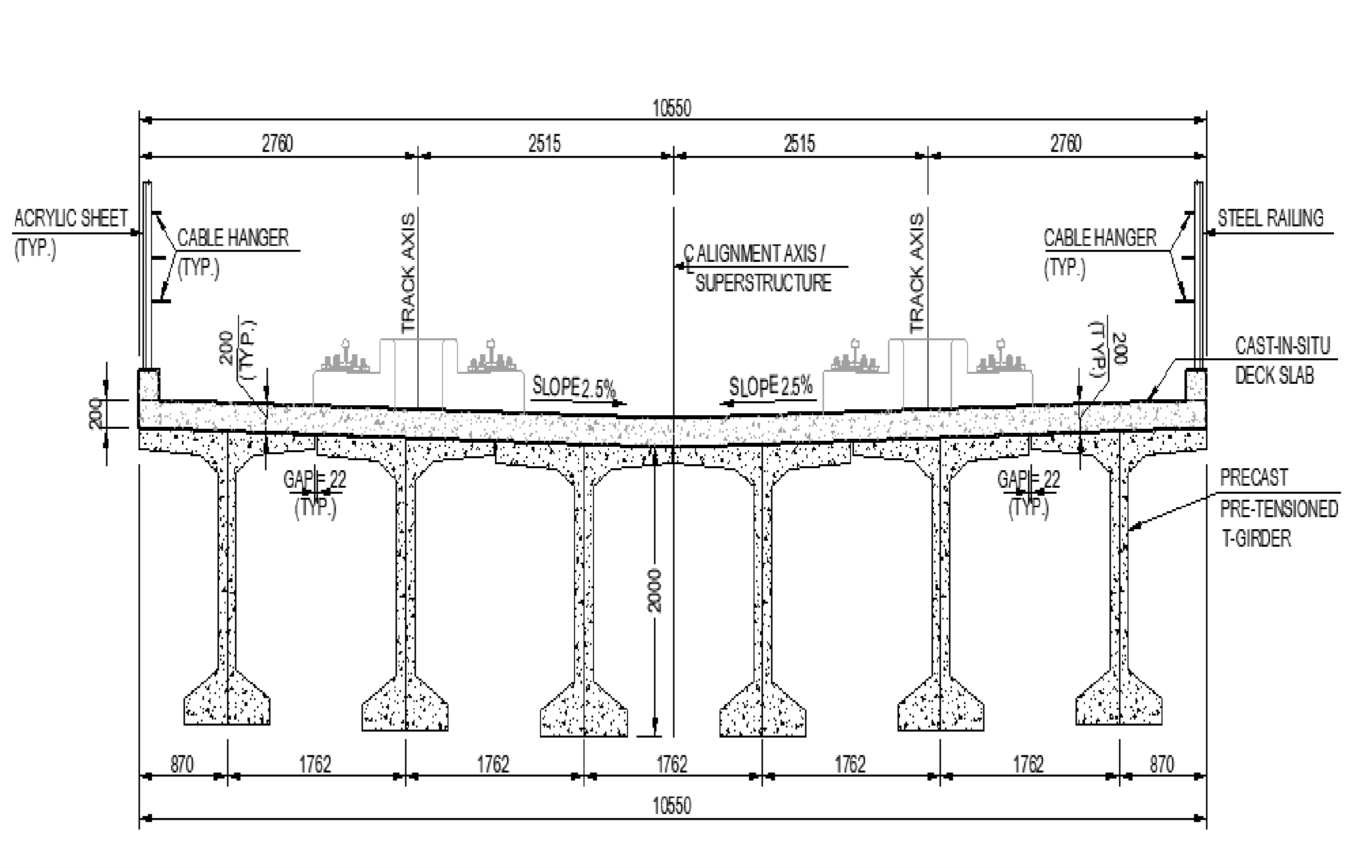


Fig 3.2 Cross Sectional View

MODEL SIMULATION OF T-GIRDER LONGITUDINALLY:

<u>Principles of Modeling</u>

The T-girder is modeled as a grillage model using MIDAS CIVIL 2020 (Ver 1.2) software. The exact Layout of Prestressing and exact sequences of construction are considered. View from MIDAS Software is as shown below.

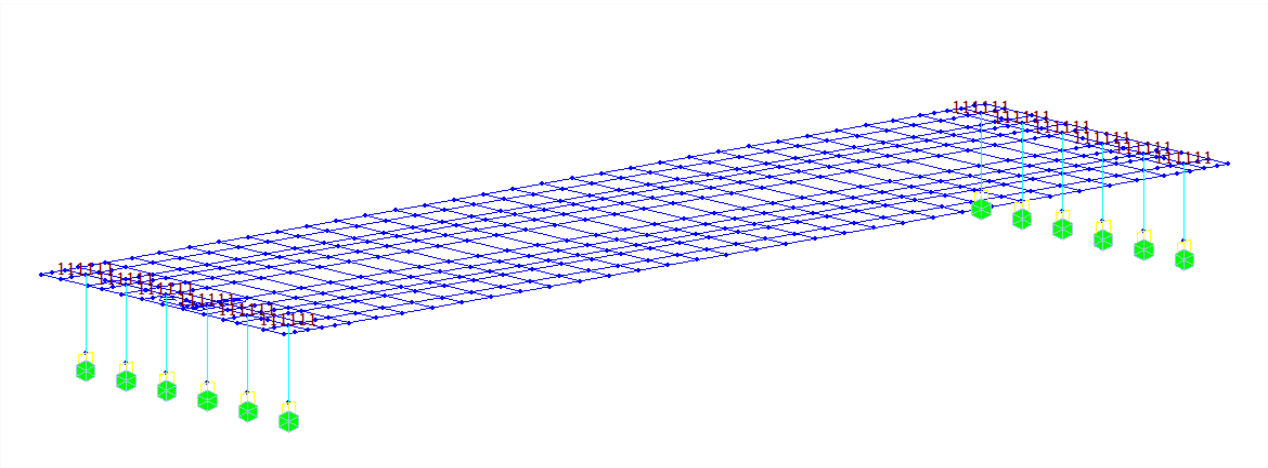


Fig 3.3 Grillage Model Showing Iso-Metric View

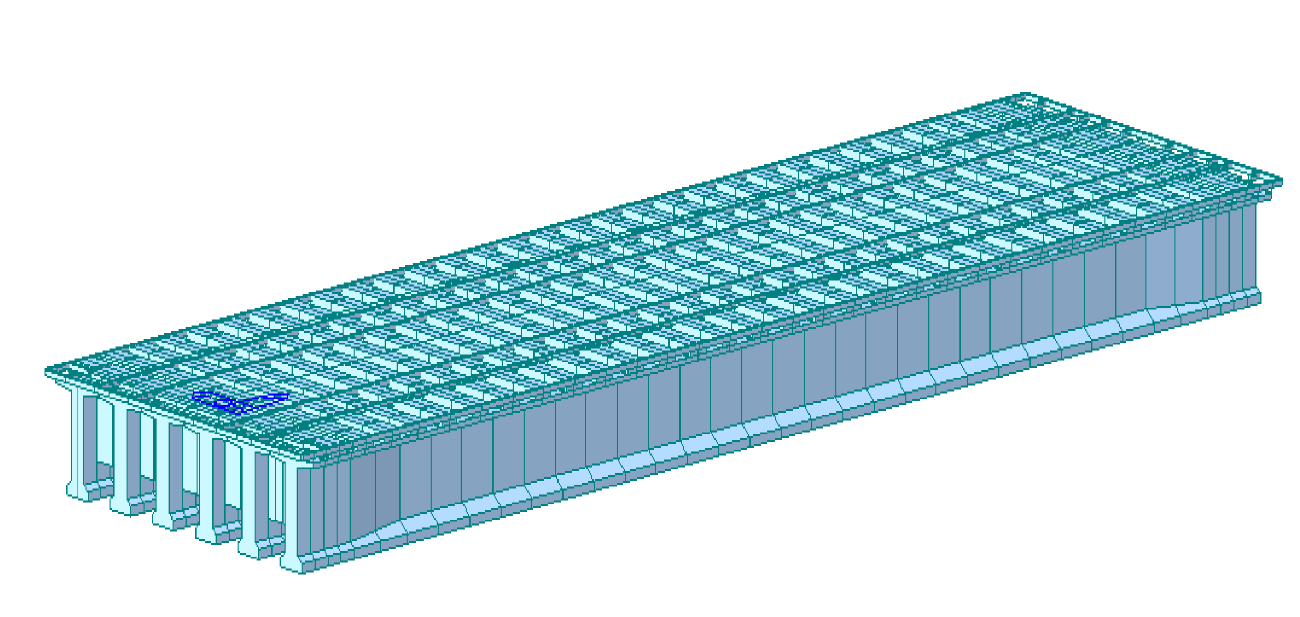


Fig3.4 Grillage Model Showing 3-D-Iso-Metric View

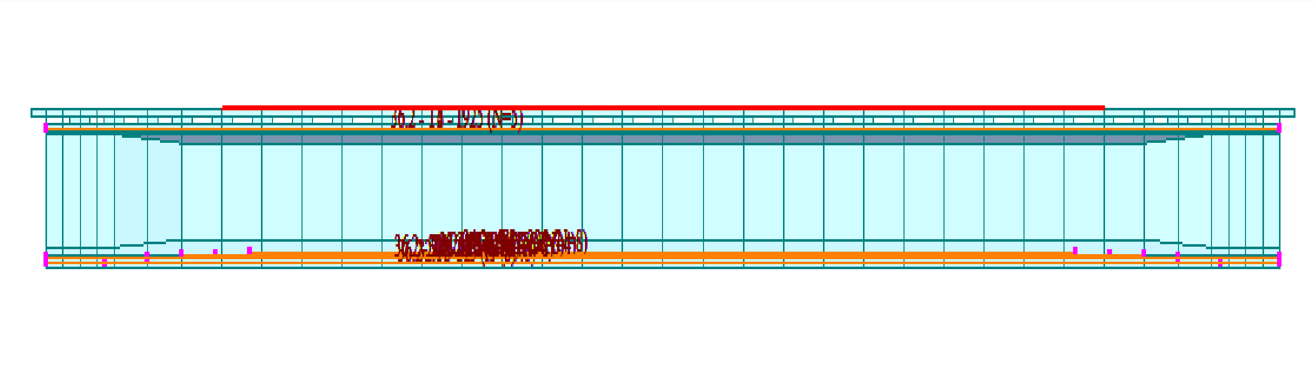


Fig 3.5 Showing Prestressing Cables

<u>Description of MIDAS Software:</u>

MIDAS is an Finite elements Method programme. The sofware generates the forces (BM, SF etc) at each section and combines them in accordance with the defined combination. To transmit the loads from one T-Girder to next T-Girder, cross-girder and slab elements are modelled in transverse direction. Bearing support is provided under each T-Girder to estimate the exact forces under each bearing.

All the loads (i.e. SIDL and Live Loads) are applied at their exact point of application with their correct magnitudes in order to have the actual reactions on each bearing, and also to have the actual behavior in longitudinal flexure of each T-Girder and Diaphragm.

Actual construction stages*:* Time variations of both topology and loading.

Effect of time on materials: Creep, Shrinkage of concrete and Prestressing losses (instantaneous and long term losses)

Main Input Data:

Material characteristics including time effects

Geometry of the structure during the different stages of the erection

Prestressing layout

External loading

Superimposed dead load Moving loads definition if any

Main Output Data:

Normal stress at top and bottom fibres

Forces

Shear stress

Displacements and reactions

Envelopes of all these results

MIDAS conventions are as follows:

My = Bending Moment (KN-m) Fz = Shear Force (KN)

Fx = Axial Force (KN)

Normal stress<0 = Compressive stress (MPa) Normal stress >0 = Tensile stress (MPa)

The input file of the MIDAS model and the listing of all loads, combinations, envelopes, and steps of construction used in the programme with their descriptions are given in Appendix 1.

Loads Applied in Modeling:

Dead Load

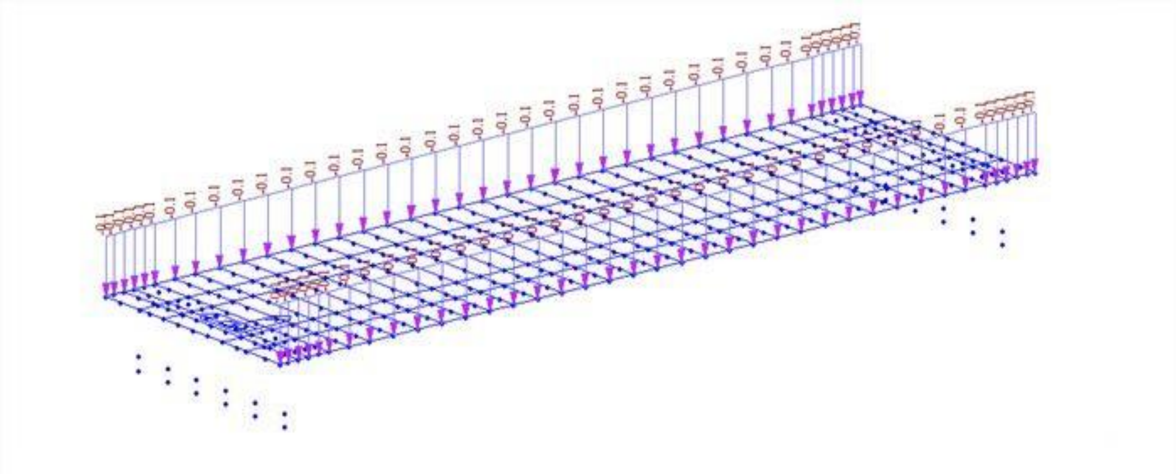
For assessment of dead load calculation, the following mass density has been considered :

Prestressed Concrete (PSC) : 25 KN/m^3= 2.55 T/m^3

For Midas model, Self Weight Command will automatically consider the effect due to dead loads.

3.3.1.1 Weight of concrete pedestal below railing

Load appied in midas at edge of deck = 0.102T/m



Super Imposed Dead Load

The following SIDL loads are applied as per OSD.

S.No.	- Element	Unfactored Load	Location
1	Parapet/Railing	0.2 t/m	end
2	Plinth	3.40 t/m	mid
3	Rail+Pads (All 4)	0.30 t/m	mid
4	Cables	0.07 t/m	end
5	Cable trays#	0.01 t/m	end
6	Deck drainage concrete (Avg. thk. 62.5mm)	0.24 t/m	mid
7	Miscll. (OHE Mast, Signalling , etc.)	0.40 t/m	end
8	Solar Panel (wherever applicable)	30kg/sqm (0.092 t/m)	end
9	Noise Barrier (wherever applicable)	0.2 t/m	end
10	PTM Pipe Line	0.06t/m	end
	Sum of Load applied at Plinth location	3.94 t/m	mid
	Sum of Load applied at edge of deck	1.039 t/m	end

The application of total SIDL is as explained below : -

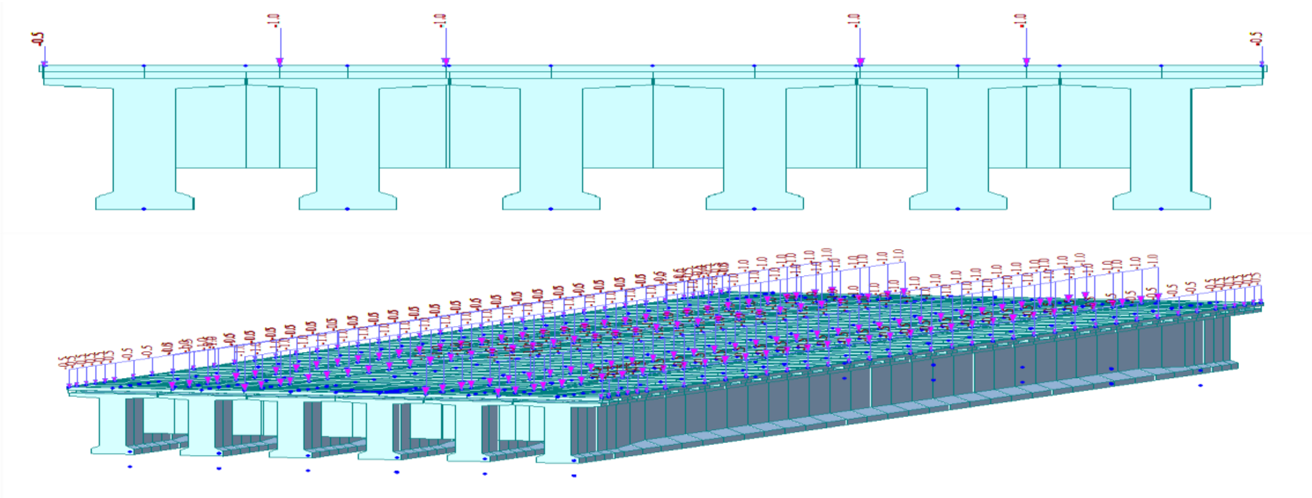
Load appied in midas model per Plinth = 3.94/4 = 0.985 T/m

Say = 1.0 T/m

Load appied in midas at edge of deck = 1.039/2 = 0.516 T/m

Say = 0.52 T/m

.

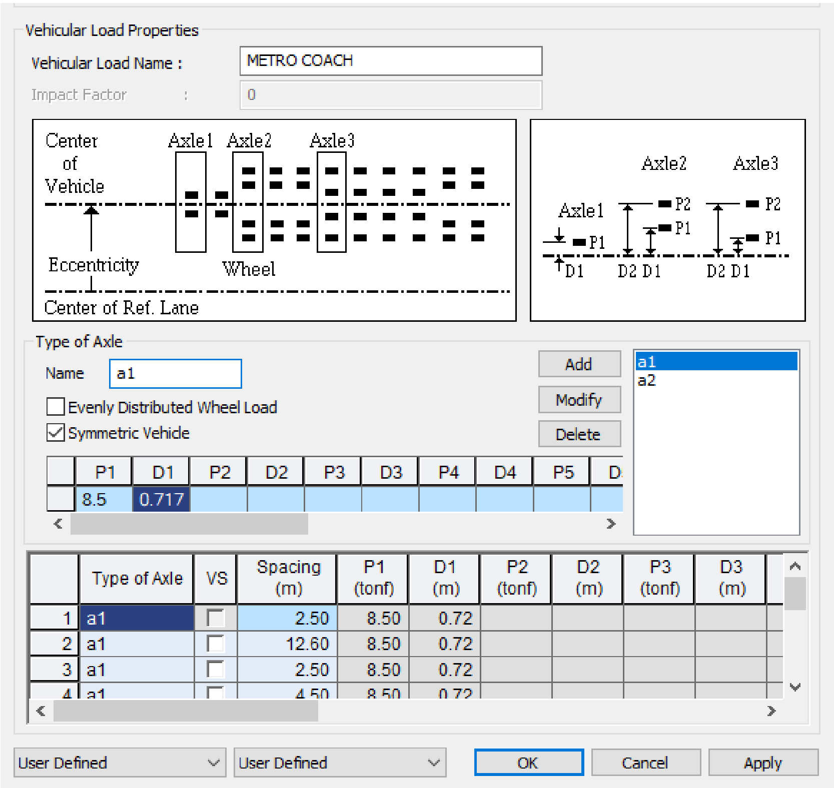


MODEL SHOWING APPLICATION OF SIDL

The following SIDL loads are applied as per OSD

Live Load Vehicle

3.3.3.1 Coefficient of Dynamic Impact (CDA)



The Impact factor on train live load for longitudinal analysis shall be 1.336 as per OSD clause 6.4.2

Wind Load

Vertical wind load on super-structure				
Hourly mean wind speed	Vertical wind pressure as per IRC-6 Table 12 corresponding to 25m Height	Pz	1309.46	N/m^2
Gust factor		G	2	
Lift coefficient		C_L	0.75	
Vertical Wind Pressure on deck			1.964	KN/m^2
Vertical Wind Load on each T-Girder (e.g. Pressure x 10.55 / 6 Nos.)			3.454	KN/m

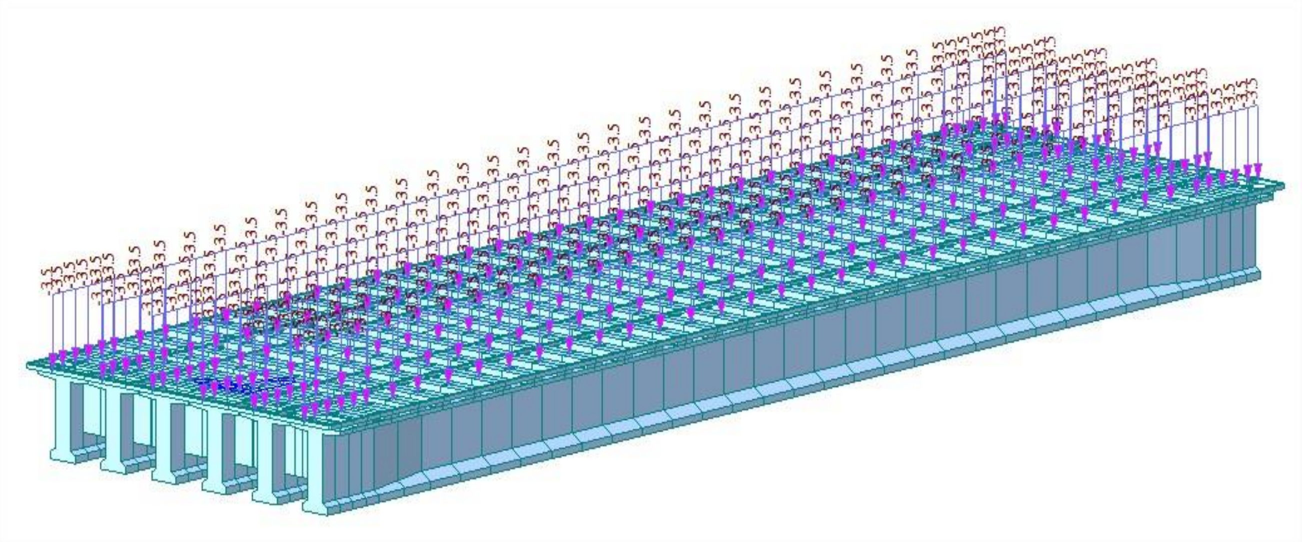


Fig_3.5 Model Showing Application Of Wind Load

Seismic Load

VERTICAL SEISMIC
SEISMIC COEFFICIENT FOR VERTICAL SEISMIC

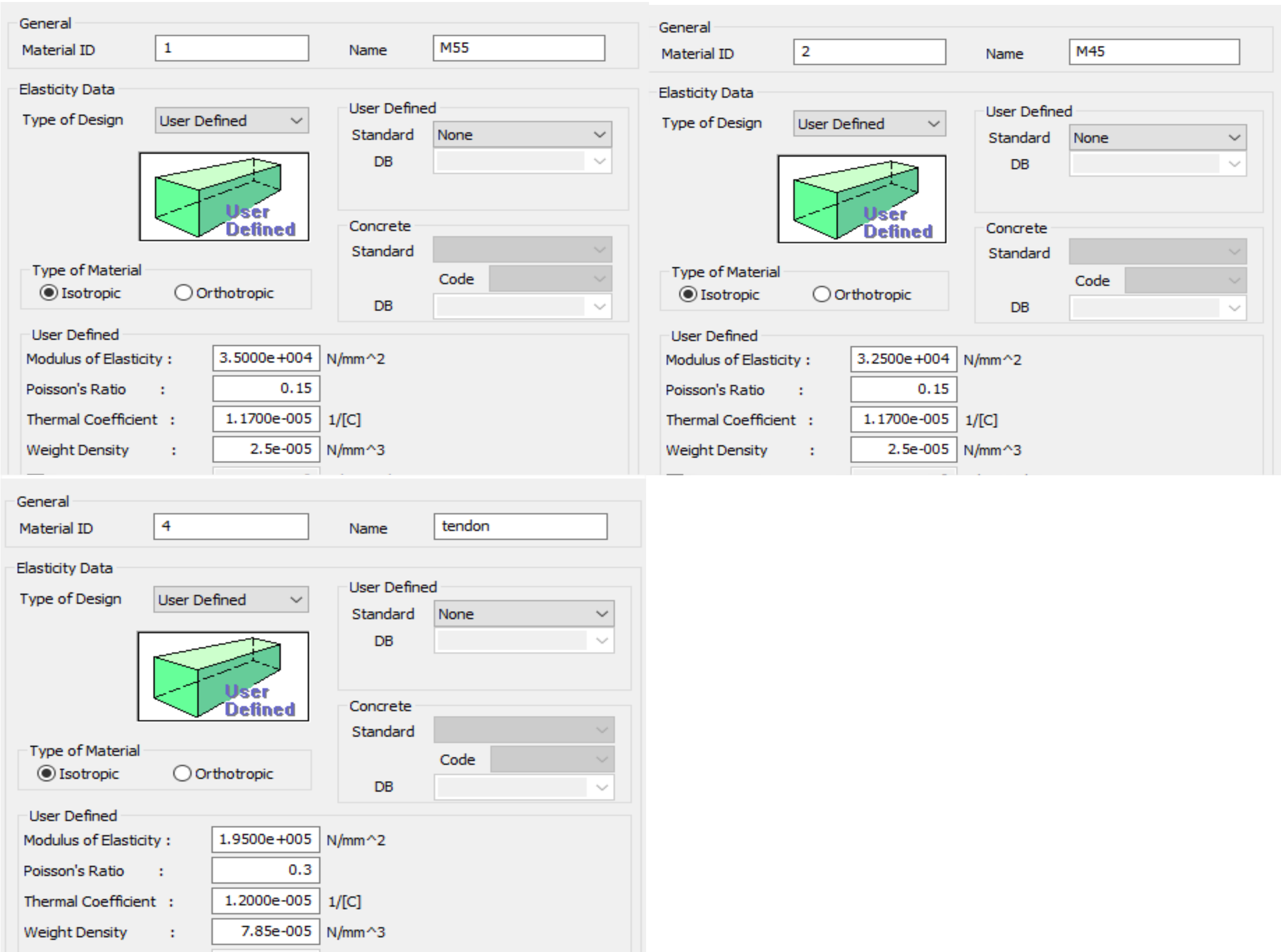
ACCORDING TO $T_v = \frac{2}{\pi} l^2 \sqrt{\frac{m}{EI}}$

Z=	0.16
I=	1.50
R=	1.0
Sa/g=	2.500
Ah=	0.300

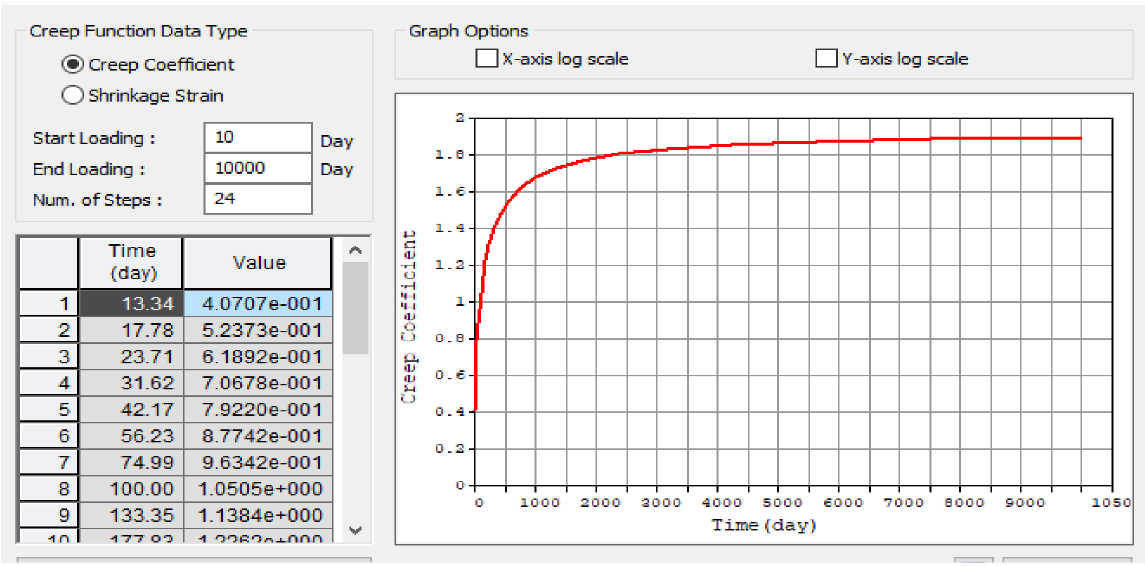
Seismic load is taken as : - 0.3 x (Dead Load + SIDL + 50% Live Load)

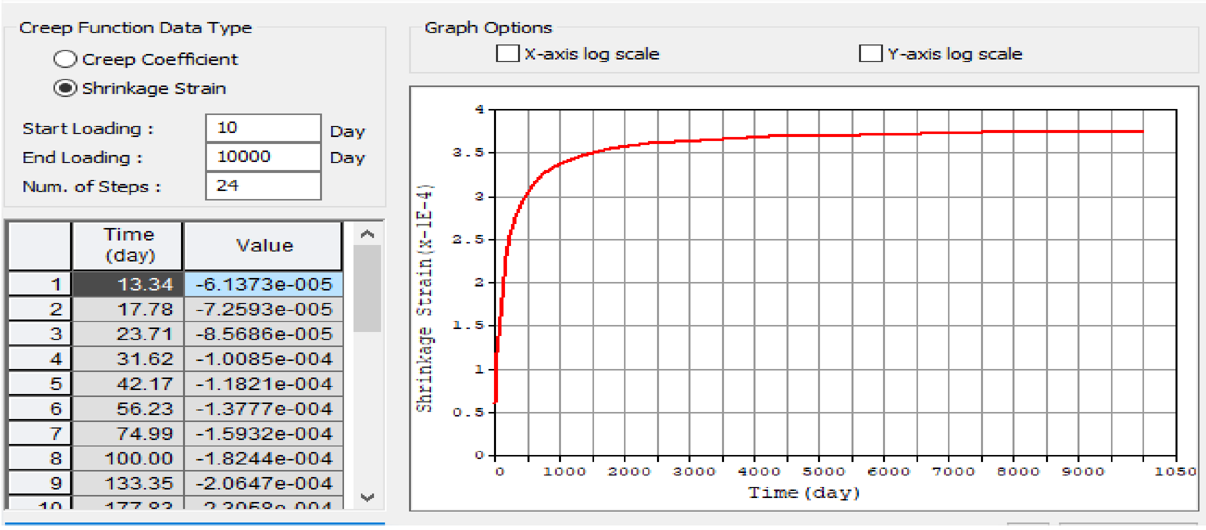
Midas Input

In this Section screen shots of MIDAS Input is presented
Material Properties

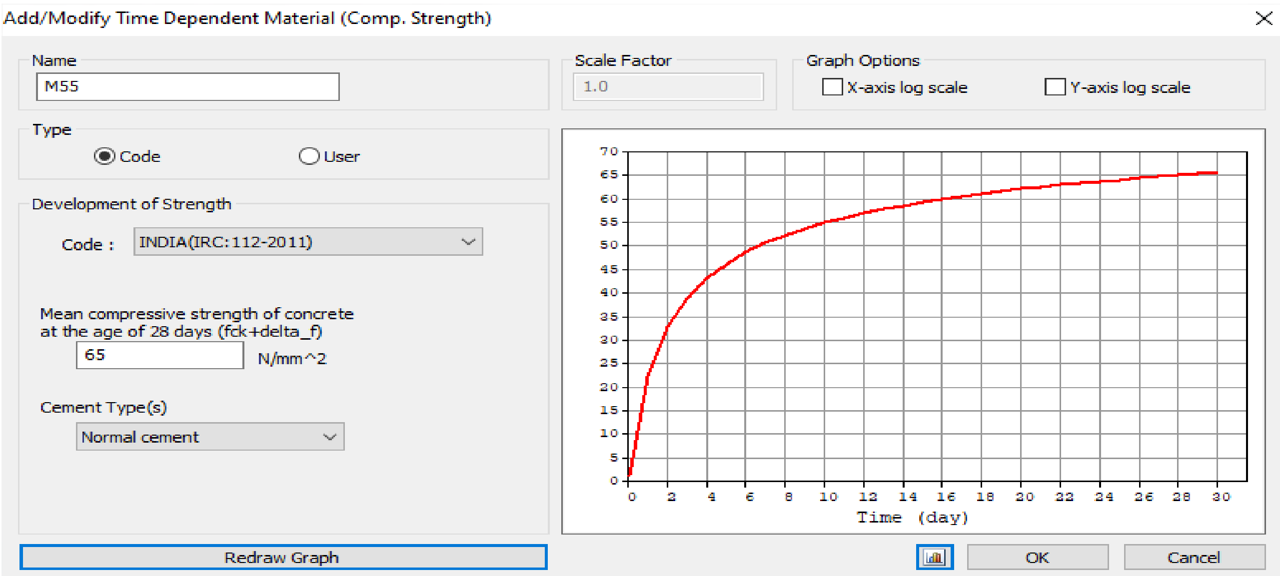


Time dependent material (Creep & Shrinkage):





Time dependent material (Compressive strength):



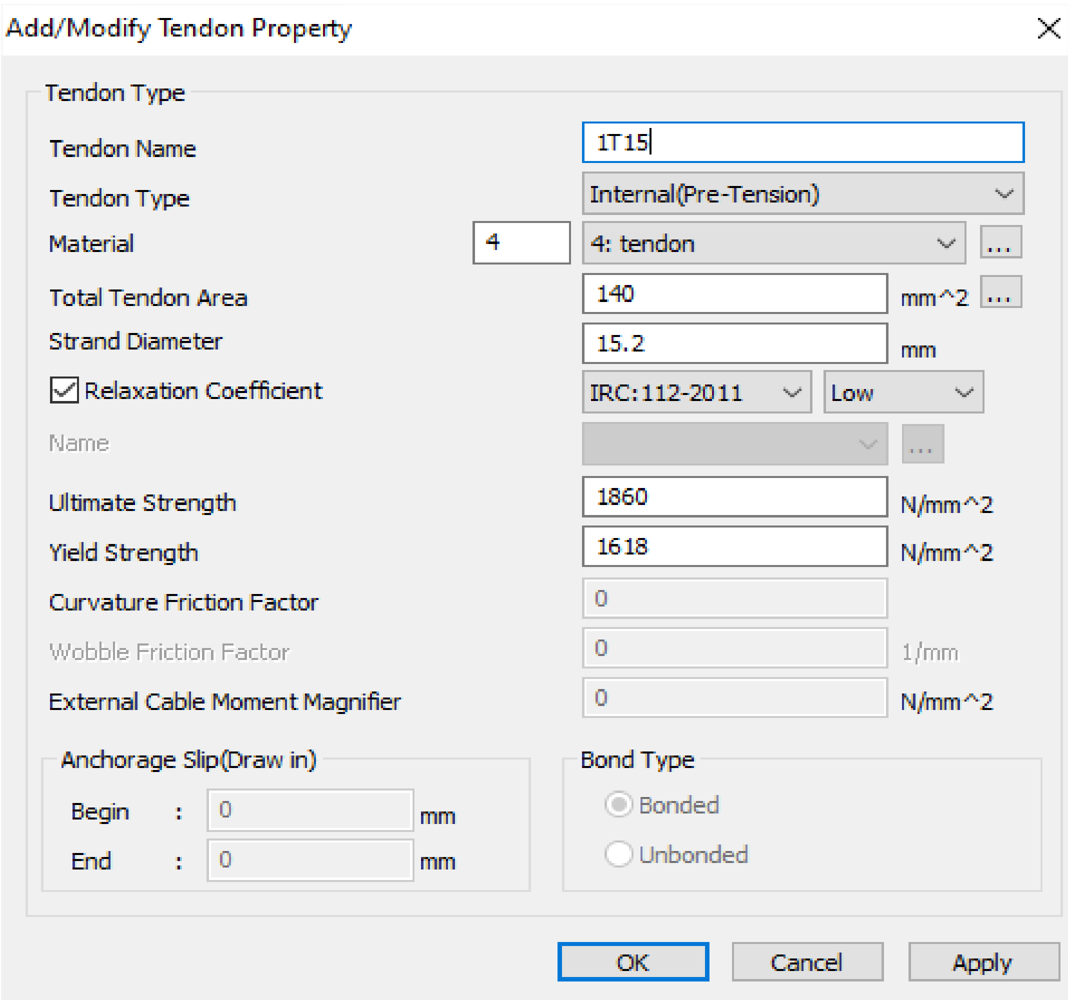
Boundary Conditions

Elastic Link Node Details

	Node	Dx	Dy	Dz	Rx	Ry	Rz	Rw	Group
	1	1	1	1	1	1	1	1	support
	367	1	1	1	1	1	1	1	support
	369	1	1	1	1	1	1	1	support
	371	1	1	1	1	1	1	1	support
	373	1	1	1	1	1	1	1	support
	375	1	1	1	1	1	1	1	support
	552	1	1	1	1	1	1	0	support
	554	1	1	1	1	1	1	0	support
	556	1	1	1	1	1	1	0	support
	558	1	1	1	1	1	1	0	support
	560	1	1	1	1	1	1	0	support
	562	1	1	1	1	1	1	0	support
*									

	No	Node1	Node2	Type	B Angle ([deg])	RIGID	SDx (N/mm)	SDy (N/mm)	SDz (N/mm)	SRx (N*mm/[rad])	SRy (N*mm/[rad])	SRz (N*mm/[rad])	Shear Spring Location	Distance Ratio SDy	Distance Ratio SDz	Group
	1	3	1	GEN	0.00	000000	1000000.0	1000000.0	1000000.0	0.00	0.00	0.00	☐	0.50	0.50	link 2
	2	368	367	GEN	0.00	000000	1000000.0	0.0000	1000000.0	0.00	0.00	0.00	☐	0.50	0.50	link 2
	3	370	369	GEN	0.00	000000	1000000.0	0.0000	1000000.0	0.00	0.00	0.00	☐	0.50	0.50	link 2
	4	372	371	GEN	0.00	000000	1000000.0	0.0000	1000000.0	0.00	0.00	0.00	☐	0.50	0.50	link 2
	5	374	373	GEN	0.00	000000	1000000.0	0.0000	1000000.0	0.00	0.00	0.00	☐	0.50	0.50	link 2
	6	376	375	GEN	0.00	000000	1000000.0	0.0000	1000000.0	0.00	0.00	0.00	☐	0.50	0.50	link 2
	7	553	552	GEN	0.00	000000	1000000.0	1000000.0	0.0000	0.00	0.00	0.00	☐	0.50	0.50	link 2
	8	559	558	GEN	0.00	000000	1000000.0	0.0000	0.0000	0.00	0.00	0.00	☐	0.50	0.50	link 2
	9	557	556	GEN	0.00	000000	1000000.0	0.0000	0.0000	0.00	0.00	0.00	☐	0.50	0.50	link 2
	10	555	554	GEN	0.00	000000	1000000.0	0.0000	0.0000	0.00	0.00	0.00	☐	0.50	0.50	link 2
	11	561	560	GEN	0.00	000000	1000000.0	0.0000	0.0000	0.00	0.00	0.00	☐	0.50	0.50	link 2
	12	563	562	GEN	0.00	000000	1000000.0	0.0000	0.0000	0.00	0.00	0.00	☐	0.50	0.50	link 2
*													☐			

Prestressing Tendon details

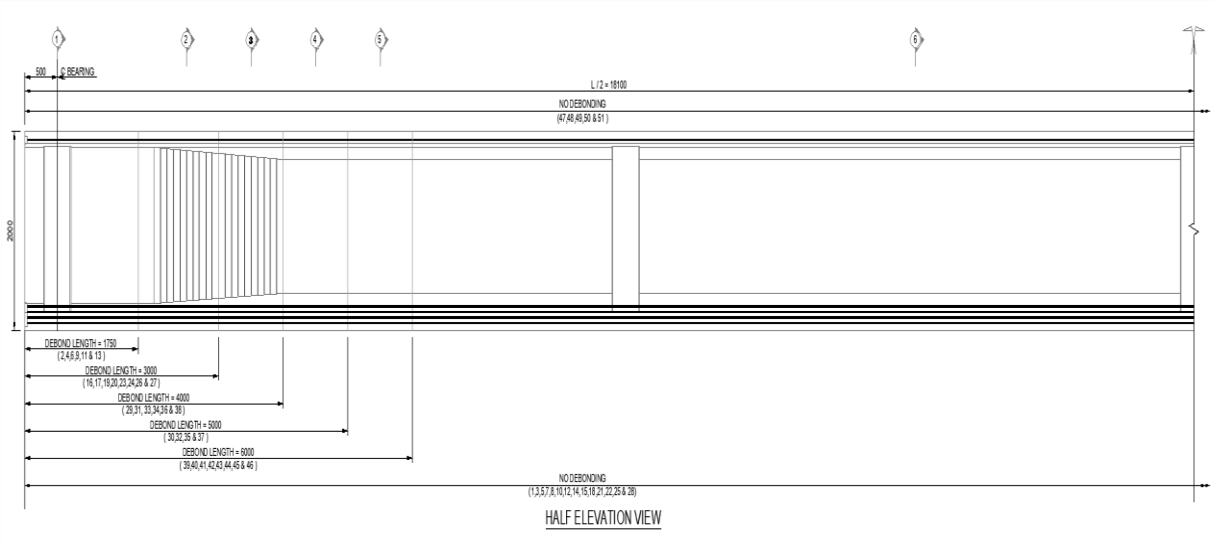


Tendon Prestress Loads

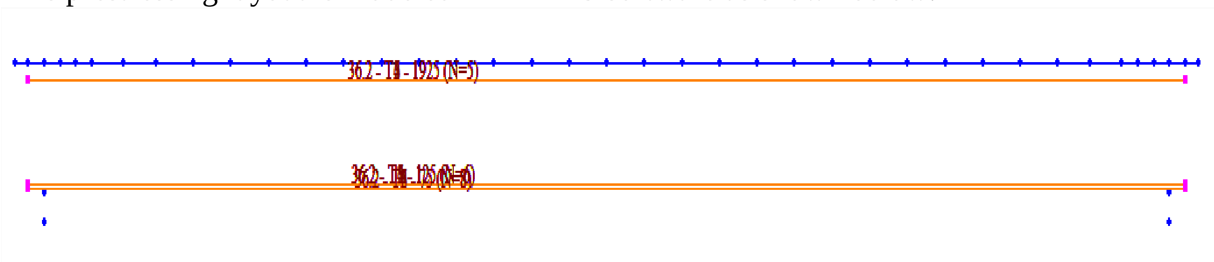
Tendon	Load Case	Type	Jacking	Stress Begin (N/mm^2)	Stress End (N/mm^2)	Force Begin (N)	Force End (N)	Grouting	Load Group
24.2 - T1 - 225	PREST	Stre	Both	1395.00	1395.00	0.00	0.00	0	PT
24.2 - T2 - 225	PREST	Stre	Both	1395.00	1395.00	0.00	0.00	0	PT
24.2 - T3 - 225	PREST	Stre	Both	1395.00	1395.00	0.00	0.00	0	PT
24.2 - T4 - 225	PREST	Stre	Both	1395.00	1395.00	0.00	0.00	0	PT
24.2 - T5 - 225	PREST	Stre	Both	1395.00	1395.00	0.00	0.00	0	PT
24.2 - T6 - 225	PREST	Stre	Both	1395.00	1395.00	0.00	0.00	0	PT
26.2 - T1 - 175	PREST	Stre	Both	1395.00	1395.00	0.00	0.00	0	PT
26.2 - T2 - 175	PREST	Stre	Both	1395.00	1395.00	0.00	0.00	0	PT
26.2 - T3 - 175	PREST	Stre	Both	1395.00	1395.00	0.00	0.00	0	PT
26.2 - T4 - 175	PREST	Stre	Both	1395.00	1395.00	0.00	0.00	0	PT
26.2 - T5 - 175	PREST	Stre	Both	1395.00	1395.00	0.00	0.00	0	PT
26.2 - T6 - 175	PREST	Stre	Both	1395.00	1395.00	0.00	0.00	0	PT
28.2 - T1 - 175	PREST	Stre	Both	1395.00	1395.00	0.00	0.00	0	PT
28.2 - T2 - 175	PREST	Stre	Both	1395.00	1395.00	0.00	0.00	0	PT
28.2 - T3 - 175	PREST	Stre	Both	1395.00	1395.00	0.00	0.00	0	PT
28.2 - T4 - 175	PREST	Stre	Both	1395.00	1395.00	0.00	0.00	0	PT
28.2 - T5 - 175	PREST	Stre	Both	1395.00	1395.00	0.00	0.00	0	PT
28.2 - T6 - 175	PREST	Stre	Both	1395.00	1395.00	0.00	0.00	0	PT
30.2 - T1 - 125	PREST	Stre	Both	1395.00	1395.00	0.00	0.00	0	PT
30.2 - T2 - 125	PREST	Stre	Both	1395.00	1395.00	0.00	0.00	0	PT
30.2 - T3 - 125	PREST	Stre	Both	1395.00	1395.00	0.00	0.00	0	PT
30.2 - T4 - 125	PREST	Stre	Both	1395.00	1395.00	0.00	0.00	0	PT
30.2 - T5 - 125	PREST	Stre	Both	1395.00	1395.00	0.00	0.00	0	PT
30.2 - T6 - 125	PREST	Stre	Both	1395.00	1395.00	0.00	0.00	0	PT
32.7 - T1 - 75	PREST	Stre	Both	1395.00	1395.00	0.00	0.00	0	PT
32.7 - T2 - 75	PREST	Stre	Both	1395.00	1395.00	0.00	0.00	0	PT
32.7 - T3 - 75	PREST	Stre	Both	1395.00	1395.00	0.00	0.00	0	PT
32.7 - T4 - 75	PREST	Stre	Both	1395.00	1395.00	0.00	0.00	0	PT
32.7 - T5 - 75	PREST	Stre	Both	1395.00	1395.00	0.00	0.00	0	PT
32.7 - T6 - 75	PREST	Stre	Both	1395.00	1395.00	0.00	0.00	0	PT
36.2 - T1 - 125	PREST	Stre	Both	1395.00	1395.00	0.00	0.00	0	PT
36.2 - T1 - 1925	PREST	Stre	Both	1395.00	1395.00	0.00	0.00	0	PT
36.2 - T1 - 75	PREST	Stre	Both	1395.00	1395.00	0.00	0.00	0	PT
36.2 - T2 - 125	PREST	Stre	Both	1395.00	1395.00	0.00	0.00	0	PT
36.2 - T2 - 1925	PREST	Stre	Both	1395.00	1395.00	0.00	0.00	0	PT
36.2 - T2 - 75	PREST	Stre	Both	1395.00	1395.00	0.00	0.00	0	PT
36.2 - T3 - 125	PREST	Stre	Both	1395.00	1395.00	0.00	0.00	0	PT
36.2 - T3 - 1925	PREST	Stre	Both	1395.00	1395.00	0.00	0.00	0	PT
36.2 - T3 - 75	PREST	Stre	Both	1395.00	1395.00	0.00	0.00	0	PT
36.2 - T4 - 125	PREST	Stre	Both	1395.00	1395.00	0.00	0.00	0	PT
36.2 - T4 - 1925	PREST	Stre	Both	1395.00	1395.00	0.00	0.00	0	PT
36.2 - T4 - 75	PREST	Stre	Both	1395.00	1395.00	0.00	0.00	0	PT
36.2 - T5 - 125	PREST	Stre	Both	1395.00	1395.00	0.00	0.00	0	PT
36.2 - T5 - 1925	PREST	Stre	Both	1395.00	1395.00	0.00	0.00	0	PT
36.2 - T5 - 75	PREST	Stre	Both	1395.00	1395.00	0.00	0.00	0	PT
36.2 - T6 - 125	PREST	Stre	Both	1395.00	1395.00	0.00	0.00	0	PT
36.2 - T6 - 1925	PREST	Stre	Both	1395.00	1395.00	0.00	0.00	0	PT
36.2 - T6 - 75	PREST	Stre	Both	1395.00	1395.00	0.00	0.00	0	PT

Prestressing Layout of T-Girder :-

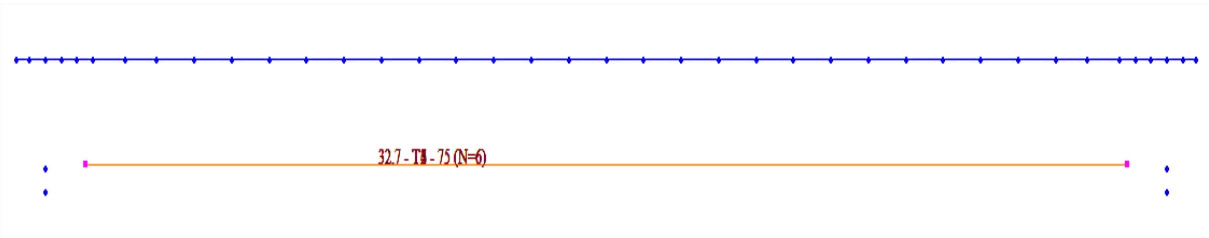
The prestressing layout for T-Girder is as shown below.



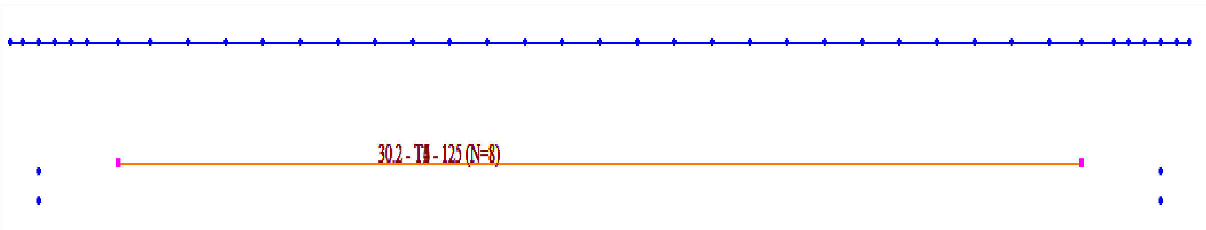
This prestressing layout is modeled in MIDAS software as shown below:



Cable Profile Of No Debonding

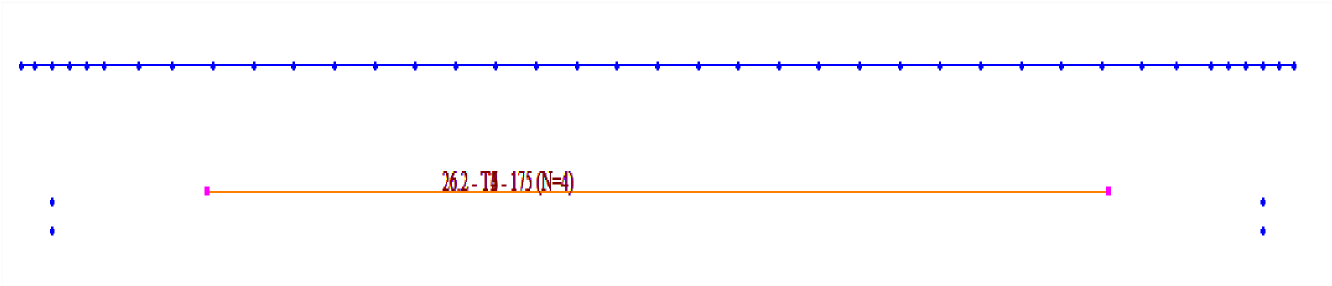


Cable Profile Of 1.75m Debonding

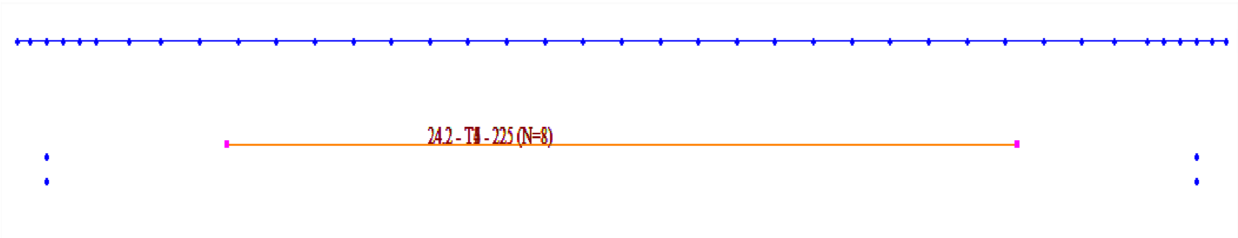


Cable Profile Of 3.0m Debonding

Cable Profile Of 4.0m Debonding



Cable Profile Of 5.0m Debonding



Cable Profile Of 6.0m Debonding

Construction Sequence

The followings are the construction phases which are considered

Critical Construction Phases

Phase: 1 : Self Weight of T-Girder + PT in Casting Yard

Phase: 2 : Application of slab wt. as green concrete

Phase: 3 : Slab hardens

Phase: 4 : Applied Super Imposed Dead load

:

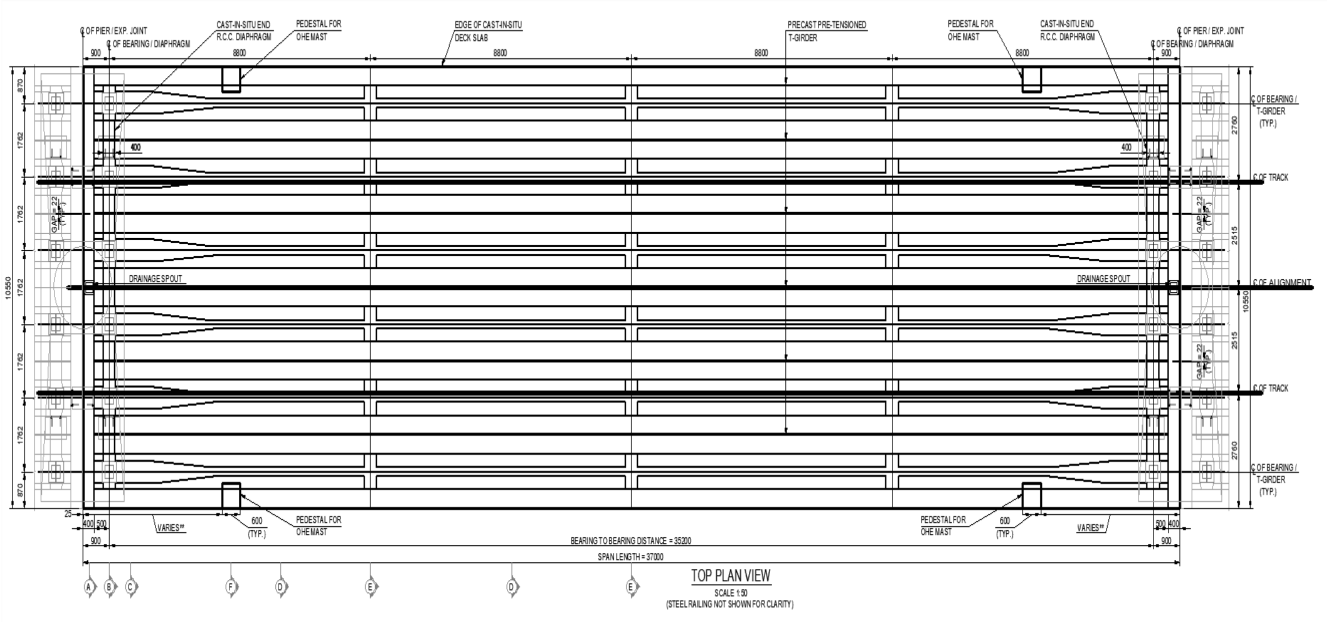
Phase: 5 : Short term at 1 day

Phase: 6 : Long term at 36500 days

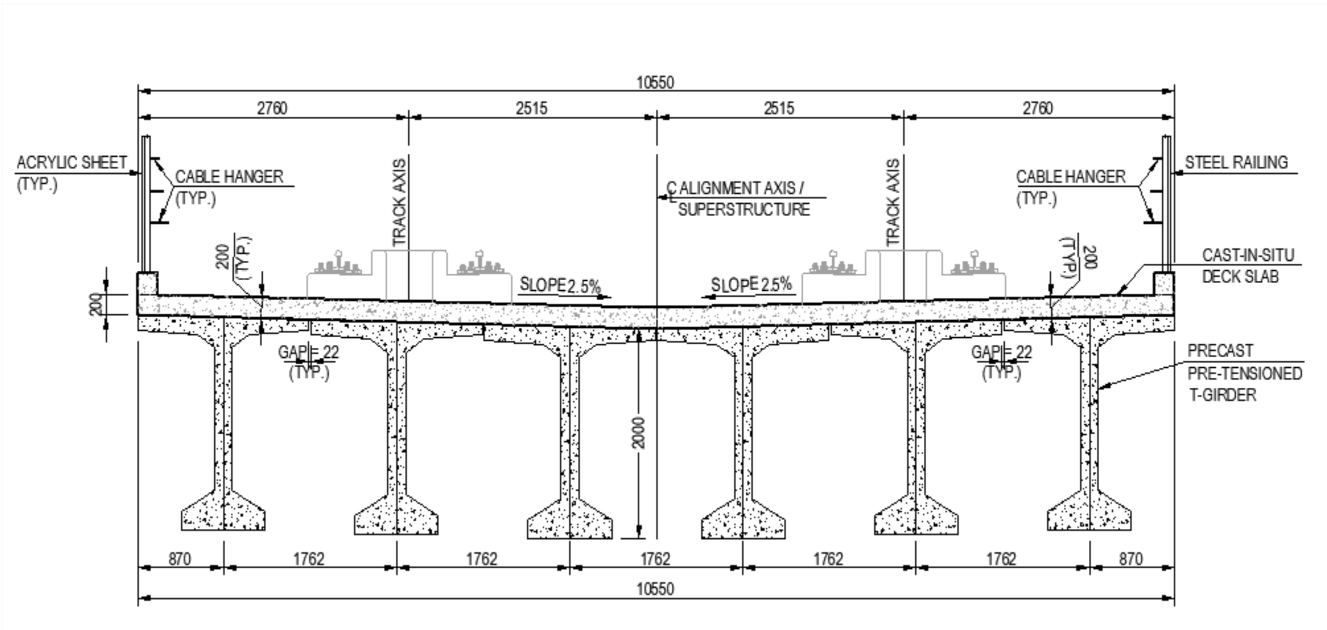
MODEL SIMULATION OF DECK SLAB OF T-GIRDER TRANSVERSELY

3.7.1 Structure Description

The superstructure consists of Pre cast Pre-Tensioned T-Girder of 36.2m length, for span length of 37.0m. Bearing to bearing length distance is 35.2m. The plan view and cross-sectional view are as shown below.



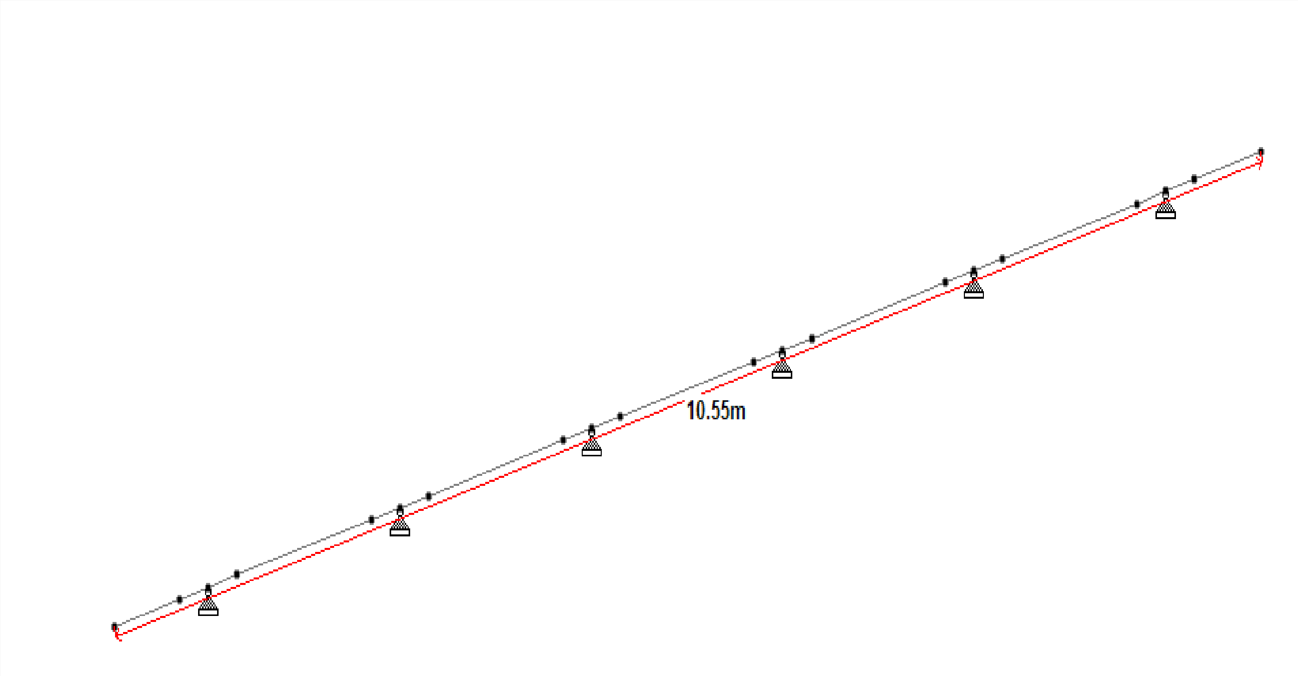
Plan view of 37m Span (6 – T Girder Straight Span)



Cross Sectional View

Description of STAAD Model

STAAD MODEL (WIDTHS = 10.55m, THICKNESS = 0.2m, LENGTHS = 1.0m)



External Loads Applied on Modeling (with-OHE)

For assessment of dead load calculation, the following mass density has been considered:-

Concrete = 25 KN/m^3

Super Imposed Dead Load

SIDL distribution for two tracks is taken from Design Basis Report Section 2.10:-

Parapet (Self Weight)+OHE.	=	6.54	t/m
Plinth	=	3.4	t/m
Rail + Pads (All 4)	=	0.3	t/m
Cables	=	0.07	t/m
Cable Trays	=	0.01	t/m
Deck Drainage	=	0.24	t/m
Miscll.	=	0.4	t/m
Solar Panel	=	0.092	t/m
Noise Barrier	=	0.2	t/m
PTM Pipe Line	=	0.06	t/m
Total SIDL		**11.310**	**t/m**

Concrete Plinth

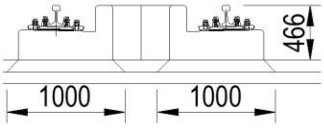
Standard distribution for 2 tracks is given below:- Rails + Pads = 0.3 T/m

Concrete Plinths = 3.4 T/m

Total Plinth Load = 3.7 T/m

For 2 tracks, this load will be proportionately increased- Final Plinth Load = 3.7 T/m

The combined load of "Rails+Pads" and "Concrete Plinths" applied as the UDL in the ***dispersion width*** as shown below:



Hence, Dispersion Width of 1 Plinth = 1.00 m

Total Dispersion Width of **4** rails along Slab in Transverse direction & UDL applied on this width is shown below:

Total Dispersion width = 4 m

Total Plinth UDL = 0.925 T/m/per meter length of Slab

Parapet

Parapet (Self Weight) 6.54 T/m

Cables 0.07 T/m

Cable Trays 0.01 T/m

Miscll. 0.4 T/m

Solar Panel 0.092

Noise Barrier 0.2

PTM Pipe Line 0.06

Total Parapet Load = 7.370 T/m

The combined load will be applied on the edge of Slab as:

Uniformly Distributed Load in the long direction -

P_L = 3.6849 T/m

Uniformly Distributed Moment in the long direction -

Lever Arm = 0.202 m

(distance between CG of Parapet and edge of Slab)

M_L = 0.744 T-m/per meter length of Slab

Drainage

Deck Drainage Concrete for Standard width of Deck (i.e. for 10.55 m) = 0.24 T/m For 10.550 m width of Deck, this load will be proportionately increased-

Deck Drainage Concrete Load for this width = 0.240 T/m

The load of "Deck Drainage Concrete" is applied as the UDL along the width of Deck Slab in the Transverse direction.

Total Deck Drainage UDL = 0.0227 T/m/per meter length of Slab

Hence,

Total SIDL = 11.310 T/m

External Loads Applied on Modeling (without - OHE)

Dead Load

For assessment of dead load calculation, the following mass density has been considered:-

Concrete =25 KN/m^3

Super Imposed Dead Load

SIDL distribution for two tracks is taken from Design Basis Report Section 2.10:- Parapet (Self Weight) = 0.20 t/m

Plinth = 3.4 t/m

Rail + Pads (All 4) = 0.3 t/m

Cables = 0.07 t/m

Cable Trays = 0.01 t/m

Deck Drainage = 0.24 t/m

Miscll. = 0.4 t/m

Solar Panel = 0.092 t/m

Noise Barrier = 0.2 t/m

PTM Pipe Line = 0.06 t/m

Total SIDL 4.972 t/m

Concrete Plinth

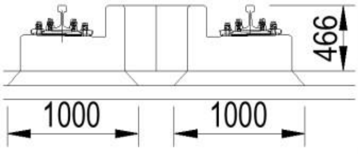
Standard distribution for 2 tracks is given below:- Rails + Pads = 0.3 T/m

Concrete Plinths = 3.4 T/m

Total Plinth Load = 3.7 T/m

For 2 tracks, this load will be proportionately increased- Final Plinth Load = 3.7 T/m

The combined load of "Rails+Pads" and "Concrete Plinths" applied as the UDL in the ***dispersion width*** as shown below:

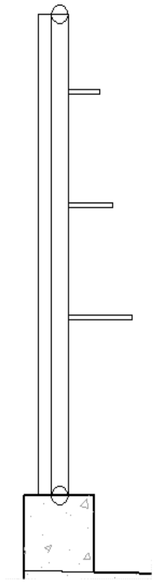


Hence, Dispersion Width of 1 Plinth = 1.00 m

Total Dispersion Width of rails along Slab in Transverse direction & UDL applied on this width is shown below: Total Dispersion width = 4 m

Total Plinth UDL = 0.925 T/m/per meter length of Slab

Parapet



Parapet (Self Weight)	0.20 T/m
Cables	0.07 T/m
Cable Trays	0.01 T/m
Miscll.	0.4 T/m
Solar Panel	0.092
Noise Barrier	0.2
PTM Pipe Line	0.06
Total Parapet Load =	1.032 T/m

The combined load will be applied on the edge of Slab as:

Uniformly Distributed Load in the long direction -

P_L =0.516 T/m

Uniformly Distributed Moment in the long direction -

Lever Arm = 0.1 m

(distance between CG of Parapet and edge of Slab)

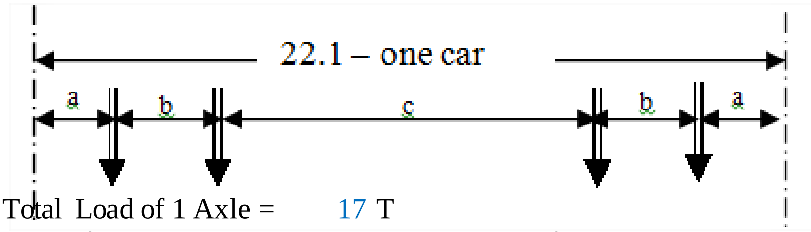
M_L = 0.052 T-m/per meter length of Slab

<u>STAAD MODEL SHOWING APPLICATION OF SIDL</u>

3.10 Live Load

The Train Live Load for this Line will have the following axle configuration (Trailer/Motor Car):

Lo



Total Load of 1 Axle = 17 T

Load of 1 Wheel = 8.5 T Maximum number of successive cars is 6 Configuration:

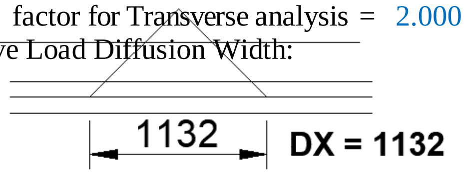
a = 2.25 m

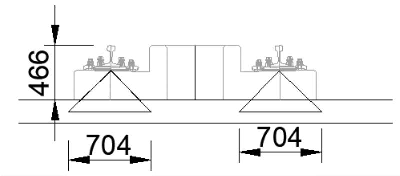
b = 2.50 m

c = 12.60 m (2a+2b+c=22.1)

Impact factor for Transverse analysis = 2.000

Live Load Diffusion Width:



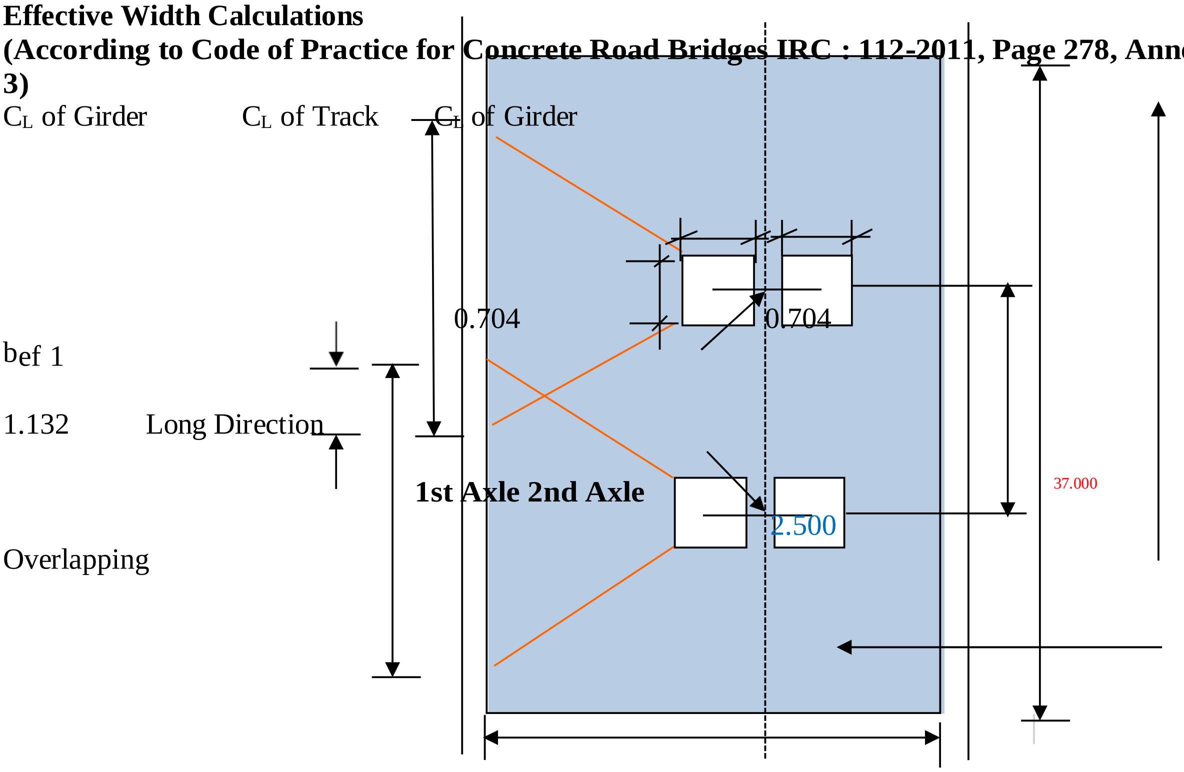


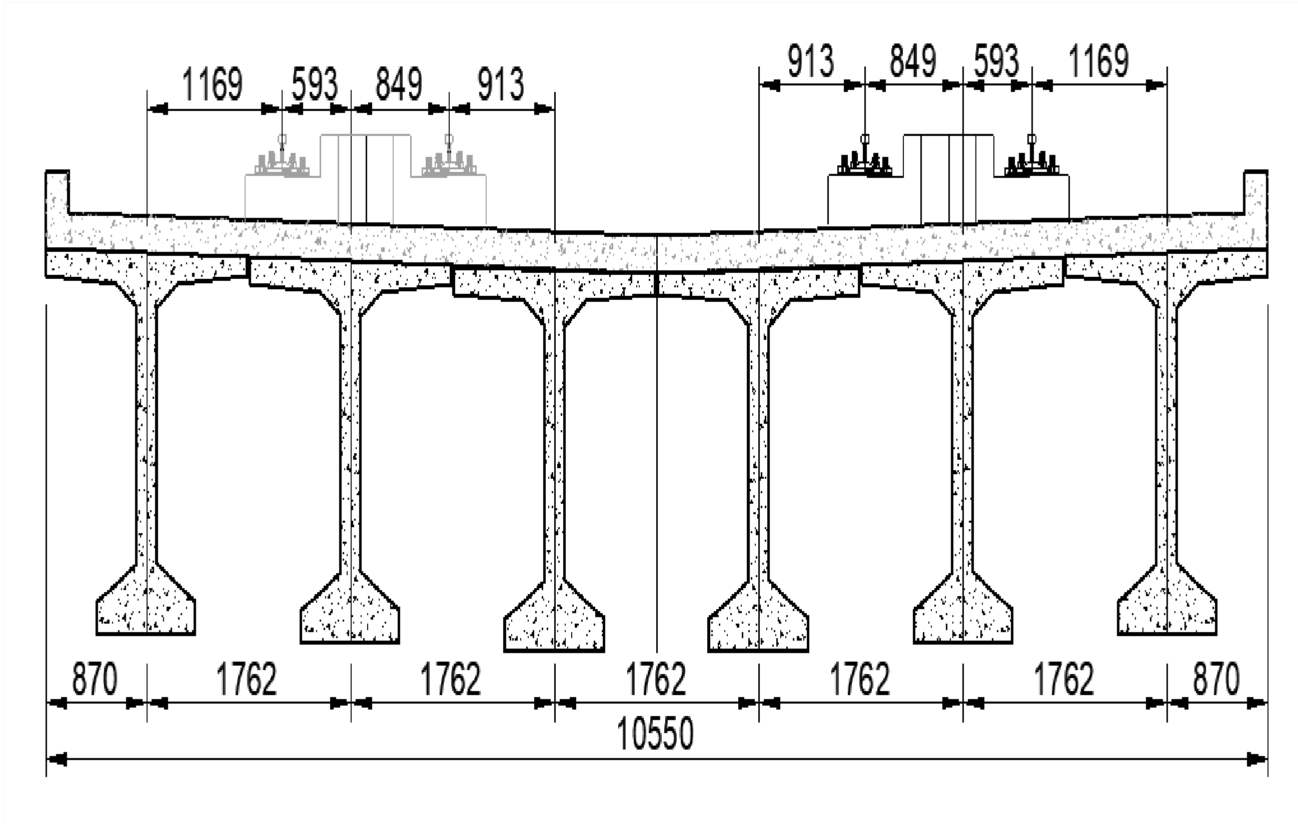
Transverse direction (D_Y) = 0.704 m

Longitudinal direction (D_X) = 1.132 m

Effective Width Calculations

(According to Code of Practice for Concrete Road Bridges IRC : 112-2011, Page 278, Annexure B-3)





Cross-Sectional View showing different values of 'a'

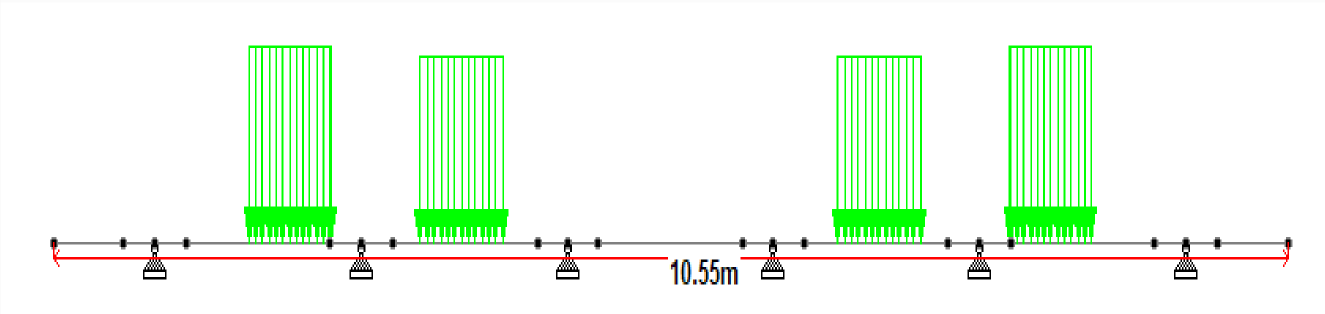
The effective width may be calculated in accordance with the following formula

$$b_{ef} = \alpha \times a\left(1 - \frac{a}{l_o}\right) + b_1$$

Where,

be	the effective width of slab on which the
f f	load acts. the effective span.
L_o	the distance of the center of gravity of the concentrated load from nearer support.
a	the breadth of concentrated area of the load, including diffusion of
b_1	load.
α	a constant depending upon the ratio b / L_{o1} , where b is the width of the slab

LL UDL for Rail 1	LL UDL for Rail 2
L_o = **1.762** m	L_o = **1.762** m
b / L_o = 20.999	b / L_o = 20.999
α = 2.600	α = 2.600
a = **0.593** m	a = **0.849** m
b_1 = 1.132 m	b_1 = 1.132 m
Therefore,	Therefore,
b_{eff} = 2.155 m Resultant b_{eff} for 2 wheel positions = 4.310 m **Live Load Distribution of each Plinth:**	b_{eff} = 2.276 m Resultant b_{eff} for 2 wheel positions = 4.552 m **Live Load Distribution of each Plinth:**
LL(with Impact) as UDL to be applied on D_Y = **11.206 T/m**	LL(with Impact) as UDL to be applied on D_Y = **10.611 T/m**
(assuming unit width of Slab)	(assuming unit width of Slab)
LL UDL for Rail 3	**LL UDL for Rail 4**
L_o = **1.762** m	L_o = **1.762** m
b / L_o = 20.999	b / L_o = 20.999
α = 2.600	α = 2.600
a = **0.849** m	a = **0.593** m
b_1 = 1.132 m	b_1 = 1.132 m
Therefore,	Therefore,
b_{eff} = 2.276 m Resultant b_{eff} for 2 wheel positions = 4.552 m **Live Load Distribution of each Plinth:**	b_{eff} = 2.155 m Resultant b_{eff} for 2 wheel positions = 4.310 m **Live Load Distribution of each Plinth:**
LL(with Impact) as UDL to be applied on D_Y = **10.611 T/m**	LL(with Impact) as UDL to be applied on D_Y = **11.206 T/m**
(assuming unit width of Slab)	(assuming unit width of Slab)



Staad Model Showing Application Of Live Load

Live Load applied in Derailment case (With-OHE & Without-OHE)

INPUT FOR SUPPORT SECTION

Girder arrangement

No of Girder = 6.00 Nos

Top Flange width of Girder = 1.740 m

Thickness of top flange = 0.100 m

Web thickness of Girder = 0.200 m

Slope of Deck Slab 2.50% = 0.025

Thickness of deck slab = 0.200 m

Length of

left cantilever (LHS edge to C/L of left most girder) CL 0.870 m

span_1 (girder_1 to girder_2) S1 1.762 m

span_2 (girder_2 to girder_3) S2 1.762 m

span_3 (girder_3 to girder_4) S3 1.762 m

span_4 (girder_4 to girder_5) S4 1.762 m

span_5 (girder_5 to girder_6) S5 1.762 m

right cantilever (RHS edge to C/L of right most girder) CR 0.870 m Total deck width 10.550 m

Track arrangement

No of Tracks 2.00

Centre to centre of rails 1.435 m

Distance from	Min	Max	
Left edge to C/L Track_1	2.760	2.760	m
Track_1 to track_2	5.030	5.030	m

TRAIN LIVE LOAD (TLL)

Maximum axle laod 17.0 t

Wheel load 8.5 t

Impact factor 2.00

Width of rail Pad in the traffic direction b_1' 0.000 m C/c axle 2.500 m

Rail height 0.214 m

Plinth height (minimum height is to be considered) 0.252 m

Dispersion through rail 1 V : 2 H

Dispersion through plinth 1 V : 1 H

Dispersed width for a wheel load = 0 + (0.252 x 1) x 2 b_1 0.504 m

Effective width calculation for single load

To workout the most unfavourable position inside an area of m on either side of track centre line, derailment loads

Derailment position from centre line of track = 2.250 on either side

Track no.		1		2	
Derailment towards		LHS	RHS	LHS	RHS
Distance from left edge	(m)	0.510	5.010	5.54	10.040
Wheel lies in span		CL	S3	S3	CR
Length of span 'L_o'	(m)	0.870	1.762	1.762	0.870
Distance 'a'	(m)	0.360	0.616	0.616	0.360
Dispersed Width 'b_1'	(m)	0.000	0.000	0.000	0.000
Effective width 'b_{eff}'	(m)	0.432	1.042	1.042	0.432
b_{eff} after overlap	(m)	0.432	1.042	1.042	0.432
Load/metre ULS	(t/m)	13.600	13.056	13.056	13.600
Load/metre SLS	(t/m)	7.771	7.461	7.461	7.771

Derailment position from centre line of track = 1.625 on either side

Track no.		1		2	
Derailment towards		LHS	RHS	LHS	RHS
Distance from left edge	(m)	1.135	4.385	6.165	9.415
Wheel lies in span		S1	S2	S4	S5
Length of span 'L_o'	(m)	1.762	1.762	1.762	1.762
Distance 'a'	(m)	0.265	0.009	0.009	0.265
Dispersed Width 'b_1'	(m)	0.000	0.000	0.000	0.000
Effective width 'b_{eff}'	(m)	0.585	0.023	0.023	0.585
b_{eff} after overlap	(m)	0.585	0.023	0.023	0.585
Load/metre ULS	(t/m)	13.600	13.600	13.600	13.600
Load/metre SLS	(t/m)	7.771	7.771	7.771	7.771

Derailment position from centre line of track = 1.325 on either side

Track no.		1		2	
Derailment towards		LHS	RHS	LHS	RHS
Distance from left edge	(m)	1.435	4.085	6.465	9.115
Wheel lies in span		S1	S2	S4	S5
Length of span 'L_o'	(m)	1.762	1.762	1.762	1.762
Distance 'a'	(m)	0.565	0.309	0.309	0.565
Dispersed Width 'b_1'	(m)	0.000	0.000	0.000	0.000
Effective width 'b_{eff}'	(m)	0.998	0.663	0.663	0.998
b_{eff} after overlap	(m)	0.998	0.663	0.663	0.998
Load/metre ULS	(t/m)	13.600	13.600	13.600	13.600
Load/metre SLS	(t/m)	7.771	7.771	7.771	7.771

Derailment position from centre line of track 1.000

Track no.		1		2	
Derailment towards		LHS	RHS	LHS	RHS
Distance from left edge	(m)	1.760	3.760	6.79	8.790
Wheel lies in span		S1	S2	S4	S5
Length of span 'L_o'	(m)	1.762	1.762	1.762	1.762
Distance 'a'	(m)	0.872	0.634	0.634	0.872
Dispersed Width 'b_1'	(m)	0.000	0.000	0.000	0.000
Effective width 'b_{eff}'	(m)	1.145	1.055	1.055	1.145
b_{eff} after overlap	(m)	1.145	1.055	1.055	1.145
Load/metre ULS	(t/m)	11.876	12.888	12.888	11.876
Load/metre SLS	(t/m)	6.786	7.364	7.364	6.786

Derailment position from centre line of track = 0.750 on either side

Track no.		1		2	
Derailment towards		LHS	RHS	LHS	RHS
Distance from left edge	(m)	2.010	3.510	7.04	8.540
Wheel lies in span		S1	S2	S4	S5
Length of span 'L_o'	(m)	1.762	1.762	1.762	1.762
Distance 'a'	(m)	0.622	0.878	0.878	0.622
Dispersed Width 'b_1'	(m)	0.000	0.000	0.000	0.000
Effective width 'b_{eff}'	(m)	1.046	1.145	1.145	1.046
b_{eff} after overlap	(m)	1.046	1.145	1.145	1.046
Load/metre ULS	(t/m)	12.998	11.875	11.875	12.998
Load/metre SLS	(t/m)	7.427	6.786	6.786	7.427

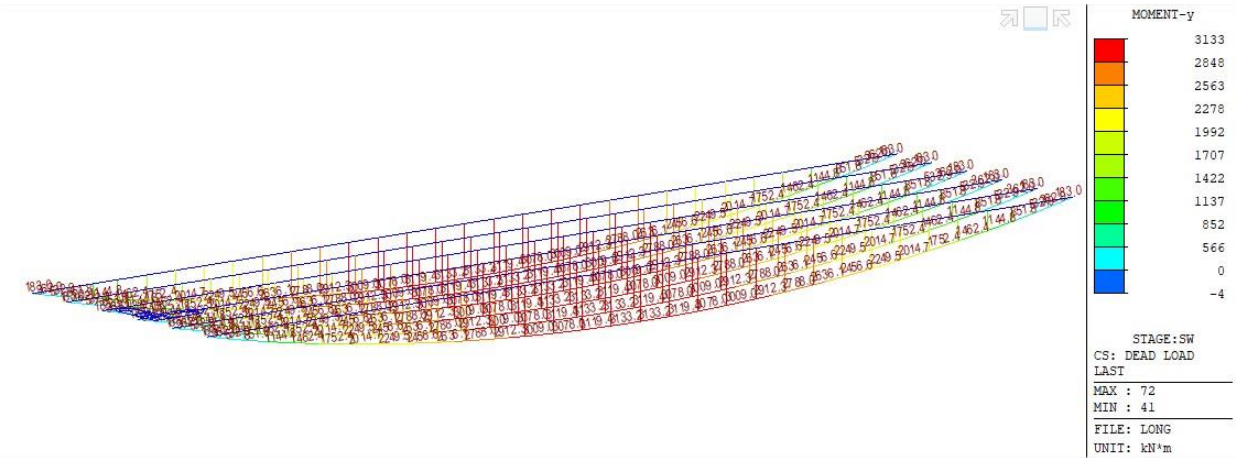
CHAPTER 4

DESIGN AND ANALYSIS RESULT

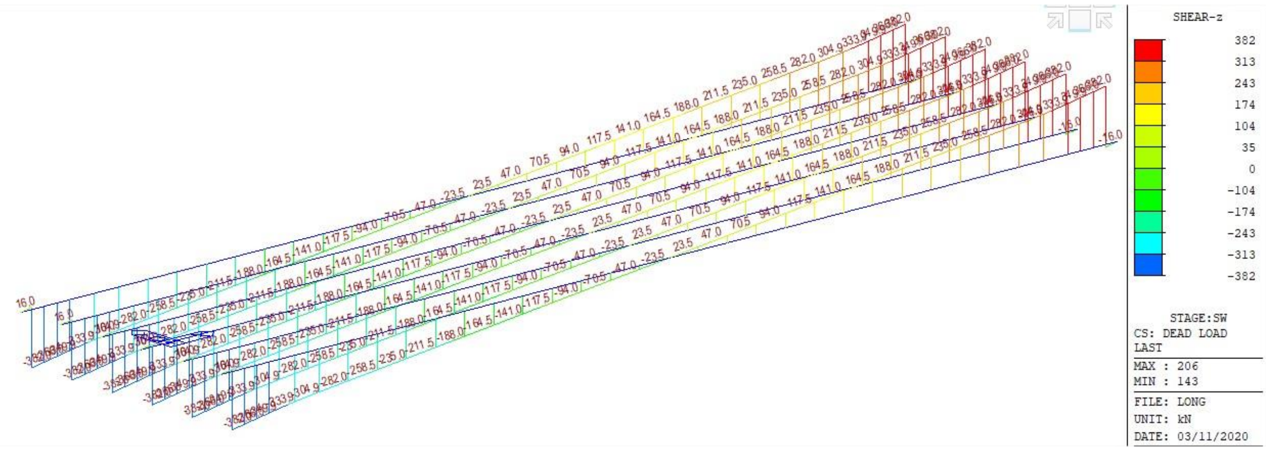
4.1 Analysis Results of T-Girder Longitudinally (Bending Moment & Shear Force)

Based on the cross section properties and the loading discussed above, MIDAS analysis outputs are Presented below (Moment in kN-m & Shear Force in KN). All loads and results are unfactored.

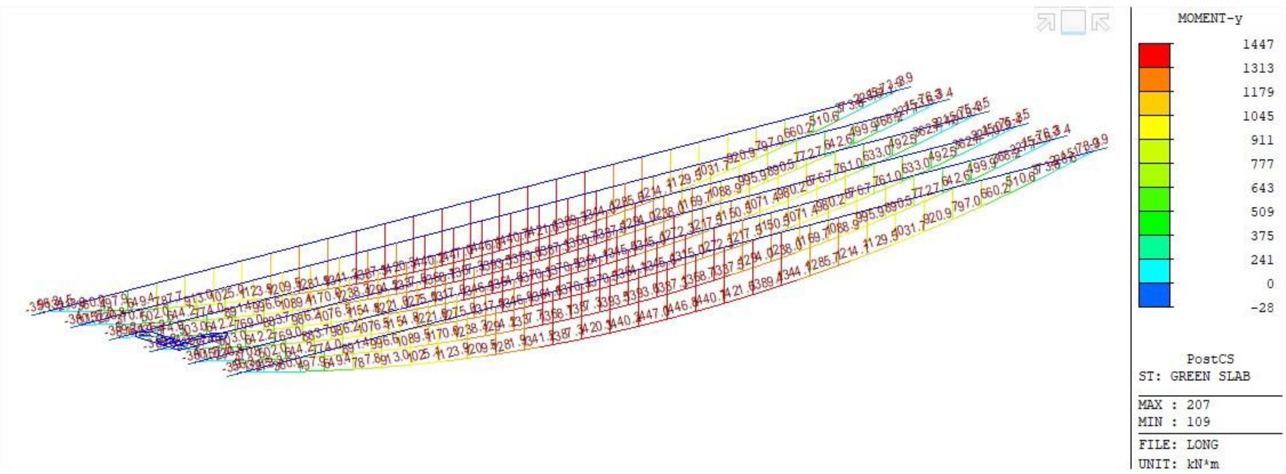
1. Dead Load (T-Girder) (Bending Moment)



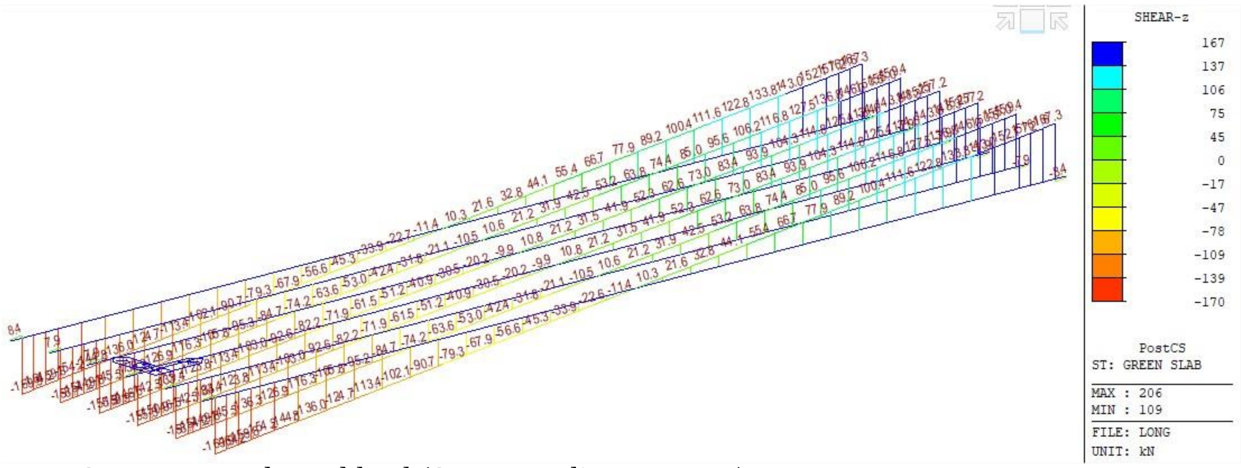
2. Dead Load (T-Girder) (Shear Force)



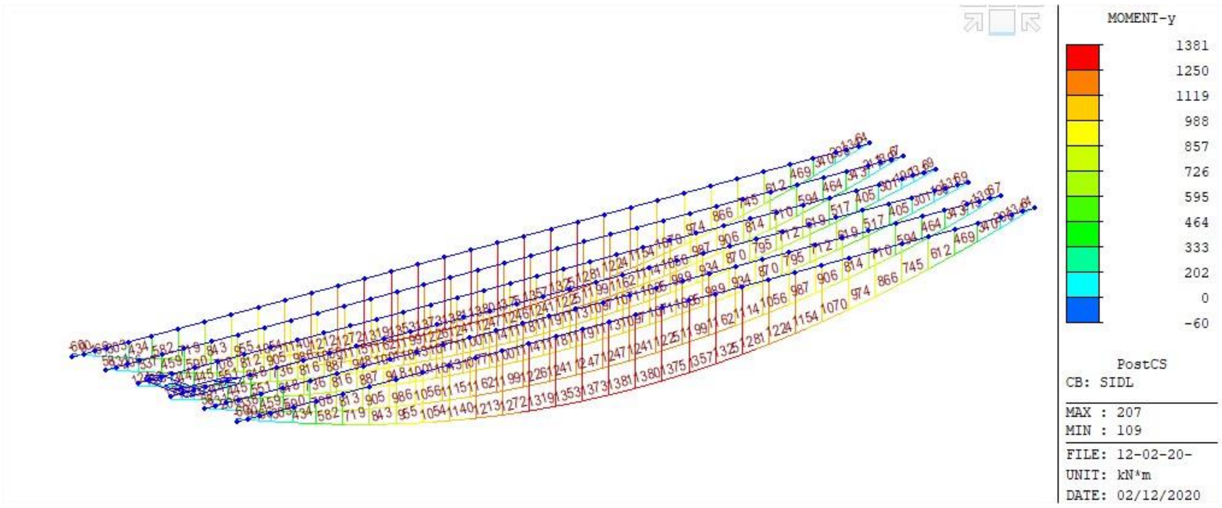
1. Dead Load (Slab) (Bending Moment)



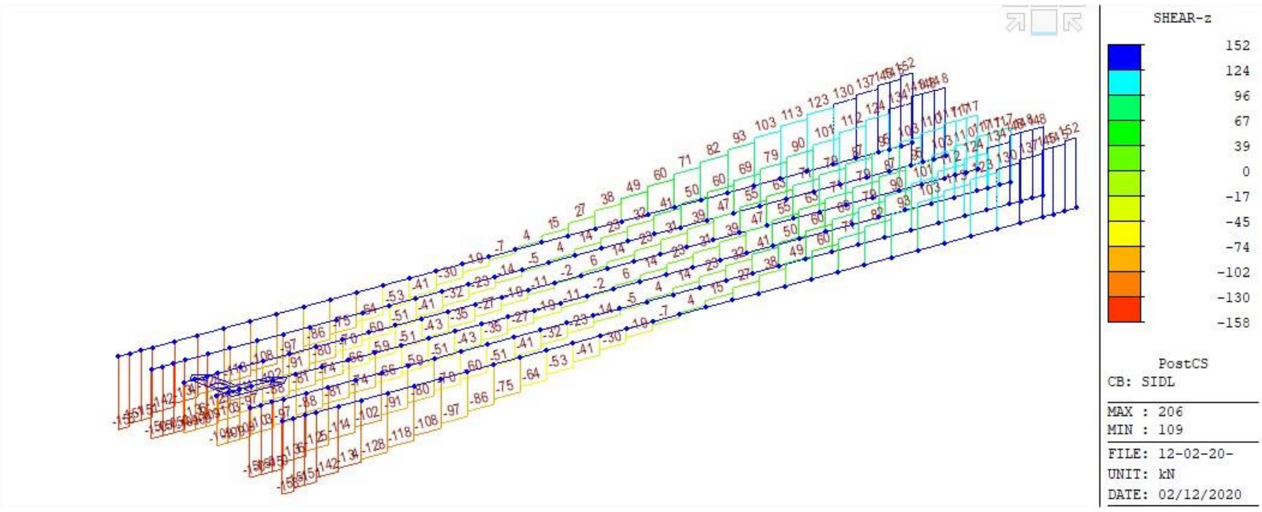
2. Dead Load (Slab) (Shear Force)



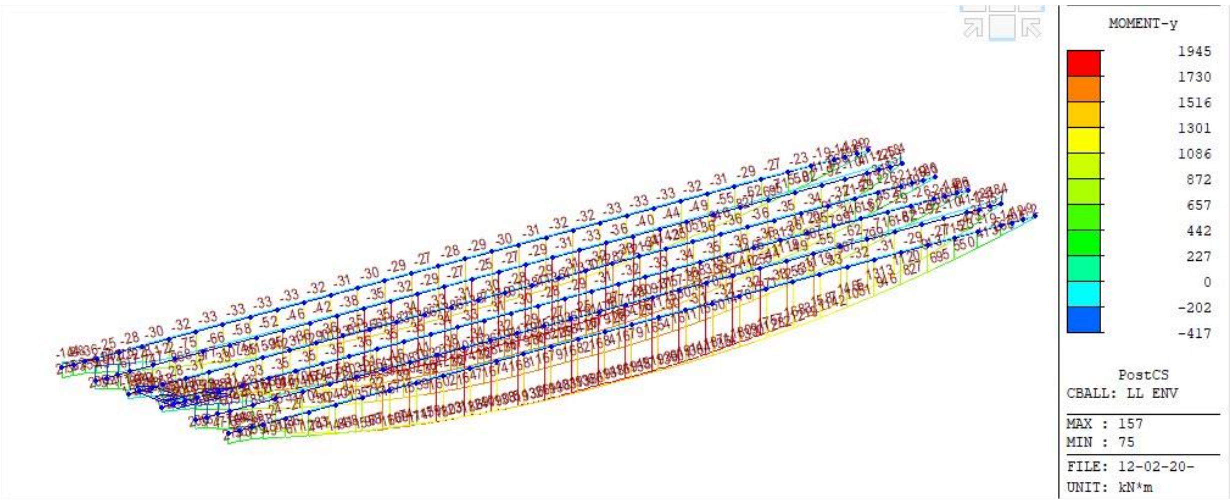
3. Super Imposed Dead load (SIDL-Bending Moment)



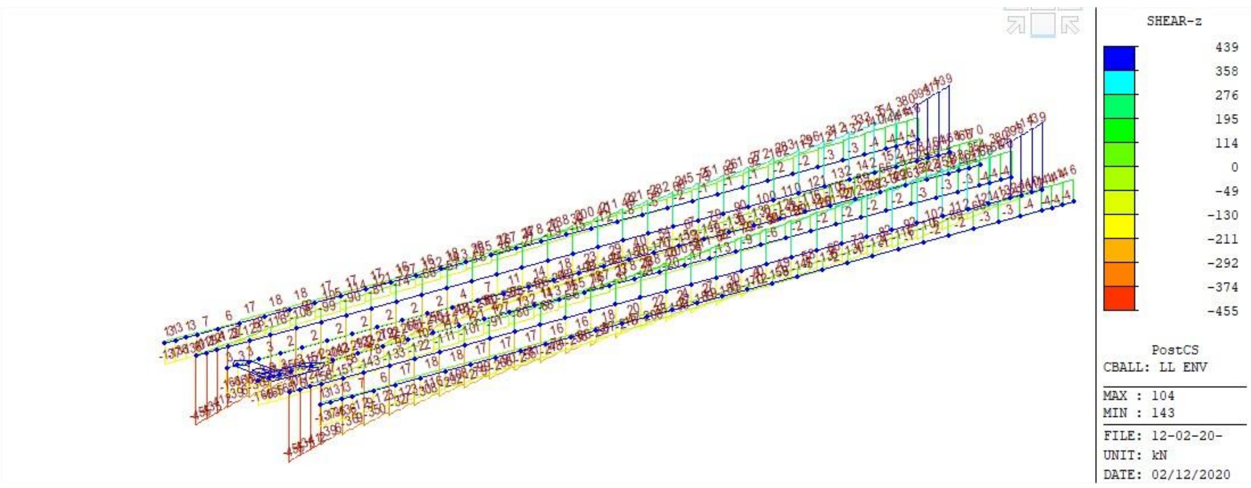
1. Super Imposed Dead load (SIDL-Shear Force)



2. Live Load - (Bending Moment)



3. Live Load - (Shear Force)



4.2CHECKING OF STRESESS

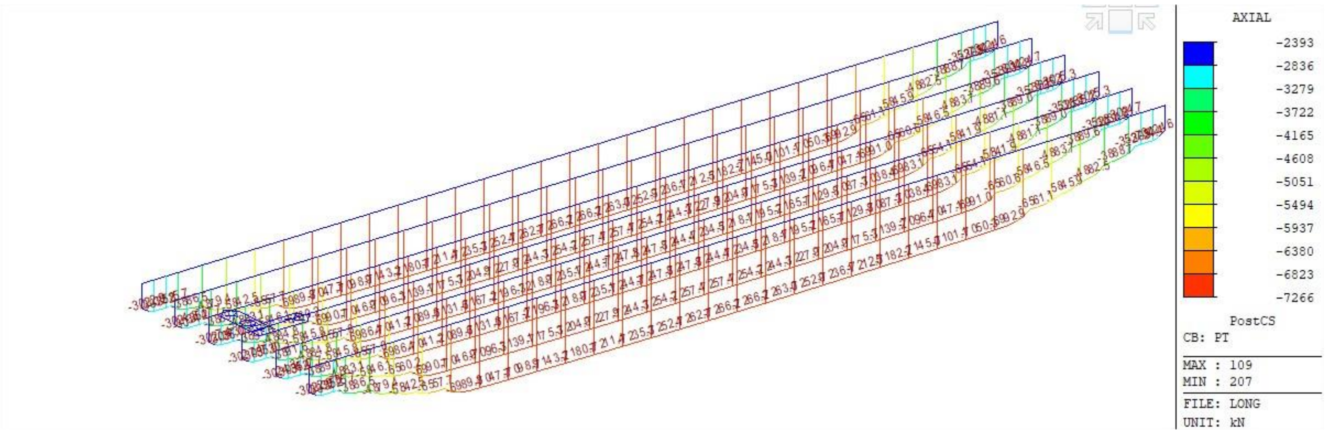
4.2.1 Total Losses

- Jacking Force = 1395*140*51/ (1000) ^2
  = 9.96 MN

  Where,

  Area of Strands = 140 mm^2

  Number of Strands = 51

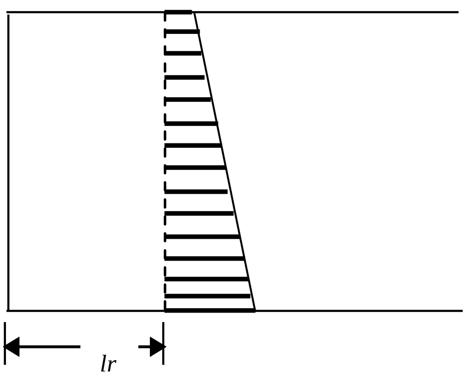


After all losses effective Pre-Stressing force at Long Term = 7.266 MN

Therefore, Total Losses = (1 – (7.245/9.96)) *100 = **27.05%**

4.2.2 Regularization Length

The regularization length defines the distance necessary from strand ends to develop linear normal stresses diagram throughout the section of the beam.



lr : regularisation length

As per BPEL, Appendix 4, Section 3.1, the regularization length lr is given by : $lr = \sqrt{[(0.8*lsn)^2 + dpi^2]}$
Where : lsn = μ/0.85*les = (fjacking/fpu) /0.85*(75*Østrand) = 1006mm = 1m dpi = 2.2 - 0.303 = 1.897m (distance from strands CG to extreme top fiber)

So $lr = \sqrt{[(0.8*1)^2 + 1.897^2]} = 2.06m$

Therefore, stress calculation shall be made only beyond 2.0 m from T-Girder end.

4.1 Normal Stresses

4.1.1 Construction Stages

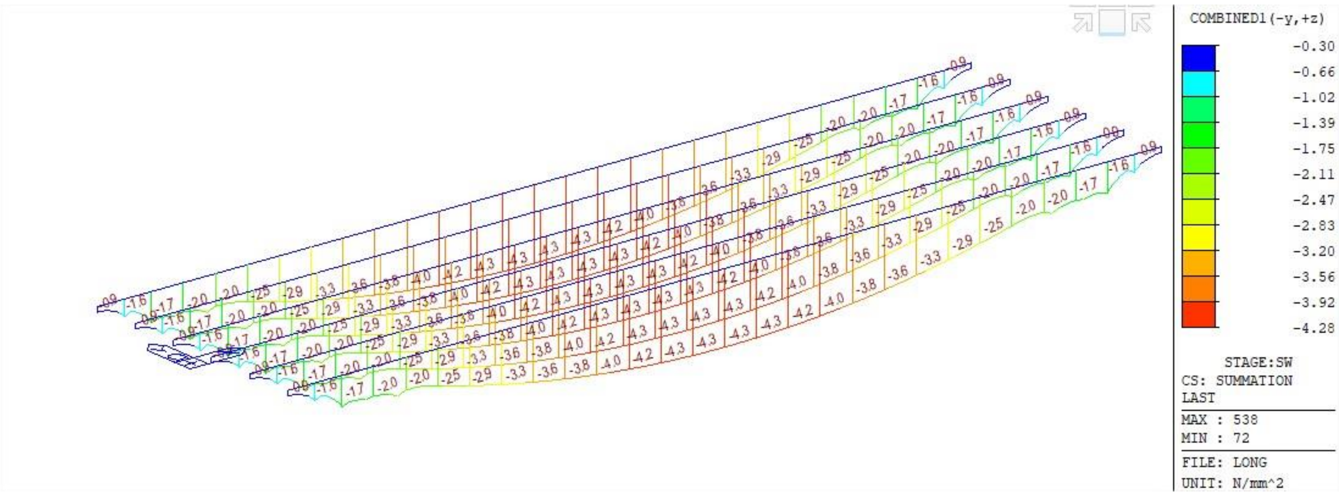
The permissible stresses for the construction stage are as follows.

TOP & BOTTOM FIBER STRESS : No Tension

: Compression should not exceed 22.5 MPa.

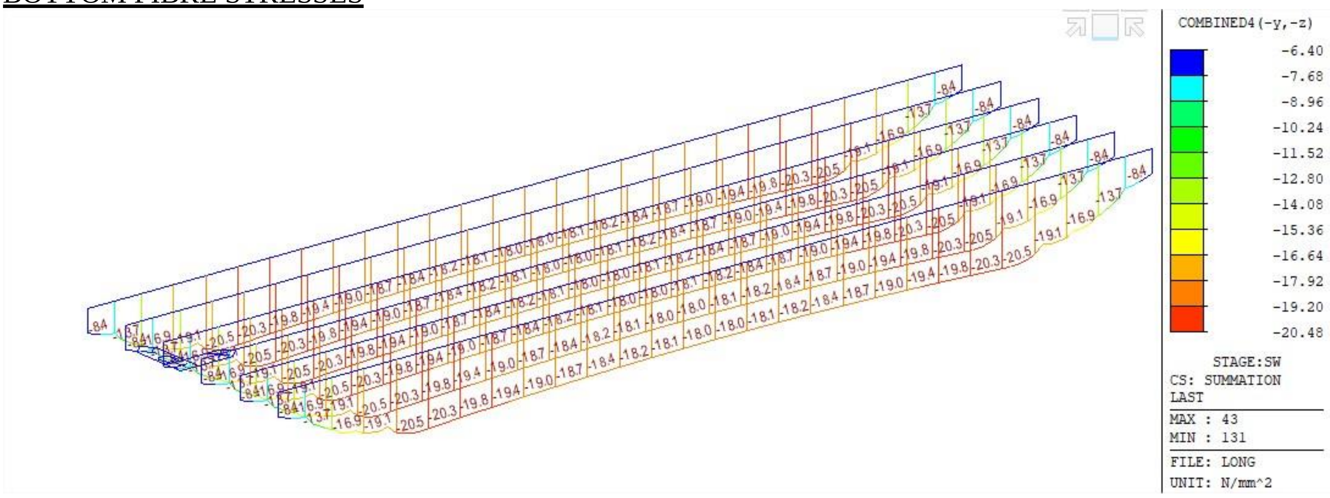
CONSTRUCTION STAGES:- PHASE 1: Self Weight of T-Girder +PT

TOP FIBRE STRESSES



PHASE 1	σ top (Mpa)	σ top permissible (Mpa)	Remark
MIN	-0.3	0	OK
MAX	-4.28	-22.5	OK

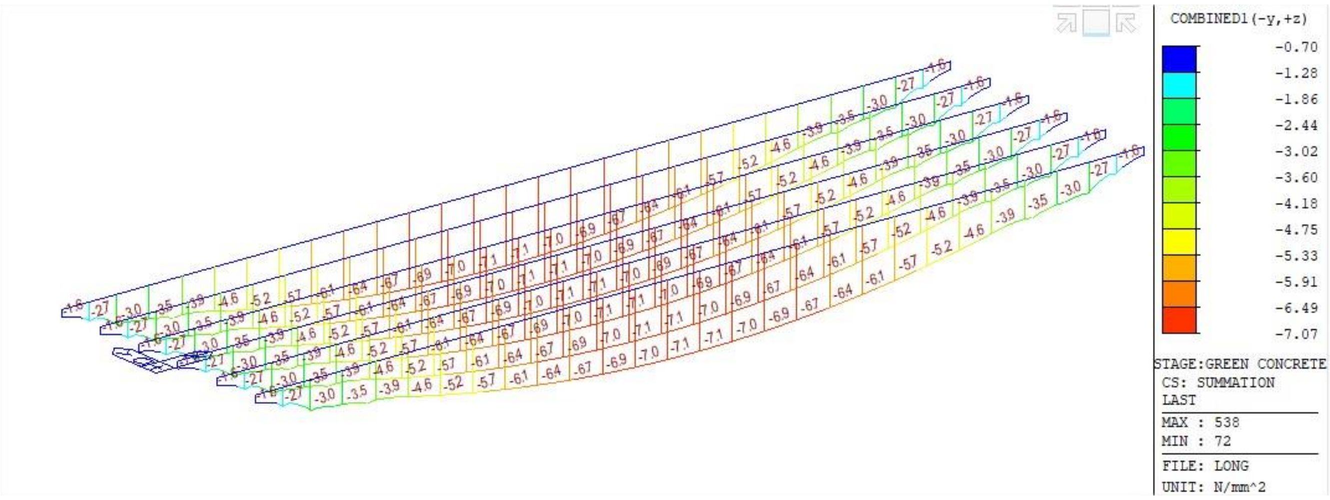
BOTTOM FIBRE STRESSES



PHASE 1	σ bottom (Mpa)	σ bottom permissible (Mpa)	Remark
MIN	-6.4	0	OK
MAX	-20.48	-22.5	OK

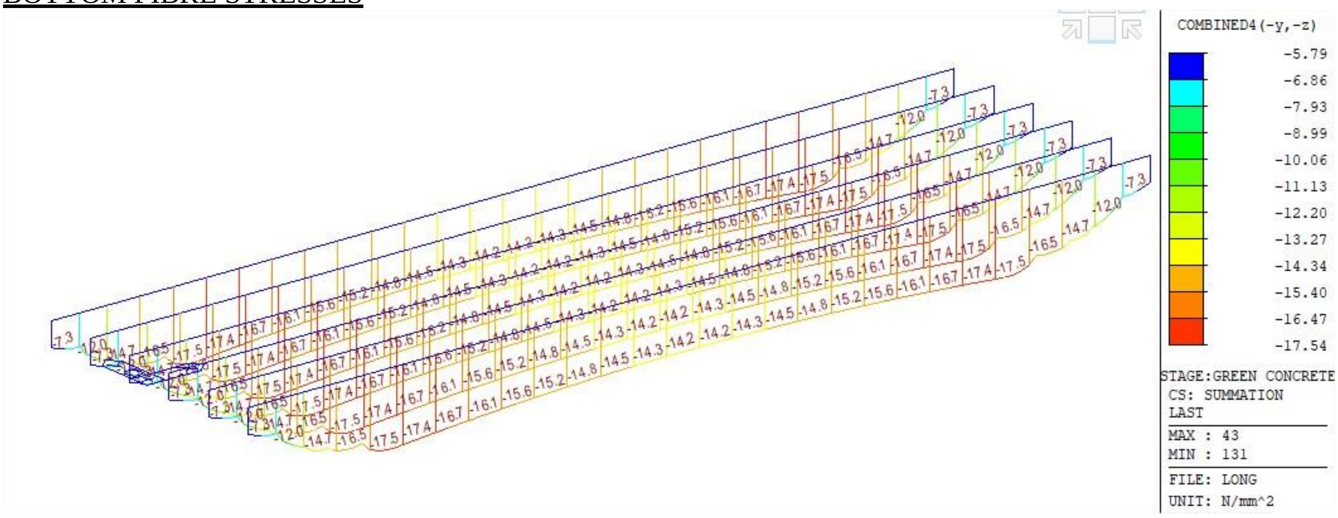
CONSTRUCTION STAGES:- PHASE 2: Application of slab weight as green concrete

TOP FIBRE STRESSES



PHASE 2	σ top (Mpa)	σ top permissible (Mpa)	Remark
MIN	-0.7	0	OK
MAX	-7.07	-22.5	OK

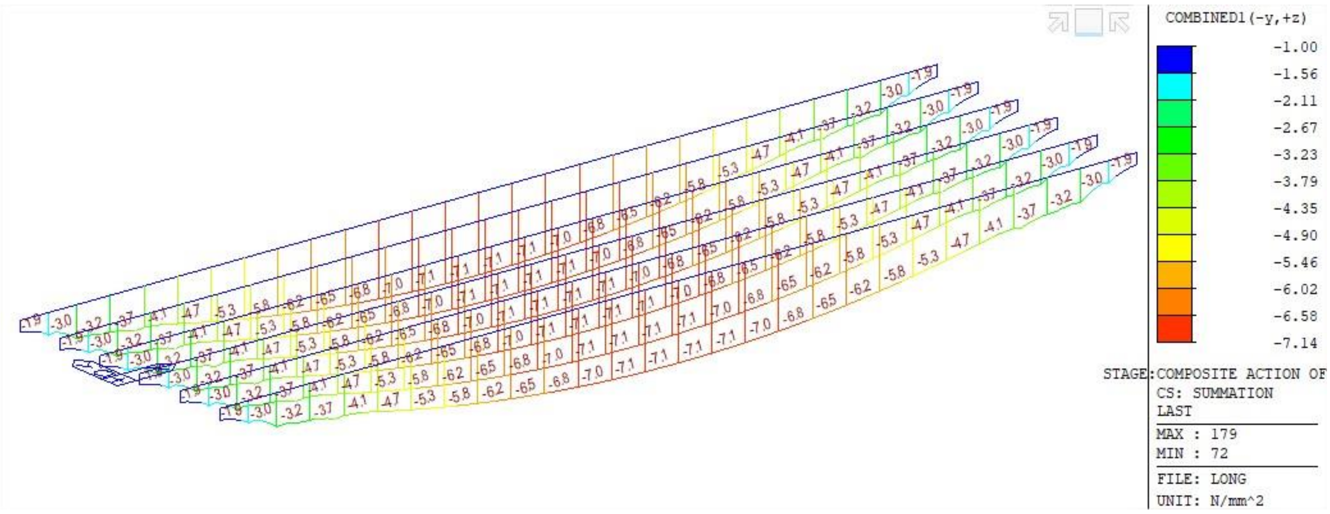
BOTTOM FIBRE STRESSES



PHASE 2	σ bottom (Mpa)	σ bottom permissible (Mpa)	Remark
MIN	-5.79	0	OK
MAX	-17.54	-22.5	OK

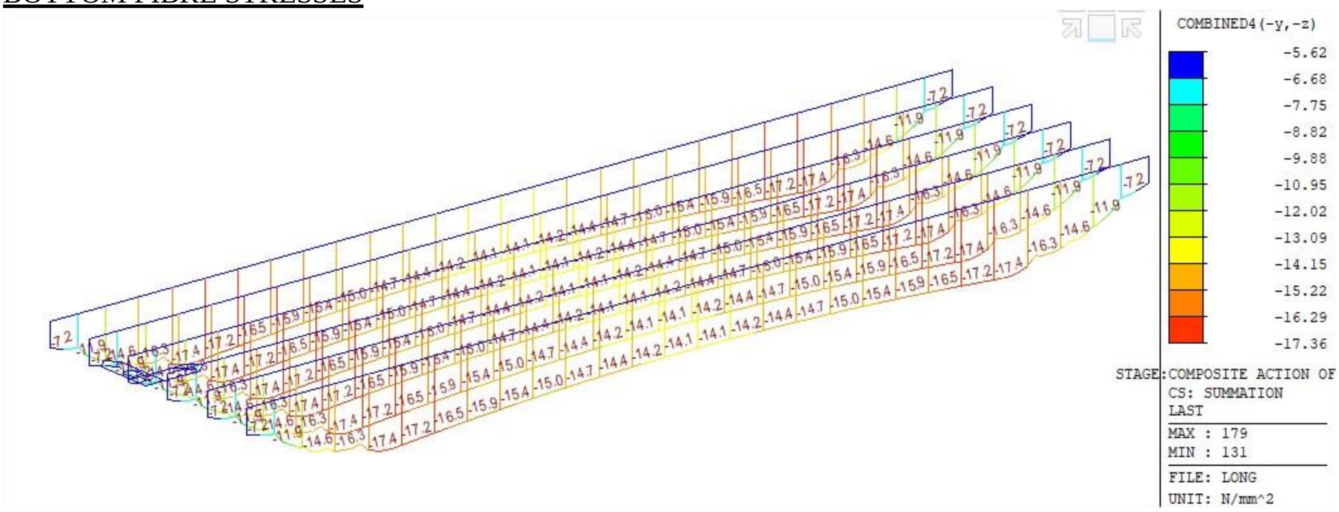
CONSTRUCTION STAGES:- PHASE 3: Slab Harden

TOP FIBRE STRESSES



PHASE 3	σ top (Mpa)	σ top permissible (Mpa)	Remark
MIN	-1.00	0	OK
MAX	-7.14	-22.5	OK

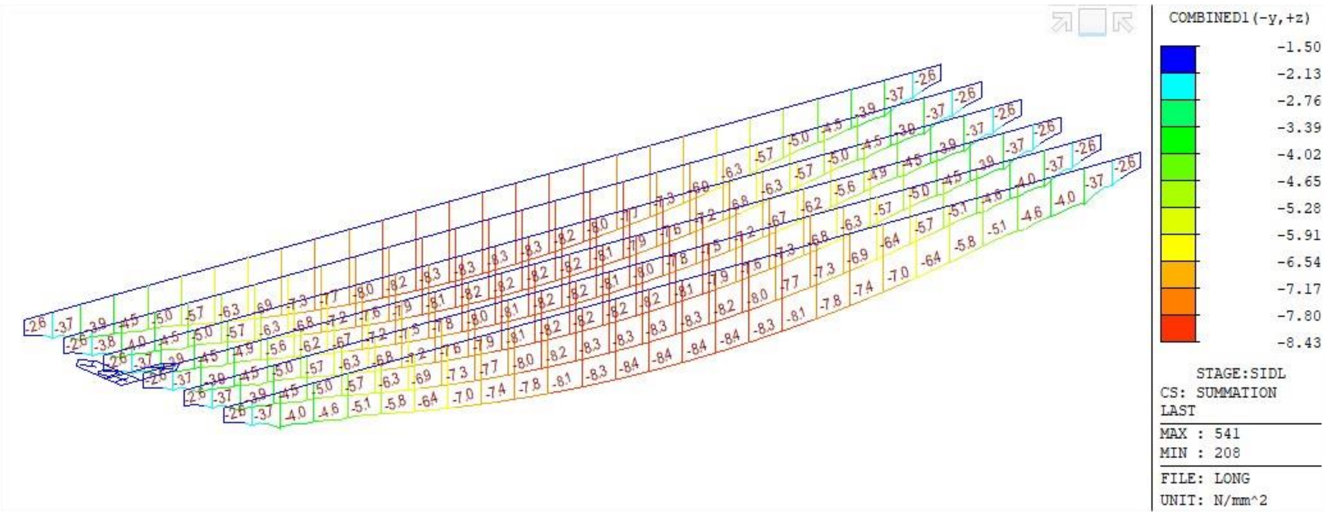
BOTTOM FIBRE STRESSES



PHASE 3	σ bottom (Mpa)	σ bottom permissible (Mpa)	Remark
MIN	-5.62	0	OK
MAX	-17.36	-22.5	OK

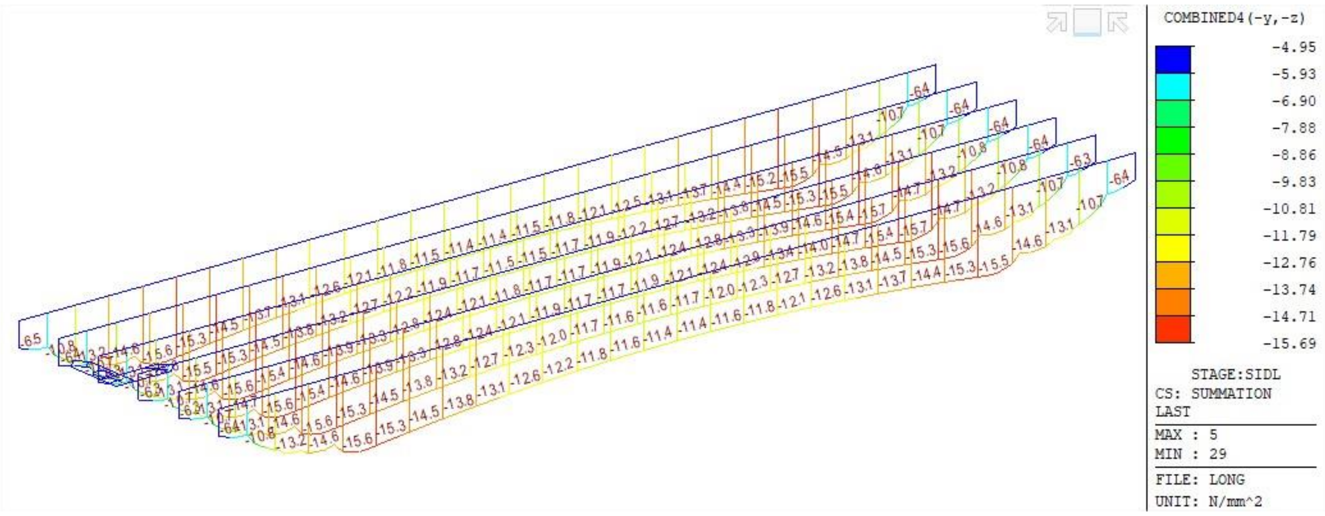
CONSTRUCTION STAGES:- PHASE 4: SIDL

TOP FIBRE STRESSES



PHASE 4	σ top (Mpa)	σ top permissible (Mpa)	Remark
MIN	-1.50	0	OK
MAX	-8.43	-22.5	OK

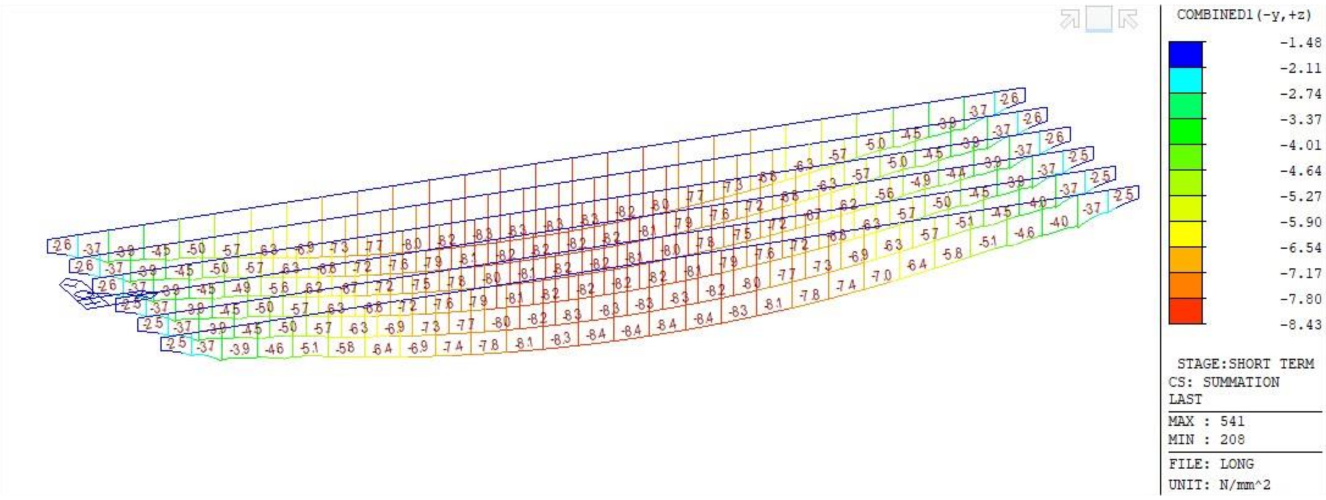
BOTTOM FIBRE STRESSES



PHASE 4	σ top (Mpa)	σ top permissible (Mpa)	Remark
MIN	-4.95	0	OK
MAX	-15.69	-22.5	OK

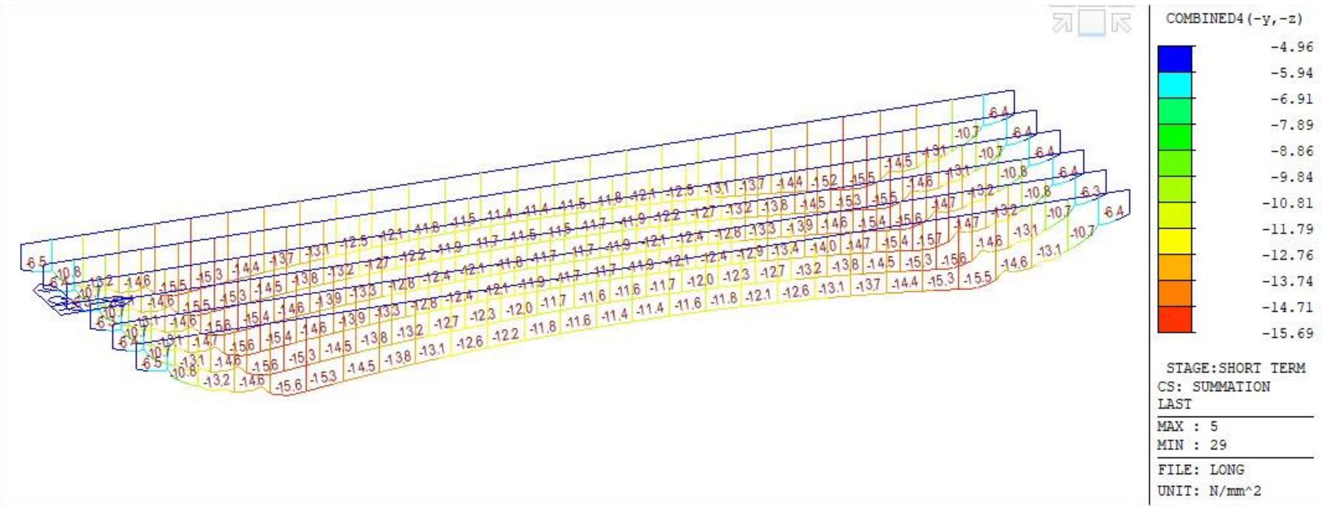
CONSTRUCTION STAGES:- PHASE 5: Short Term

TOP FIBRE STRESSES



PHASE 5	σ top (Mpa)	σ top permissible (Mpa)	Remark
MIN	-1.48	0	OK
MAX	-8.43	-22.5	OK

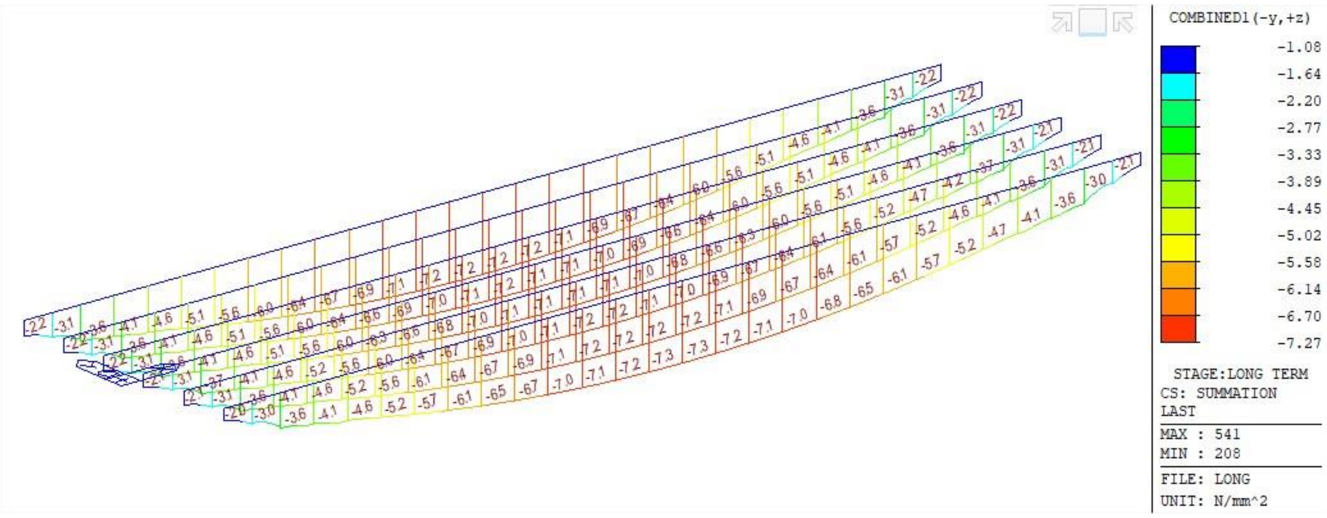
BOTTOM FIBRE STRESSES



PHASE 5	σ bottom (Mpa)	σ bottom permissible (Mpa)	Remark
MIN	-4.96	0	OK
MAX	-15.69	-22.5	OK

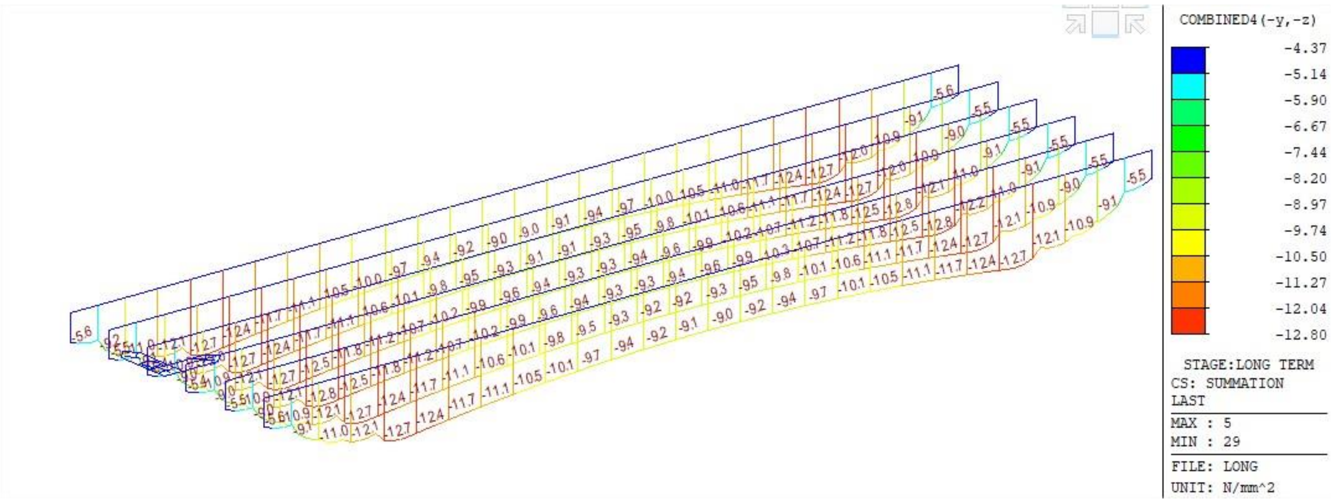
CONSTRUCTION STAGES:- PHASE 6: Long Term

TOP FIBRE STRESSES



PHASE 6	σ top (Mpa)	σ top permissible (Mpa)	Remark
MIN	-1.08	0	OK
MAX	-7.27	-22.5	OK

BOTTOM FIBRE STRESSES



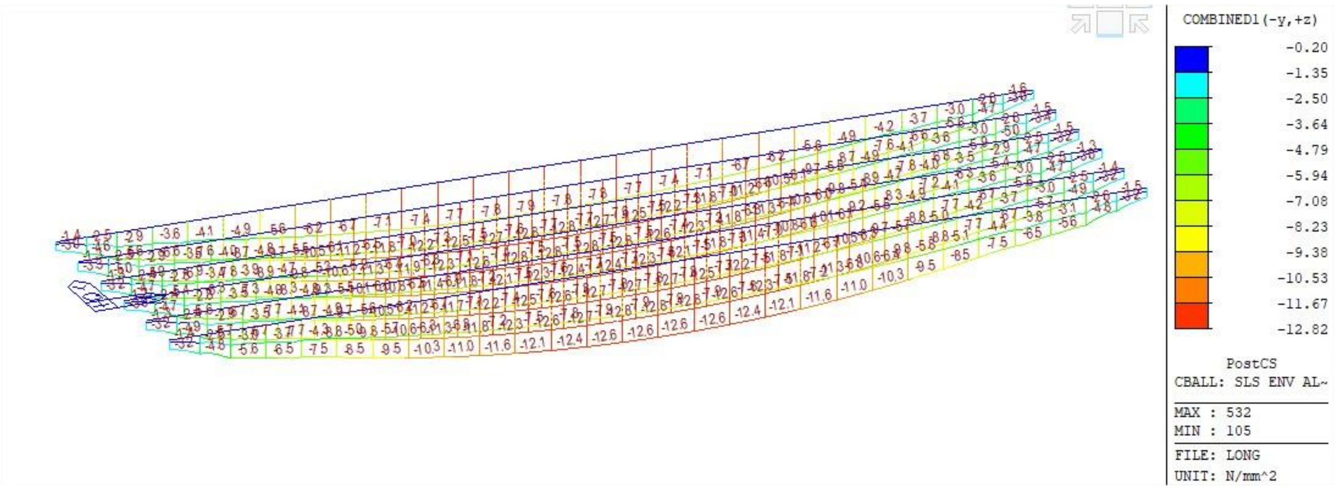
PHASE 6	σ bottom (Mpa)	σ bottom permissible (Mpa)	Remark
MIN	-4.37	0	OK
MAX	-12.80	-22.5	OK

4.1.2 Service Stage after 1 day (Short Term Stresses)

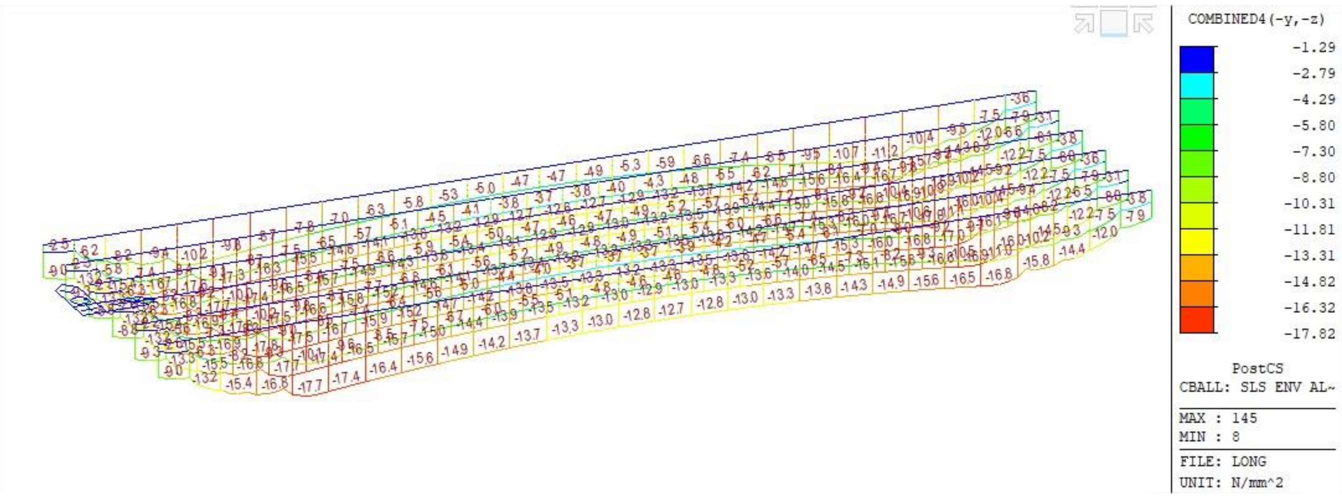
The permissible stresses for the Service stage are as follows.

- TOP & BOTTOM FIBER STRESS : No Tension
  : Compression should not exceed 22 MPa.

<u>TOP FIBRE STRESSES: SLS ENVELOPE UNDER GI, GII & GIII</u>



<u>BOTTOM FIBRE STRESSES: SLS ENVELOPE UNDER GI, GII & GIII</u>



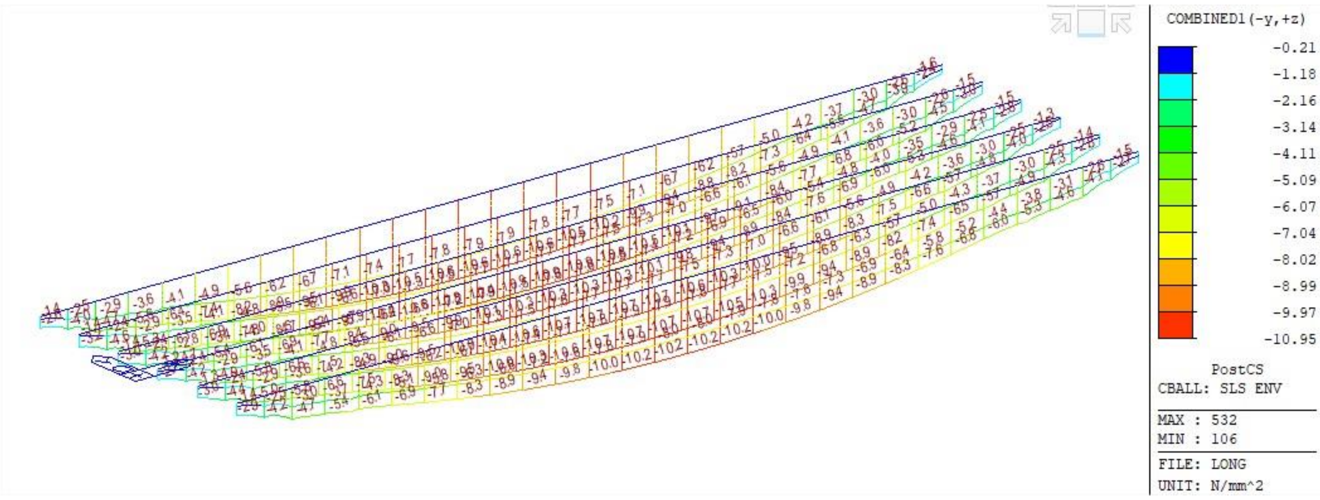
SERVICE STAGE (Short Term)	σ_{top} (Mpa)	σ_{bot} (Mpa)	σ_{top} permissible (Mpa)	σ_{bot} permissible (Mpa)	Remark
MIN	-0.20	-1.29	0	0	OK
MAX	-12.82	-17.82	-22	-22	OK

4.1.3 Service Stage after 100 years (Long Term Stresses)

The permissible stresses for the Service stage are as follows.

- TOP & BOTTOM FIBER STRESS : No Tension
  : Compression should not exceed 22 MPa.

<u>TOP FIBRE STRESSES: SLS ENVELOPE UNDER GI, GIII</u>



<u>BOTTOM FIBRE STRESSES: SLS ENVELOPE UNDER GI, GIII</u>

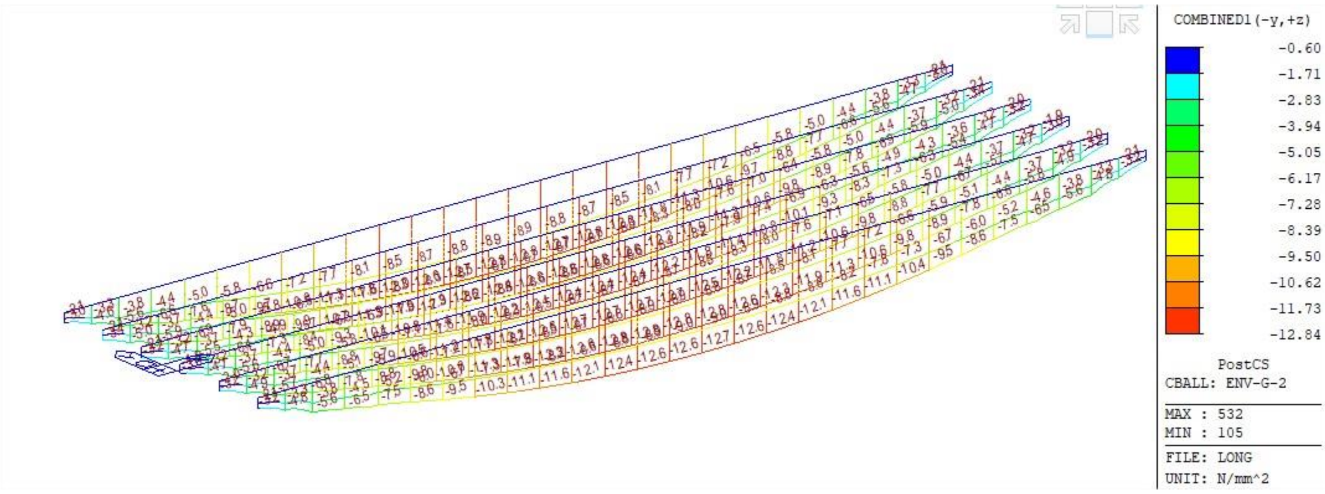
SERVICE STAGE (Long Term)	σ_{top} (Mpa)	σ_{bot} (Mpa)	σ_{top} permissible (Mpa)	σ_{bot} permissible (Mpa)	Remark
MIN	-0.21	-0.59	0	0	OK
MAX	-10.95	-14.84	-22	-22	OK

4.1.4 Max stresses (G-II envelope in wind & Seismic) cases

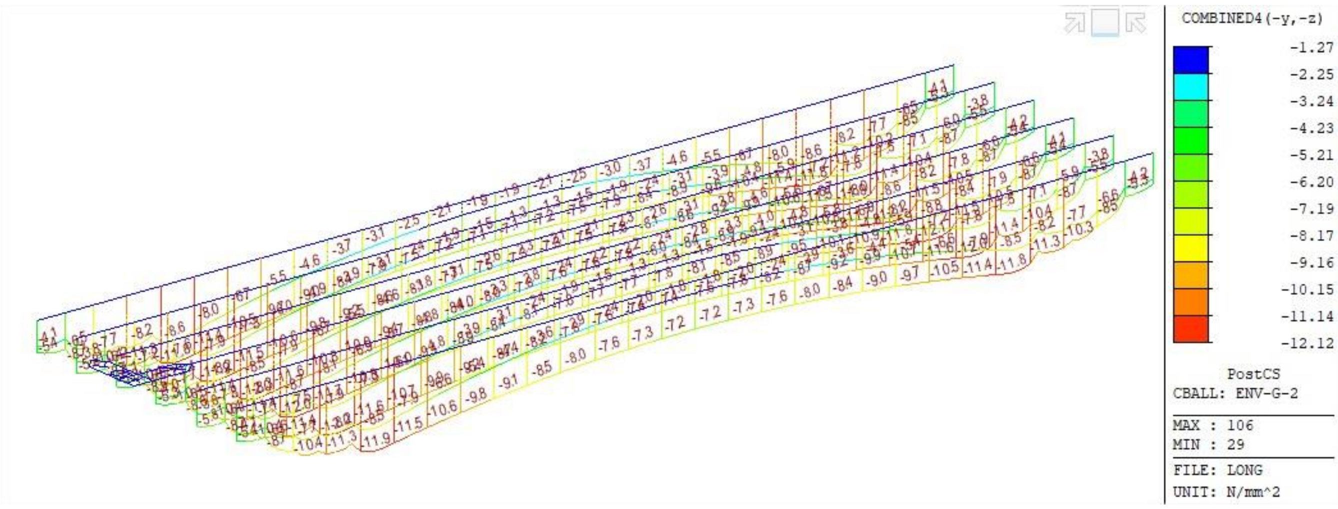
The permissible stresses for the Service stage are as follows.

- TOP & BOTTOM FIBER STRESS : No Tension
  : Compression should not exceed 22 MPa.

<u>TOP FIBRE STRESSES: SLS ENVELOPE UNDER GII (WIND / SEISMIC)</u>



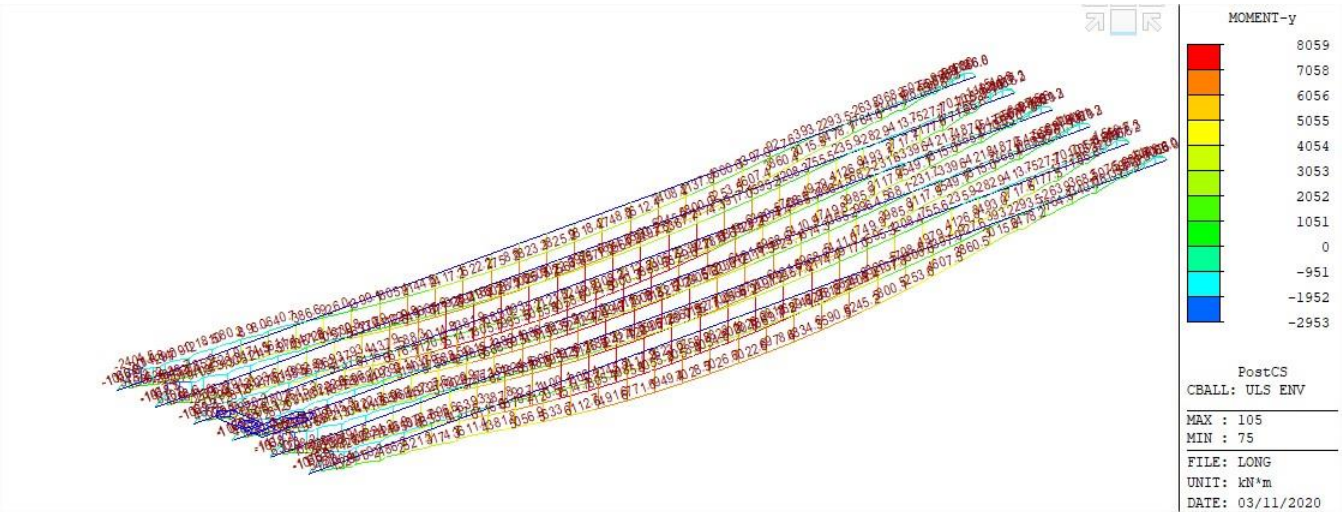
<u>BOTTOM FIBRE STRESSES: SLS ENVELOPE UNDER GII (WIND / SEISMIC)</u>



SERVICE STAGE (Long Term)	σ_{top} (Mpa)	σ_{bot} (Mpa)	σ_{top} permissible (Mpa)	σ_{bot} permissible (Mpa)	Remark
MIN	-0.60	-1.27	0	0	OK
MAX	-12.84	-12.12	-22	-22	OK

4.2 Ultimate Bending Moment Verification

The ultimate bending moment corresponding to ULS is shown below.



The ultimate moment capacity is checked by using the IRS-CBC guidelines.

According to IRS CBC, Section 16.4.3.1, the applied moment increased by 15%. i.e. ratio of (Applied moment -+

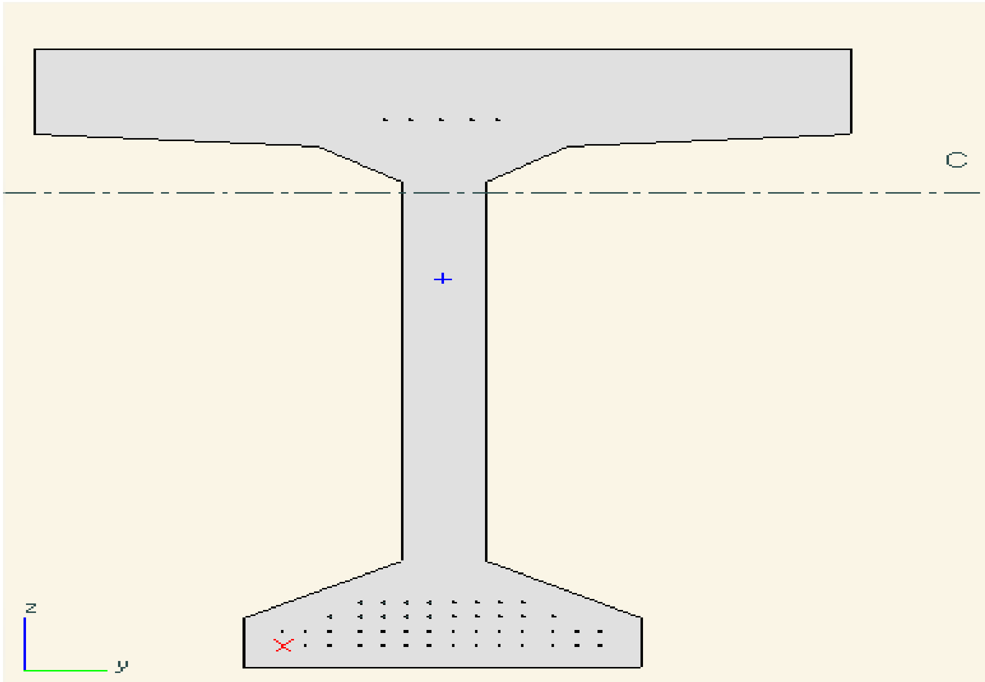
*1.15) to ultimate moment M_u should be less than 1.

RESULTS:

Section	Applied Moment*1.15 M (KN-m)	Ultimate Moment M_u (KN-m)	M/M_u	Permissible M/M_u	Result
Section at mid	**-8059*1.15 = -9268**	**-17460**	**0.531**	**1.0**	**OK**

Please find below the details of calculation of Moment capacity

4.3 Oasys Section check at Mid of T-Girder-Span



Section Material Properties

Type		Concrete
Name		M55
Weight		Normal Weight
Density	ρ	$2.300 t/m^3$
Cube Strength	f_{cu}	55.00MPa
Tensile Strength	f_{ct}	0.0MPa
Elastic Modulus (short term)	E	38310.MPa
Poisson's Ratio	ν	0.2000
Coeff. Thermal Expansion	α	12.00E-6/°C
Partial Safety Factor	$\gamma_{mc,ULS}$	1.500
	$\gamma_{mc,SLS}$	1.000
Maximum Strain		0.003500[-]
ULS Compression Curve		Recto-parabolic
ULS Tension Curve		No-tension
SLS Compression Curve		Linear
SLS Tension Curve		No-tension
Aggregate Size		0.0mm

Applied loads

Load Case	N	M_{yy}	M_{zz}
	[kN]	[kNm]	[kNm]
1	-1.000	9268.	0.0

- **ULS Check**

Strength Analysis - Summary

Governing conditions are defined as:
A - reinforcing steel tension strain limit
B - concrete compression strain limit
Effective centroid is reported relative to the reference point.

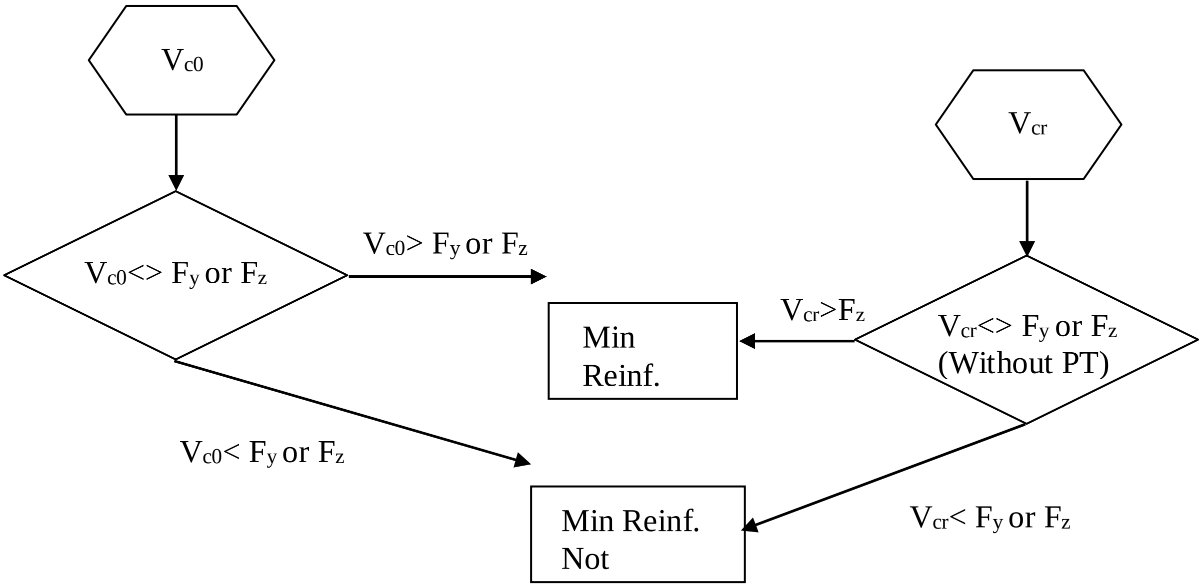
Case	Eff. Centroid (y)	Eff. Centroid (z)	N [kN]	M [kNm]	M_u [kNm]	M/M_u	Governing Condition
Maxima							
1	-	-	-1.000	9268.	17460.	0.5309	A: Bar 1
Minima							
1	-	-	-1.000	9268.	17460.	0.5309	A: Bar 1

Ratio of M/Mu *(=0.531)* < 1.000. **O.K.**

4.6 MAXIMUM SHEAR CHECK

4.6.1 Shear Reinforcement (links)

According to IRS CBC, Section 16.4.4.1.1 at any section the ultimate shear resistance of the concrete alone, V_c, shall be considered for the section both uncracked (V_{c0}) and cracked (V_{cr}) in flexure, and if necessary shear reinforcement shall be provided.



Fy or Fz = Ultimate Shear Force corresponding to ULS

4.6.1 Section Uncracked in Flexure

As per IRS CBC, Section 16.4.4.2.2 the vertical component of the prestressing shall be algebraically added to Vco: it shall be taken as positive when it increases the shear resistance of the section.

V_{co} + PT <> F no PT
or, V_{co} <> F with PT

$$V_{c0} = 0.67bh\sqrt{(f_t^2 - f_{cp} f)}$$

As per IRS CBC, Section 16.4.4.2

Where $f_t = 0.24\sqrt{f_{ck}} = 0.24\sqrt{55} = 1.78$ MPa
f_{cp} = compressive stress at the centroidal axis due to prestressing hence fcp= N/ (B*H)
N = Normal force due to Prestressing after all losses with 0.87 factor

4.6.2 Section cracked in Flexure

$$V_{cr} = \max\left(\left(0.037bd\sqrt{f_{cu}} + \frac{M_{cr}}{M}V\,;\, 0.1bd\sqrt{f_{cu}}\right)\right)$$

As per IRS CBC, Section 16.4.4.3

Where, d is the distance from the extreme compression fiber to the centroid of the tendons.

Mcr is the cracking moment, given by:

$$M_{cr} = (0.37\sqrt{f_{cu}} + f_{PT})\frac{I}{y}$$

Where, $f_{PT} = N/A + (N*e_0) * y_g/I$

Where,

fPT= Stress due to PT only at the tensile fiber at a distance y_g from the centroid of the section (which has a second moment of area of I).

M & V are due to the ultimate loads.

N = Normal force due to Prestressing after all losses with 0.87

factor A = Cross sectional area of section

e_0 = Distance between COG of section to COG of

tendons y_g = Distance from tensile fiber to COG of

section

I = Inertia of section

Minimum reinforcement (to IRS CBC, Section 16.4.4.4.1):

$$\frac{A_{sv}}{s_v} = \frac{0.4b}{0.87 f_{yv}}$$

A_{sv} = Total cross sectional area of the legs of the stirrups/links S_v = Spacing of the stirrups/links along the length.

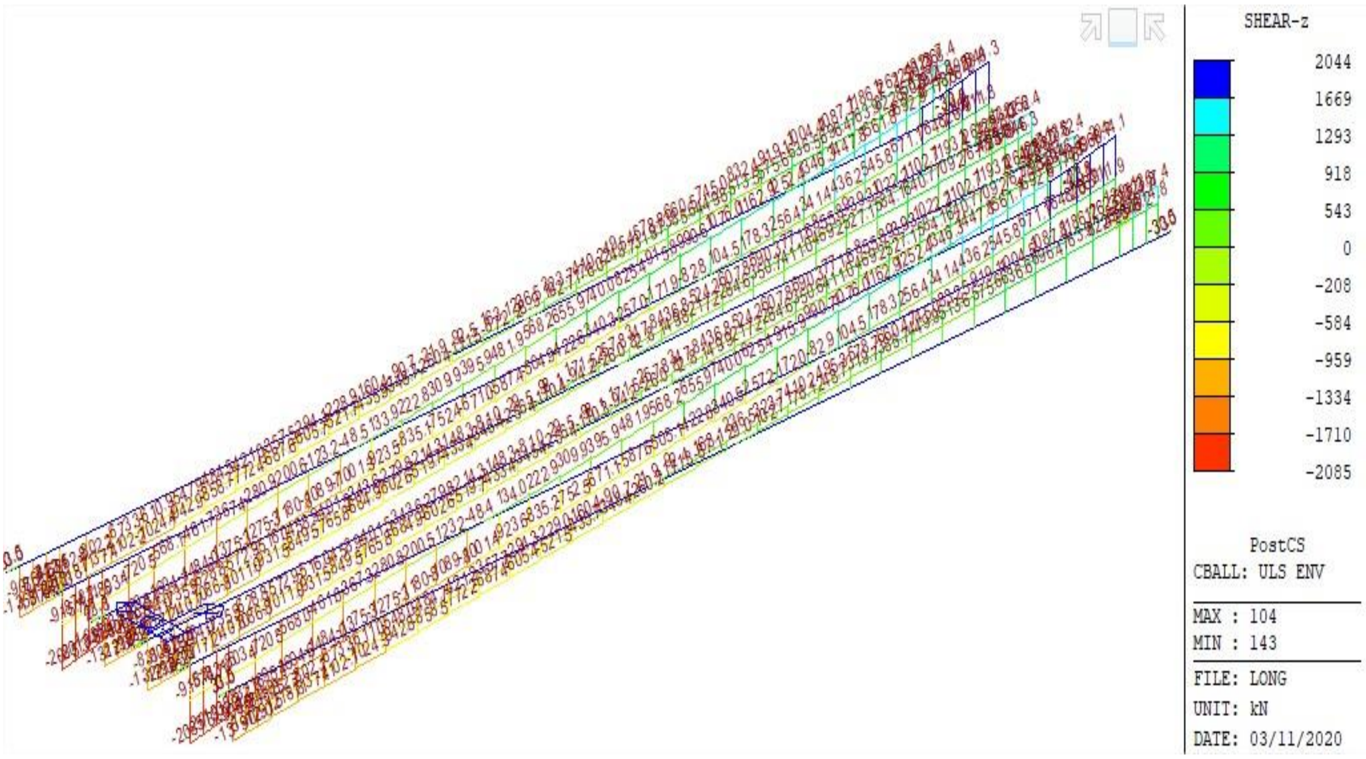
f_{yv} = Characteristic strength of the stirrups/links Reinf.

If minimum reinforcement is not enough, we provide (to IRS CBC, Section 16.4.4.4.2):

$$\frac{A_{sv}}{s_v} = \frac{(V - V_c) + 0.4bd_t}{0.87 f_{yv} d_t}$$

Where dt is the effective depth from the extreme compression fibre to either the longitudinal bars around which the stirrups pass or the centroid of the tendons, whichever is the greater.

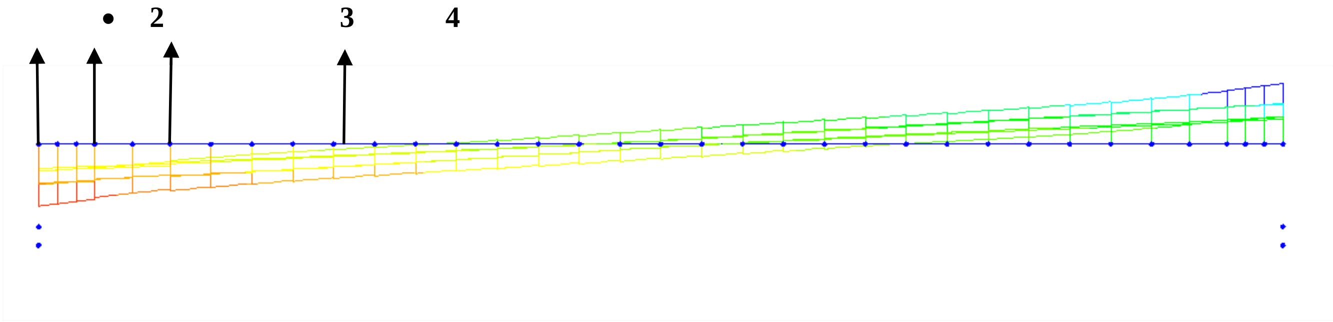
Shear Force Diagram:



Maximum Shear Force due to Ultimate Loads = 2085 kN

Maximum Shear Stress for M-55 = 5.55 N/mm^2

Shear check location at given position:



4.6.4 Shear check.

1: (at C/L of Bearing)

ULTIMATE SHEAR RESISTANCE (IRS Concrete Bridge Code..1997, Cl. 16.4.4)

1.) Input Data :

2b	=	0.530	m	: Thickness of Web
H	=	2.20	m	: Total Height of Section
A	=	1.650	m^2	: Cross Sectional area
f_{ck}	=	55	N/mm^2	: Characteristic Compressive Strength of Concrete
d	=	1.622	m	: Distance from Top fiber to the COG of Steel
I	=	0.842	m^4	: Inertia of Section
Y_g	=	1.29	m	: Distance from Bottom fiber to the Center of Gravity of Section
W_g	=	0.910	m	: Distance from Top fiber to the Center of Gravity of Section
e_0	=	0.712	m	: Distance between C.O.G of Section to C.O.G. of Tendons
f_{yv}	=	415	N/mm^2	: Characteristic Strength of Link Reinforcement
V_u	=	2.121	MN	: Applied Ultimate Shear Force (ULS-GI :- 1.25DL+2SIDL+1.75LL)
M_u	=	1.968	MN-m	: Applied Ultimate Moment (ULS-GI :- 1.25DL+2SIDL+1.75LL)

	V_u (MN)	Mu (MN-m)
DL	0.552	0.007
SIDL	0.158	0.061
LL+I	0.637	1.050

2.) Section Uncracked in Flexure :

f_t	=	1.780	N/mm^2	: Maximum principal tensile stress at the centroidal axis
N	=	2.081	MN	: Normal Force due to Prestressing after all losses with 0.87 factor
f_{cp}	=	1.261	N/mm^2	: Compressive Stress at the Centroidal axis due to PT
V_c	=	1.818	MN	

3.) Section Cracked in Flexure :

f_{pt}	=	3.532	N/mm^2	: Stress at the Tensile Fiber due to PT only with 0.87 factor
M_{cr}	=	4.097	MN-m	: Cracking Moment at the Section Considered
V_{cr}	=	4.650	MN	

Section is Uncracked

4.) Shear Reinforcement :

V_u	=	2.121	MN	: Applied Ultimate Shear Force (ULS-GI :- 1.25DL+2SIDL+1.75LL)
V_c	=	1.818	MN	: Minimum of V_{co} and V_{cr}
A_{sv}/S_v	=	**11.04**	Cm^2/m	: Required Reinforcement

Bar Mark	Spacing	Dia.	Legs
1C	100	12	2
2B	100	12	2

5.) Maximum Shear Stress :

v	=	2.466	N/mm^2	: Applied Shear Stress
v_{max}	=	5.55	N/mm^2	: IRS, Table 26: Maximum Shear Stress

2: (at 1.5m from C/L of Bearing)

2: (at 1.5m from C/L of Bearing)

ULTIMATE SHEAR RESISTANCE (IRS Concrete Bridge Code..1997, Cl. 16.4.4)

1.) Input Data :

2b	=	0.530	m	: Thickness of Web
H	=	2.20	m	: Total Height of Section
A	=	1.650	m²	: Cross Sectional area
fck	=	55	N/mm²	: Characteristic Compresive Strength of Concrete
d	=	1.743	m	: Distance from Top fiber to the COG of Steel
I	=	0.842	m^4	: Inertia of Section
Y_g	=	1.29	m	: Distance from Bottom fiber to the Center of Gravity of Section
W_g	=	0.910	m	: Distance from Top fiber to the Center of Gravity of Section
e_0	=	0.833	m	: Distance between C.O.G of Section to C.O.G. of Tendons
fyv	=	415	N/mm²	: Characteristic Strength of Link Reinforcement
V_u	=	1.872	MN	: Applied Ulimate Shear Force (ULS-GI :- 1.25DL+2SIDL+1.75LL)
M_u	=	3.327	MN-m	: Applied Ulimate Moment (ULS-GI :- 1.25DL+2SIDL+1.75LL)

	V_u (MN)	Mu (MN-m)
DL	0.488	0.743
SIDL	0.142	0.241
LL+I	0.559	1.095

2.) Section Uncracked in Flexure :

f_t	=	1.780	N/mm²	: Maximum principal tensile stress at the centroidal axis
N	=	3.055	MN	: Normal Force due to Prestressing after all losses with 0.87 factor
fcp	=	1.851	N/mm²	: Compressive Stress at the Centroidal axis due to PT
Vc	=	1.986	MN	

3.) Section Cracked in Flexure :

fpt	=	5.750	N/mm²	: Stress at the Tensile Fiber due to PT only with 0.87 factor
Mcr	=	5.544	MN-m	: Cracking Moment at the Section Considered
Vcr	=	3.373	MN	

Section is Uncracked

4.) Shear Reinforcement :

V_u	=	1.872	MN	: Applied Ulimate Shear Force (ULS-GI :- 1.25DL+2SIDL+1.75LL)
V_c	=	1.986	MN	: Minimum of V_{co} and V_{cr}
A_{sv}/S_v	=	**5.87**	Cm^2/m	: Required Reinforcement
A_{sv}	=	38.3	Cm^2/m	: Provided reinforcement

Bar Mark	Spacing	Dia.	Legs
1D	100	10	2
2B	100	12	2

5.) Maximum Shear Stress :

v	=	2.027	N/mm²	: Applied Shear Stress
v_{max}	=	5.55	N/mm²	: IRS, Table 26: Maximum Shear Stress

O.K.

3: (at 3.5m from C/L of Bearing)

ULTIMATE SHEAR RESISTANCE (IRS Concrete Bridge Code..1997, Cl. 16.4.4)

1.) Input Data :

2b	=	0.200	m	: Thickness of Web
H	=	2.20	m	: Total Height of Section
A	=	1.170	m²	: Cross Sectional area
f_{ck}	=	55	N/mm²	: Characteristic Compresive Strength of Concrete
d	=	1.823	m	: Distance from Top fiber to the COG of Steel
I	=	0.718	m^4	: Inertia of Section
Y_g	=	1.388	m	: Distance from Bottom fiber to the Center of Gravity of Section
W_g	=	0.812	m	: Distance from Top fiber to the Center of Gravity of Section
e_0	=	1.011	m	: Distance between C.O.G of Section to C.O.G. of Tendons
f_{yv}	=	415	N/mm²	: Characteristic Strength of Link Reinforcement
V_u	=	1.648	MN	: Applied Ulimate Shear Force (ULS-GI :- 1.25DL+2SIDL+1.75LL)
M_u	=	5.762	MN-m	: Applied Ulimate Moment (ULS-GI :- 1.25DL+2SIDL+1.75LL)

	V_u (MN)	Mu (MN-m)
DL	0.418	1.646
SIDL	0.128	0.457
LL+I	0.497	1.595

2.) Section Uncracked in Flexure :

f_t	=	1.780	N/mm²	: Maximum principal tensile stress at the centroidal axis
N	=	4.421	MN	: Normal Force due to Prestressing after all losses with 0.87 factor
f_{cp}	=	3.778	N/mm²	: Compressive Stress at the Centroidal axis due to PT
V_{co}	=	0.927	MN	

3.) Section Cracked in Flexure :

f_{pt}	=	12.423	N/mm²	: Stress at the Tensile Fiber due to PT only with 0.87 factor
M_{cr}	=	7.846	MN-m	: Cracking Moment at the Section Considered
V_{cr}	=	2.343	MN	

Section is Uncracked

4.) Shear Reinforcement :

V_u	=	1.648	MN	: Applied Ulimate Shear Force (ULS-GI :- 1.25DL+2SIDL+1.75LL)
V_c	=	0.927	MN	: Minimum of V_{co} and V_{cr}
A_{sv}/S_v	=	**13.16**	Cm^2/m	: Required Reinforcement
A_{sv}	=	22.6	Cm^2/m	: Provided reinforcement

Bar Mark	Spacing	Dia.	Legs
-	0	0	0
2B	100	12	2

5.) Maximum Shear Stress :

v	=	4.518	N/mm²	: Applied Shear Stress
v_{max}	=	5.55	N/mm²	: IRS, Table 26: Maximum Shear Stress

O.K.

4: *(at 8.5m from C/L of Bearing)*

ULTIMATE SHEAR RESISTANCE (IRS Concrete Bridge Code..1997, Cl. 16.4.4)

1.) Input Data :

2b	=	0.200	m	: Thickness of Web
H	=	2.20	m	: Total Height of Section
A	=	1.170	m²	: Cross Sectional area
f_{ck}	=	55	N/mm²	: Characteristic Compressive Strength of Concrete
d	=	1.887	m	: Distance from Top fiber to the COG of Steel
I	=	0.718	m^4	: Inertia of Section
Y_g	=	1.388	m	: Distance from Bottom fiber to the Center of Gravity of Section
W_g	=	0.812	m	: Distance from Top fiber to the Center of Gravity of Section
e_0	=	1.075	m	: Distance between C.O.G of Section to C.O.G. of Tendons
f_{yv}	=	415	N/mm²	: Characteristic Strength of Link Reinforcement
V_u	=	1.220	MN	: Applied Ultimate Shear Force (ULS-GI :- 1.25DL+2SIDL+1.75LL)
M_u	=	10.725	MN-m	: Applied Ultimate Moment (ULS-GI :- 1.25DL+2SIDL+1.75LL)

	V_u (MN)	Mu (MN-m)
DL	0.278	3.372
SIDL	0.086	0.952
LL+I	0.400	2.632

2.) Section Uncracked in Flexure :

f_t	=	1.780	N/mm²	: Maximum principal tensile stress at the centroidal axis
N	=	6.111	MN	: Normal Force due to Prestressing after all losses with 0.87 factor
f_{cp}	=	5.223	N/mm²	: Compressive Stress at the Centroidal axis due to PT
V_{co}	=	1.041	MN	

3.) Section Cracked in Flexure :

f_{pt}	=	17.919	N/mm²	: Stress at the Tensile Fiber due to PT only with 0.87 factor
M_{cr}	=	10.689	MN-m	: Cracking Moment at the Section Considered
V_{cr}	=	1.320	MN	

Section is Uncracked

4.) Shear Reinforcement :

V_u	=	1.220	MN	: Applied Ulimate Shear Force (ULS-GI :- 1.25DL+2SIDL+1.75LL)
V_c	=	1.041	MN	: Minimum of V_{co} and V_{cr}
A_{sv}/S_v	=	4.85	Cm^2/m	: Required Reinforcement
A_{sv}	=	10.5	Cm^2/m	: Provided reinforcement

Bar Mark	Spacing	Dia.	Legs
-	0	0	0
2C	150	10	2

5.) Maximum Shear Stress :

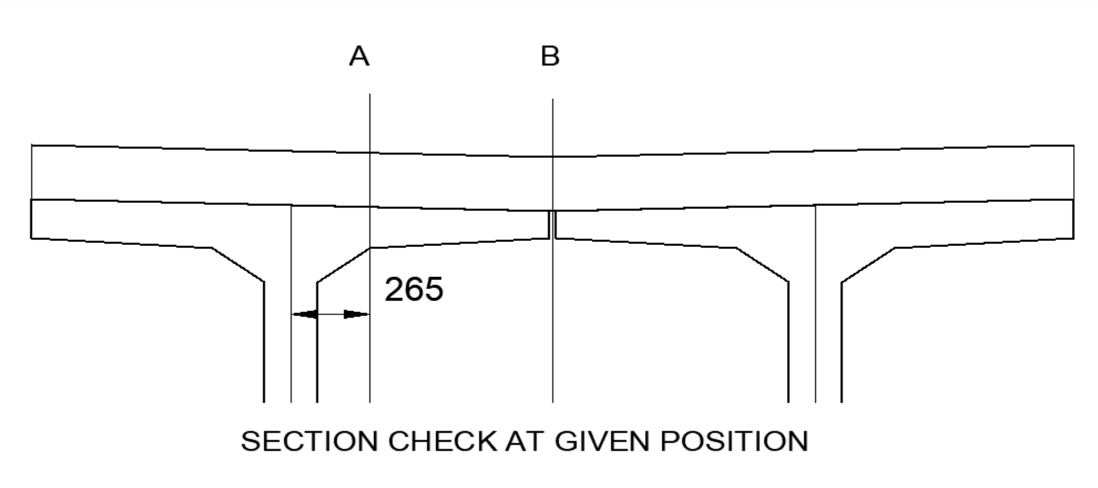
v	=	3.234	N/mm²	: Applied Shear Stress
v_{max}	=	5.55	N/mm²	: IRS, Table 26: Maximum Shear Stress

O.K.

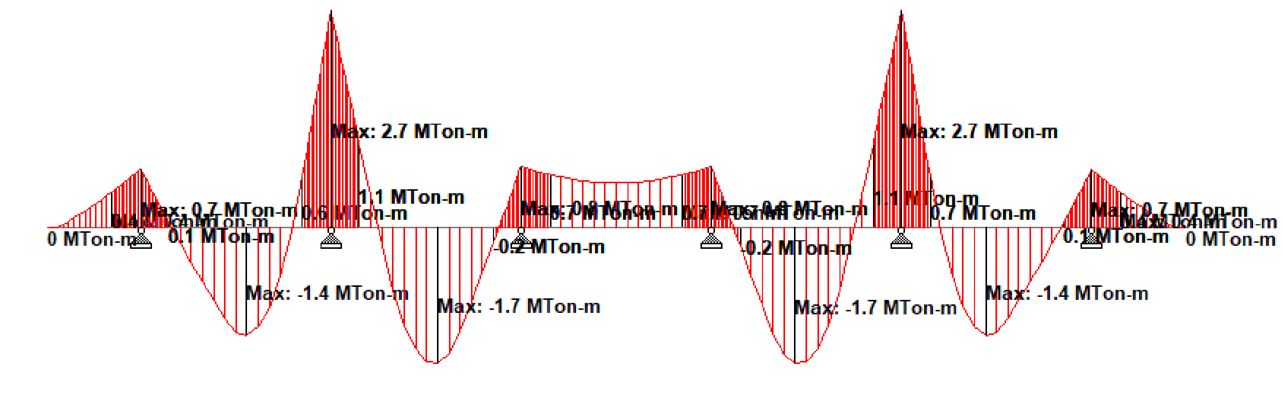
4.7 ANALYSIS RESULTS OF T-GIRDER DECK SLAB TRANSVERSELY:-

4.7.1 STAAD Output for 10.55 width of Deck Slab

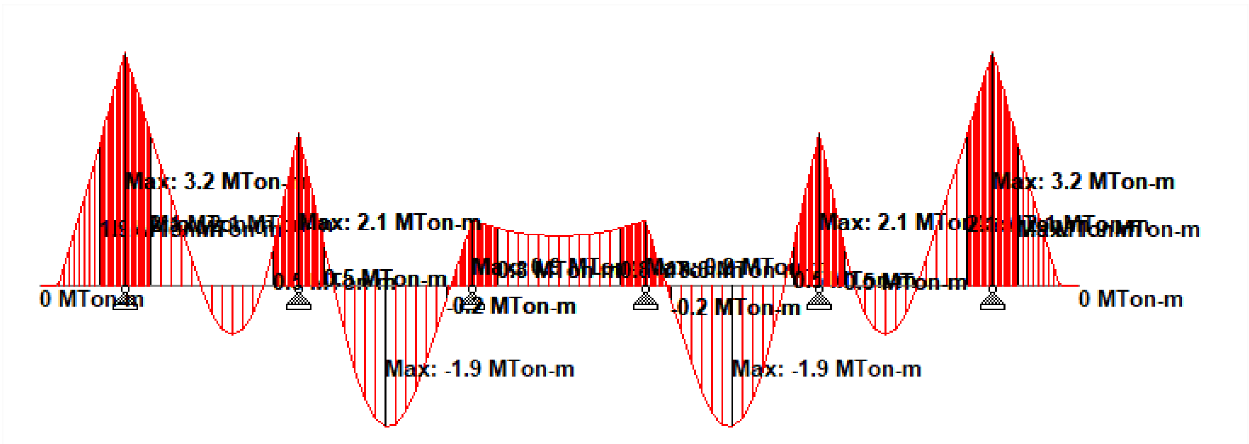
For design of slab at critical Section B, thickness of 200mm is considered (only depth of slab) conservatively & Section A thickness of slab considered 355mm.



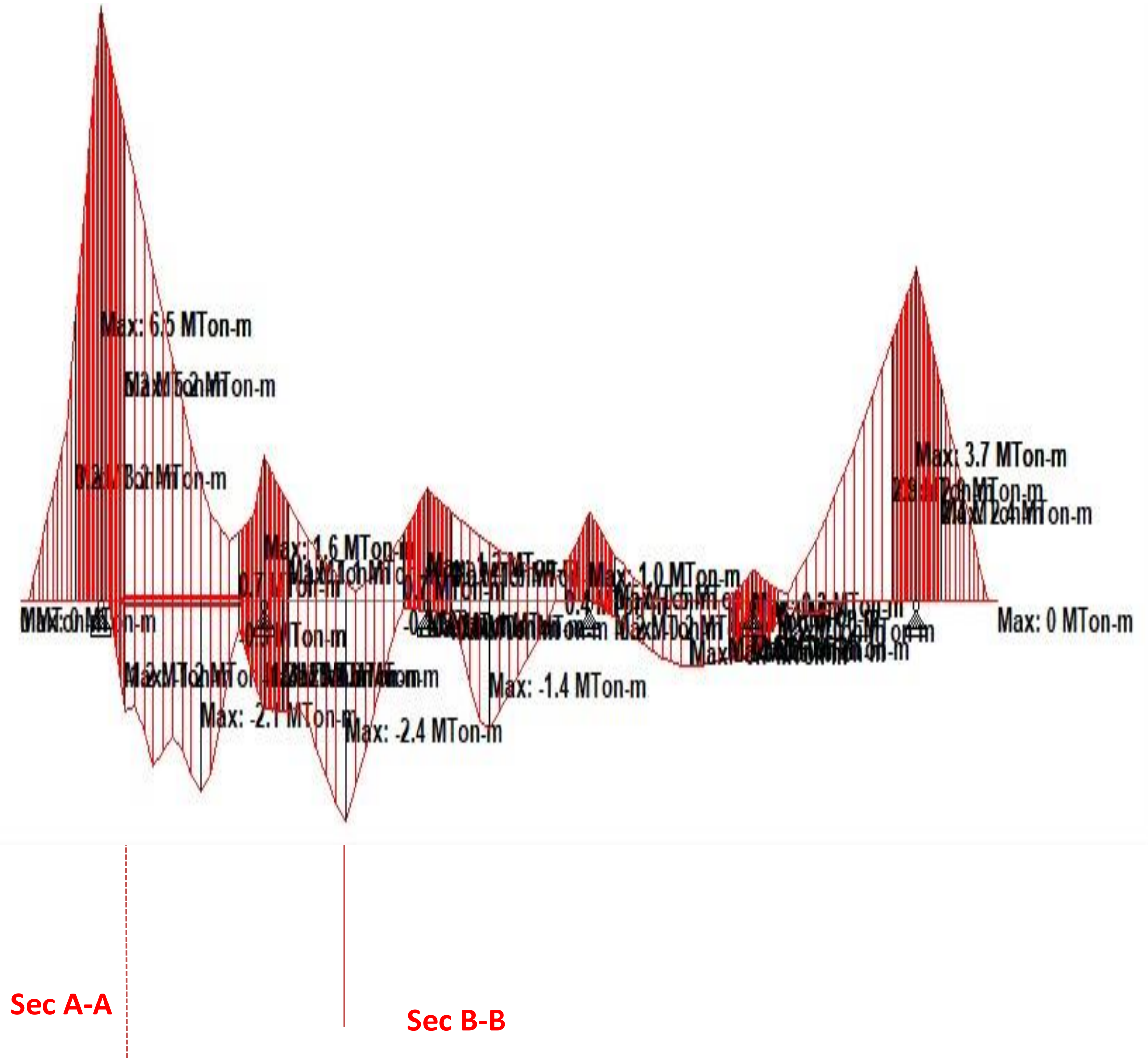
Bending Moment(Without-OHE) Due To SLS Envelope Of DL+1.2SIDL+1.1LL



Bending Moment(With-Ohe) Due To SLS Envelope Of DL+1.2SIDL+1.1LL



Bending Moment In Derailment Case Due To SLS Envelope



4.7.1 Summary of Critical Bending Moments & Bar Mark considered in design: -

Section	Bar Mark	Spacing	Dia.
Sec A-A (Max HOGGING)	1	150mm C/C	10
	3B	150mm C/C	10
Sec B-B (Max SAGGING)	2A	300mm C/C	10
	2B	300mm C/C	12

Summary of Critical Bending Moments			
Bending Moment	***Section***	***Position***	***Value of BM (T-m)***
Max HOGGING	A-A	at 0.265m from the support	5.20
Max SAGGING	B-B	at mid from support	2.40

4.7.1.1 Stress Check at Critical Locations

4.7.1.1 At Section A-A (Max Hogging)

Width b	1.000 m	n=Ea/Ec	6.22	
Depth h	0.355 m	Axial force N	-0.1	T (+ in comp)
		Bending moment M	5.20	T.m

<u>STRESS CALCULATION</u>

	Number		Area (m^2)	Diameter (mm)	di (m)	s (MPa)
As1	13.33	10	0.0010		0.305	**-172.78**
As2	0	0	0.0000		0.045	**8.16**
As3	0	0	0.0000		0.000	**39.48**
As4	0	0	0.0000		0.000	**39.48**
As5	0	0	0.0000		0.000	**39.48**
As6	0	0	0.0000		0.000	**39.48**
As7	0	0	0.0000		0.000	**39.48**
As8	0	0	0.0000		0.000	**39.48**

Alpha=d_c/d_1	0.1860	c (Mpa)
Depth of concrete in compression d_c (m)	0.057	**6.34**
Capable bending moment M (T.m)	5.16	

<u>CRACK WITH CALCULATION</u>

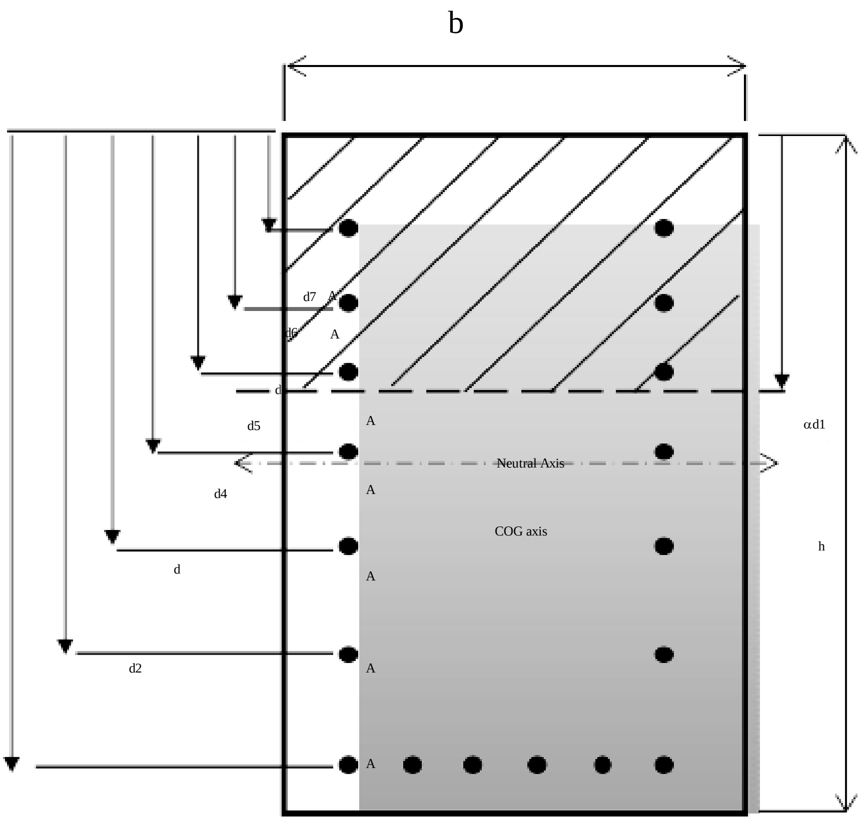
INPUT PARAMETERS

h	0.355 m	: Depth of section
bt	1.000 m	: Breadth of section
dc	0.057 m	: Depth of concrete in compression
d1	0.010 m	: Diameter of main reinforcement
d2	0.000 m	: Diameter of stirrup
As	0.0010 m^2	: Area of tension reinforcement
cmin	0.040 m	: Clear cover to the outermost reinforcement
cnom	0.025 m	: Nominal cover for crack width calculation
s	0.000864	: Maximum strain in steel

OUTPUT PARAMETERS

Mq/Mg	1	: Moment due to LL / Moment due to permanent loads
e	0.150 m	: Spacing between main reinforcement bars
a'	0.340 m	: Depth to the surface where cracking is estimated
acr	0.078 m	: Distance from cracking surface to nearest main bar
ε_1	0.000986	: Strain at face where cracking is estimated (+ : tensile strain)
ε_m	0.000986	: Strain allowing the stiffening effect of the concrete

w **<u>0.170</u> <u>mm</u>** **: Design crack width**



4.7.2.1 At Section B-B (Max Hogging)

Width b	1.000 m	n=Ea/Ec	6.22	
Depth h	0.200 m	Axial force N	-0.1	T (+ in comp)
		Bending moment M	2.4	T.m

STRESS CALCULATION

	Number		Area (m^2)	di (m)	σs (MPa)
		Diameter (mm)			
As1	3.33	10	0.0003	0.150	**-264.56**
As2	3.33	12	0.0004	0.150	**-264.56**
As3	0	0	0.0000	0.000	**68.10**
As4	0	0	0.0000	0.000	**68.10**
As5	0	0	0.0000	0.000	**68.10**
As6	0	0	0.0000	0.000	**68.10**
As7	0	0	0.0000	0.000	**68.10**
As8	0	0	0.0000	0.000	**68.10**

Alpha=d_c/d_1	0.2047	σc (Mpa)
Depth of concrete in compression d_c (m)	0.031	**10.94**
Capable bending moment M (T.m)	2.35	

CRACK WITH CALCULATION

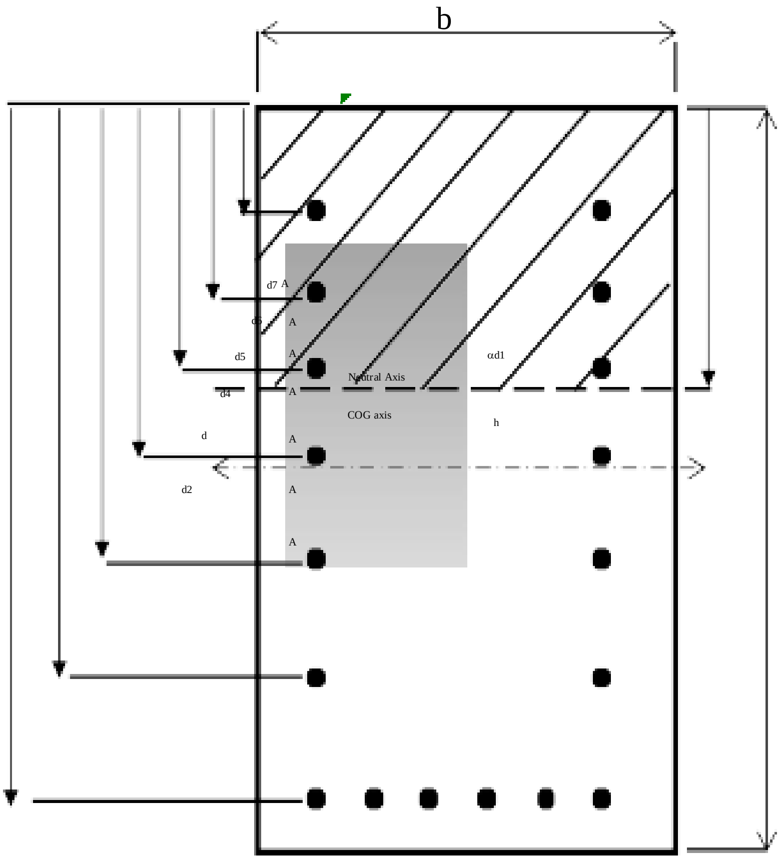
INPUT PARAMETERS

h	0.200 m	: Depth of section
bt	1.000 m	: Breadth of section
dc	0.031 m	: Depth of concrete in compression
d1	0.010 m	: Diameter of main reinforcement
d2	0.012 m	: Diameter of stirrup
As	0.0006 m^2	: Area of tension reinforcement
cmin	0.040 m	: Clear cover to the outermost reinforcement
cnom	0.025 m	: Nominal cover for crack width calculation
ε_s	0.001323	: Maximum strain in steel

OUTPUT PARAMETERS

Mq/Mg	1	: Moment due to LL / Moment due to permanent loads
e	0.150 m	: Spacing between main reinforcement bars
a'	0.185 m	: Depth to the surface where cracking is estimated
acr	0.077 m	: Distance from cracking surface to nearest main bar
ε_1	0.001711	: Strain at face where cracking is estimated (+ : tensile strain)
ε_m	0.001711	: Strain allowing the stiffening effect of the concrete

w **<u>0.244mm</u>** **: Design crack width**



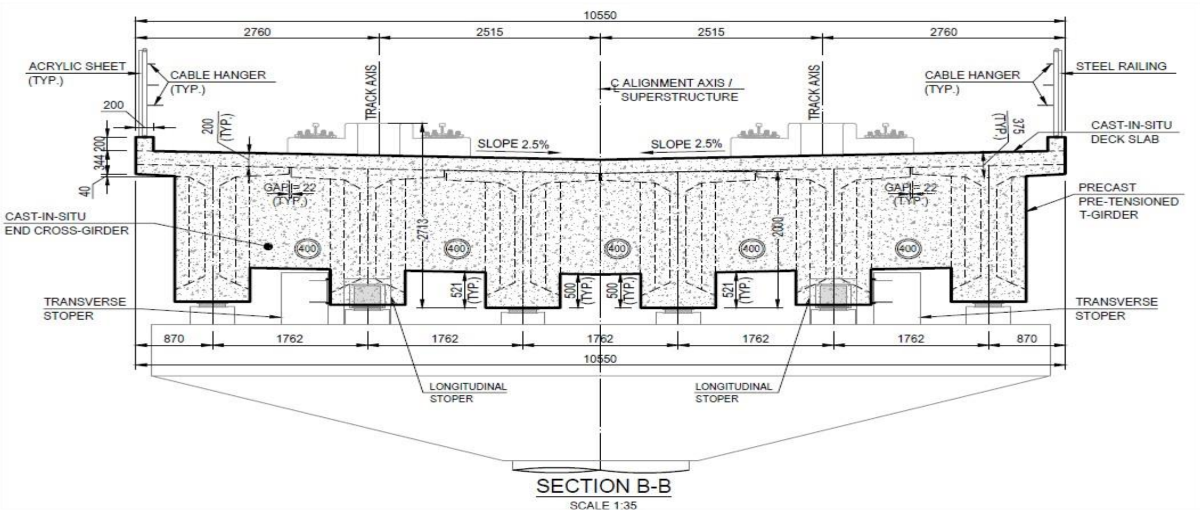
SUMMARY OF RESULT CHECKS

Summary of Reinforcement & Stresses								
Section	*Reinforcement Provided*	*Stresses in Concrete*		*Stresses in Steel*		*Crack Width*		*Remarks m*
		Applied(Mpa	*Allowable(Mpa)*	*Applied(Mpa*	*Allowable(Mpa)*	*Applied(mm)*	*Allowable (m*	
Sec A-A (Max HOGGING)	Φ10@150C/C BUNDLED	*6.34*	22.50	-172.78	375	0.1699	0.25	**OK**
	Φ0@150C/C							
Sec B-B (Max SAGGING)	Φ10@150C/C ALTERNATE	*10.94*	22.50	-264.56	375	0.2445	0.25	**OK**
	Φ12@150C/C ALTERNATE							

4.8 DESIGN OF CAST-IN-SITU DIAPHRAGM

4.8.1 Structure Description

Cast –In –Situ Diaphragms (400 mm thick width) are provided at each end bearing locations The Cross-section view is shown below.

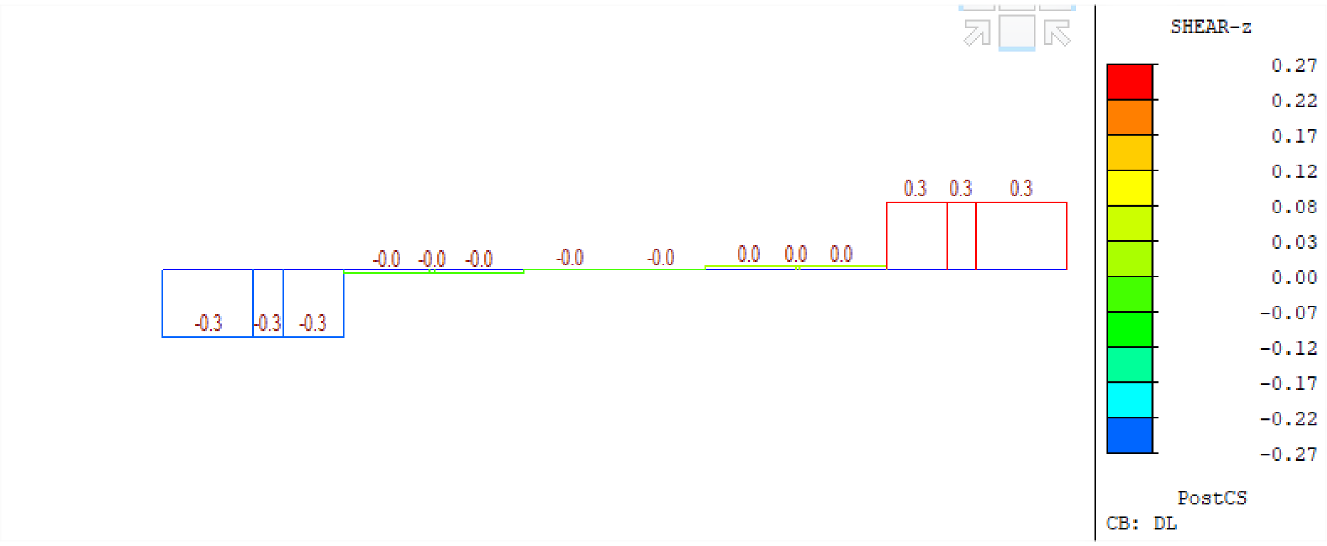


Cross-Section View of Diaphragm

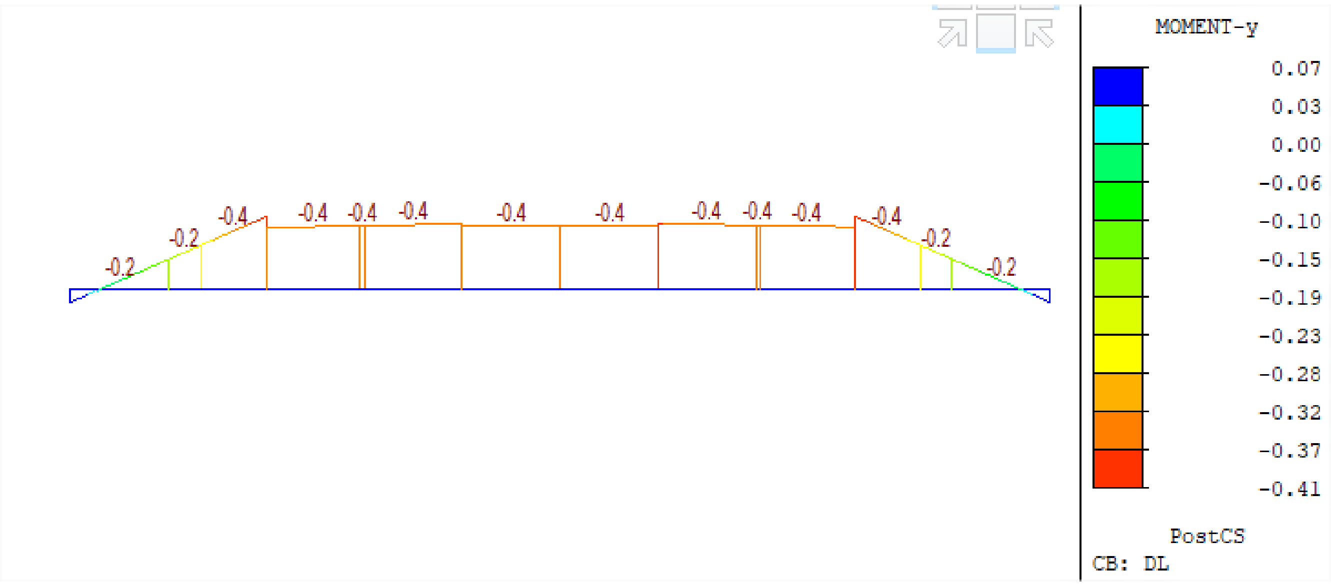
For more details of structure description please refer to Drawing Nos. BIC-F-STR-DC01-05001 to 05003

4.9 MIDAS CIVIL OUTPUT RESULTS

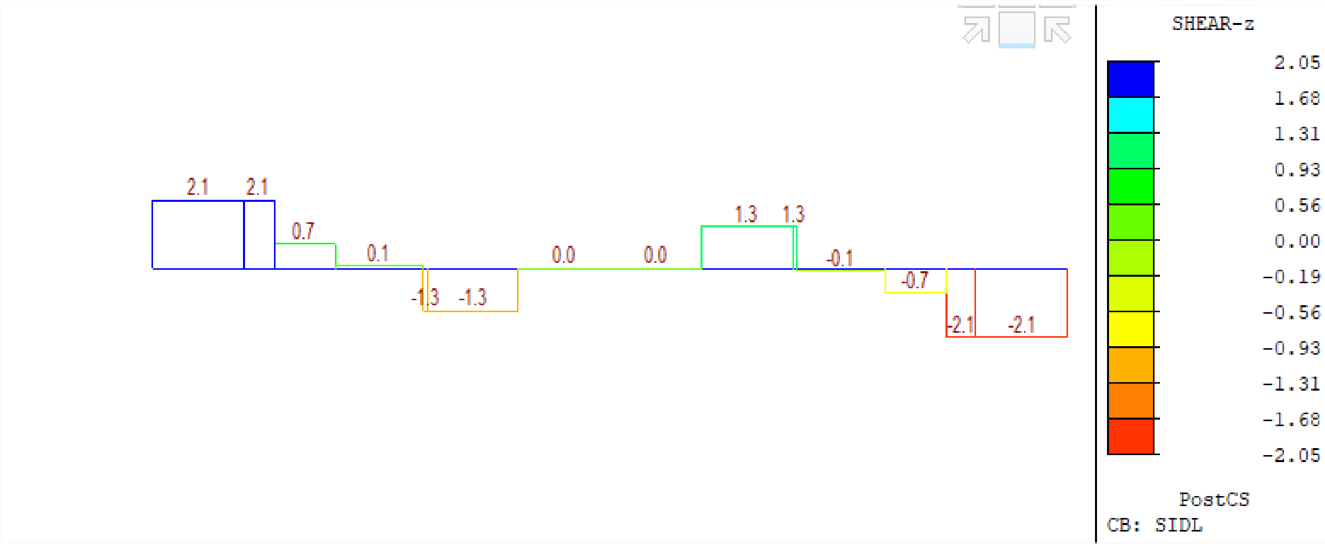
4.9.2 Shear Force on End Diaphragm due to DL (Units: T)



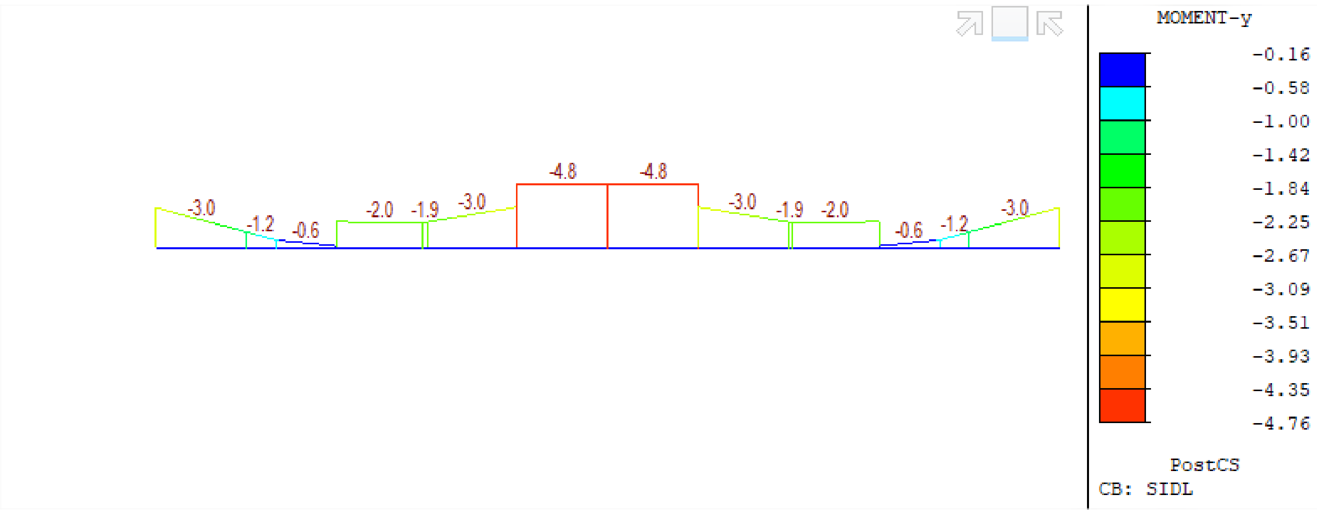
4.9.3 Bending Moment on End Diaphragm due to DL (Units: T – m)



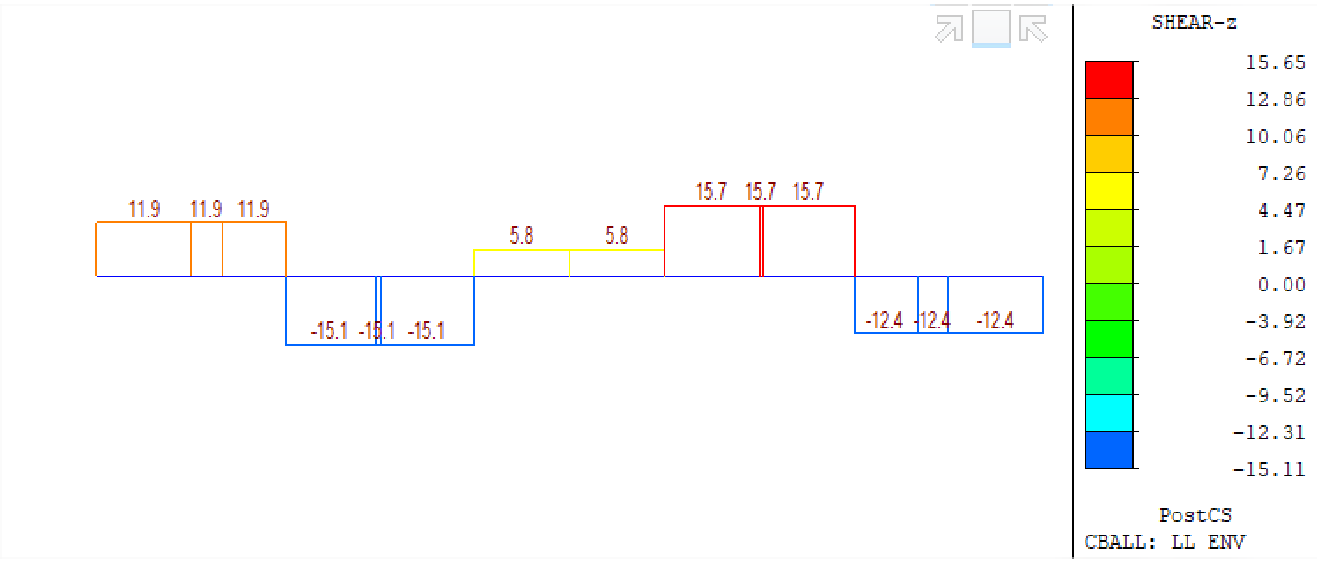
4.9.4 Shear Force on End Diaphragm due to SIDL (Units: T)



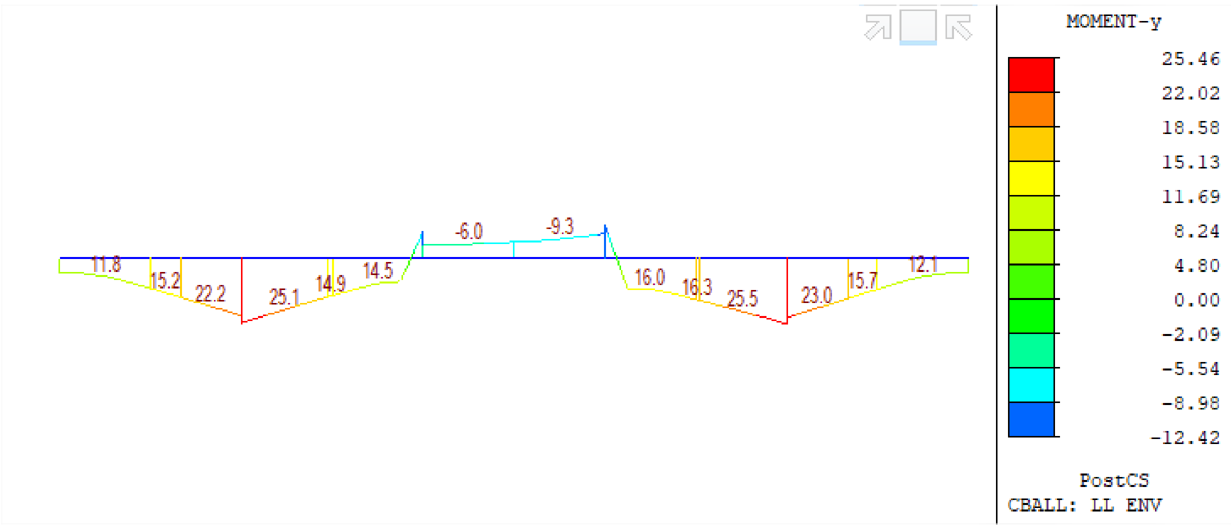
4.9.5 Bending Moment on End Diaphragm due to SIDL (Units: T – m)



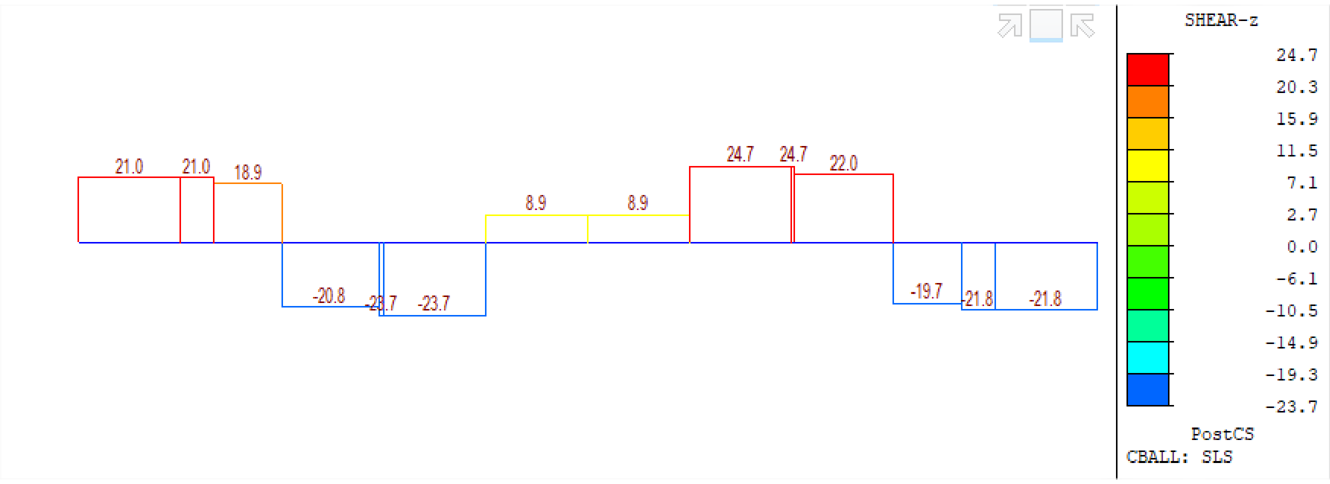
4.9.6 Shear Force on End Diaphragm due to LL (Units: T)



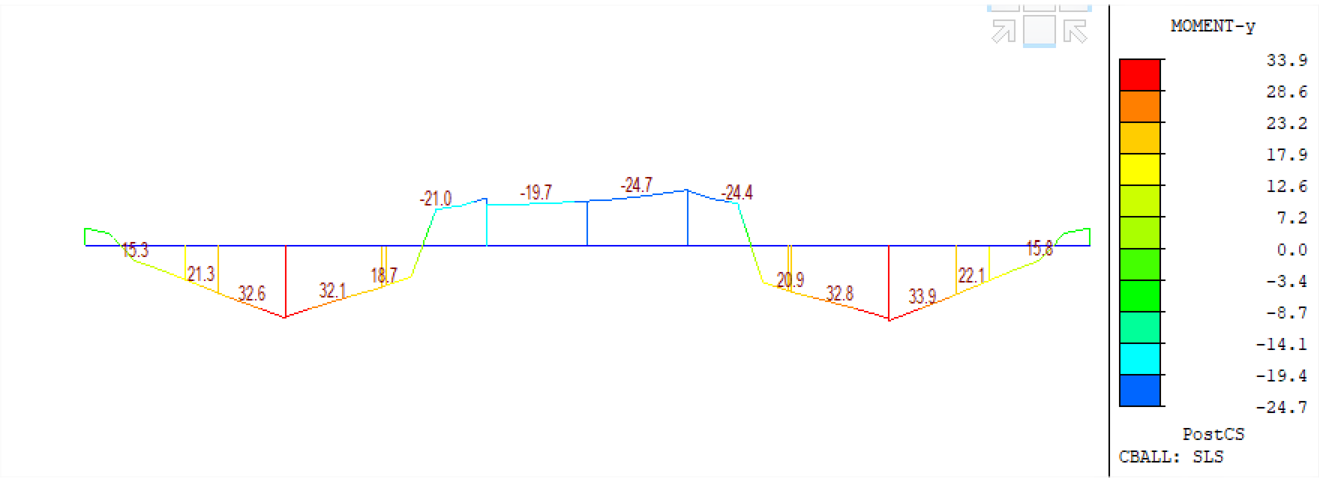
4.9.6 Bending Moment on End Diaphragm due to LL (Units: T – m)



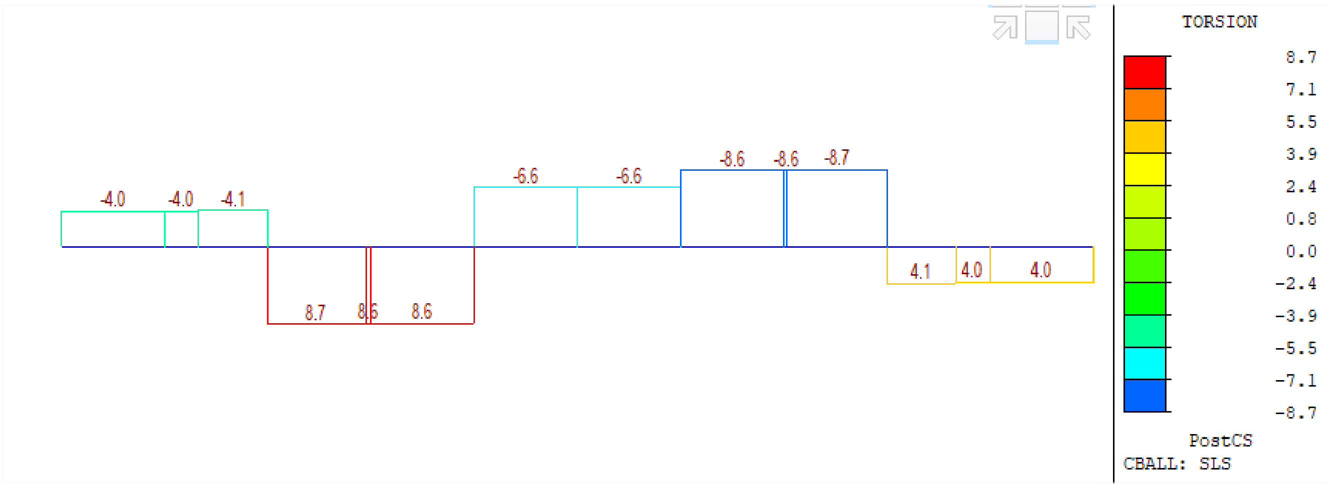
4.9.7 Shear Force on End Diaphragm due to SLS (Units: T)



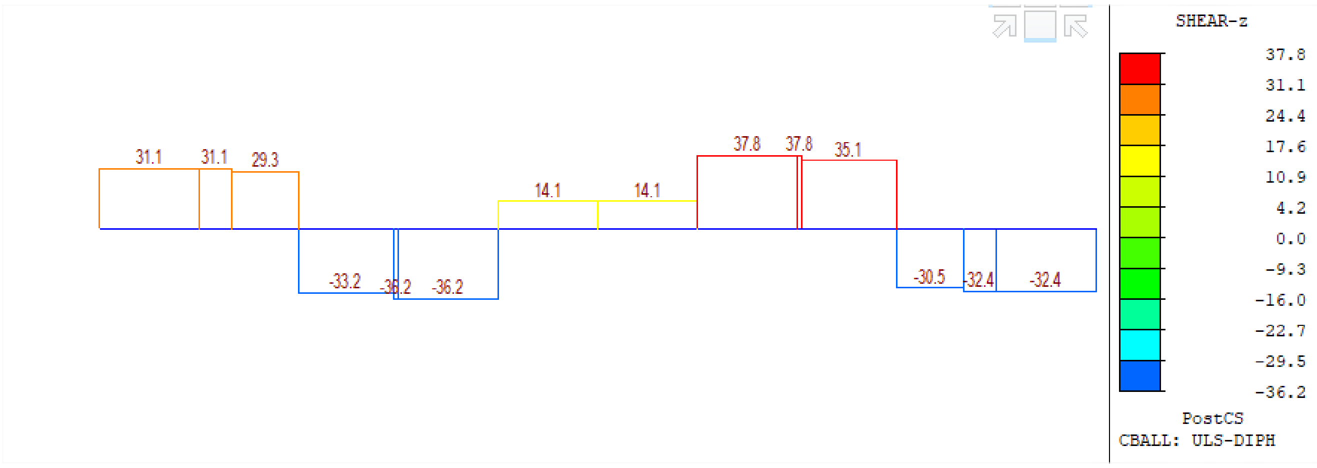
4.9.8 Bending Moment on End Diaphragm due to SLS (Units: T – m)



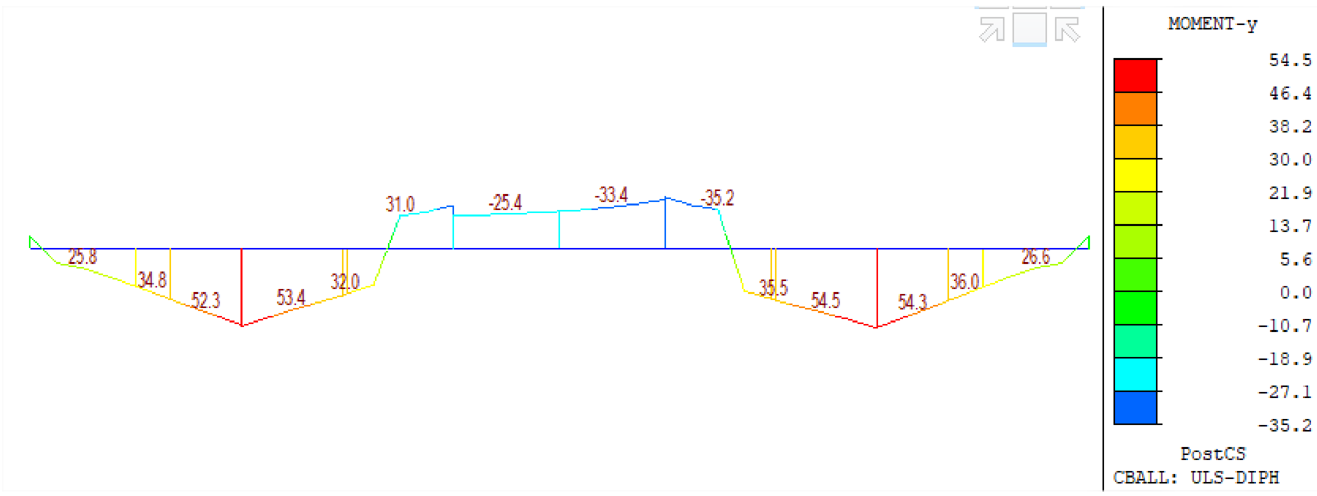
4.9.9 Torsional moment on End Diaphragm due to SLS (Units: T – m)



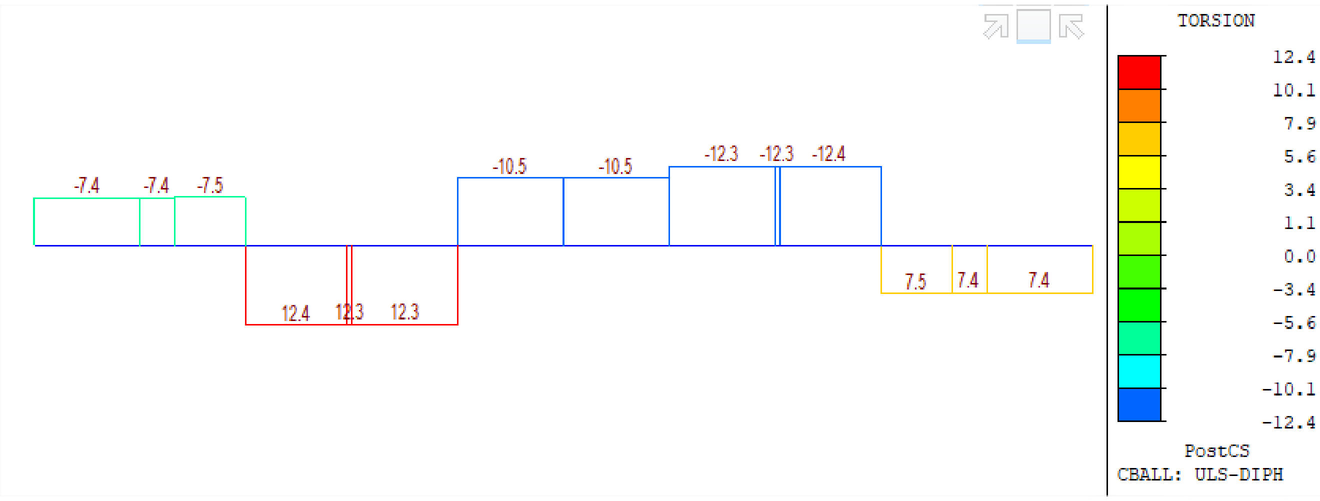
4.9.10 Shear Force on End Diaphragm due to Ultimate Load (Units: T)



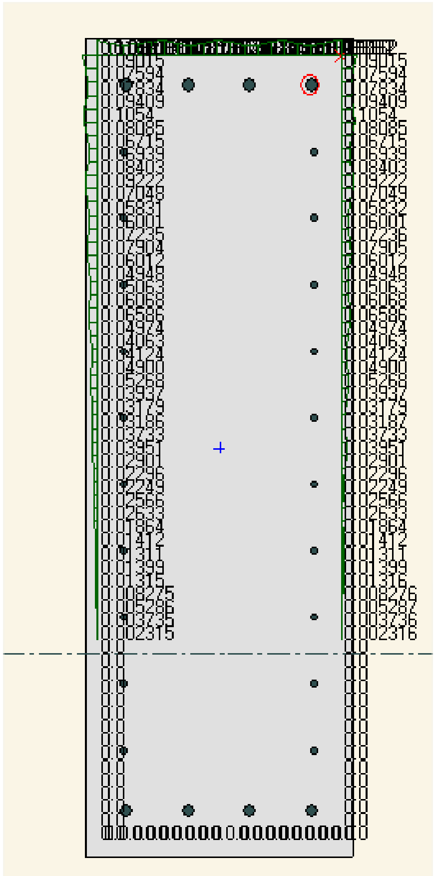
4.9.11 Bending Moment on End Diaphragm due to Ultimate Load (Units: T – m)



4.9.12 Torsional moment on End Diaphragm due to Ultimate Load (Units: T – m)



4.10 OASYS SECTION CHECK FOR END DIAPHRAGM – (Max Hogging Moment)



Stresses in Concrete (SLS Factors)

Section Material Stresses/Strains at SLS Loads

Case	Bar	Coordinates y [mm]	Coordinates z [mm]	Strain [-]	Stress [MPa]
Maxima					
1	4	1181.	-667.6	254.7E-6	4.095
1	4	1181.	-667.6	254.7E-6	4.095
Minima					
1	2	1581.	562.4	-772.0E-6	0.0
1	1	1181.	562.4	-772.0E-6	0.0

Maximum stresses in Concrete *(= 4.095 Mpa)* <0.5fck *(0.5*45 = 22.5Mpa).* **O.K.**

- **Stresses in Steel (SLS Factors)**

Reinforcement Stresses/Strains at SLS Loads

Case	Bar	Coordinates y [mm]	Coordinates z [mm]	Strain [-]	Stress [MPa]	
Maxima						
1	12	1243.	-597.6	196.3E-6	39.26	FE500
1	12	1243.	-597.6	196.3E-6	39.26	FE500
Minima						
1	22	1519.	492.4	-713.6E-6	-142.7	FE500
1	22	1519.	492.4	-713.6E-6	-142.7	FE500

Maximum stresses in Steel *(=142.7 Mpa)* < 0.5fy *(0.75*500 = 375 Mpa).* **O.K.**

- **Crack Width Results (SLS Factors)**

Case	Face	Point	Coordinates y [mm]	Coordinates z [mm]	Strain E_m	Strain E_1	b_t	Control Bar	a_{cr} [mm]	Cover c_{min} [mm]	Cover From	h [mm]	x [mm]	Crack Width [mm]
Maxima														
1	1	1	1564.	537.4	-751.1E-6	-751.1E-6	0.4000	22	53.65	52.00	Face 2	1230.	305.2	0.1162

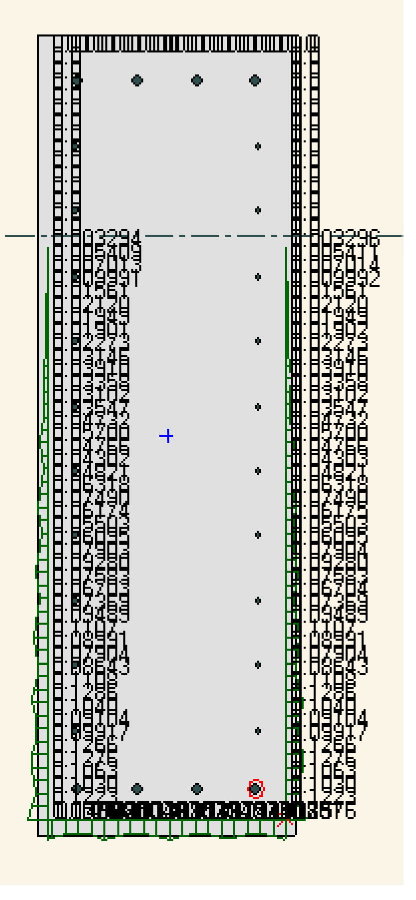
Maximum Crack Width *(=0.116mm)* < 0.250 mm. **O.K.**

- **ULS Check**

Case	Eff. Centroid (y)	Eff. Centroid (z)	N [kN]	M [kNm]	M_u [kNm]	M/M_u	Governing Condition
Maxima							
1	66.61	-2.650	0.0	400.0	1123.	0.3562	B: Node 4
Minima							
1	66.61	-2.650	0.0	400.0	1123.	0.3562	B: Node 4

Ratio of M/Mu *(=0.356)* < 1.000. **O.K.**

4.11 OASYS SECTION CHECK FOR END DIAPHRAGM – (Max Sagging Moment)



- **Stresses in Concrete (SLS Factors)**

Section Material Stresses/Strains at SLS Loads

Case	Bar	Coordinates y [mm]	z [mm]	Strain [-]	Stress [MPa]
Maxima					
2	1	1181.	562.4	347.8E-6	5.591
2	1	1181.	562.4	347.8E-6	5.591
Minima					
2	3	1581.	-667.6	-0.001047	0.0
2	3	1581.	-667.6	-0.001047	0.0

Maximum stresses in Concrete *(=5.591 Mpa)* < 0.5fck *(0.5*45 = 22.5Mpa).* **O.K.**

- **Stresses in Steel (SLS Factors)**

Reinforcement Stresses/Strains at SLS Loads

Case	Bar	Coordinates y [mm]	Coordinates z [mm]	Strain [-]	Stress [MPa]	
Maxima						
2	10	1243.	492.4	268.4E-6	53.67	FE500
2	10	1243.	492.4	268.4E-6	53.67	FE500
Minima						
2	24	1519.	-597.6	-967.9E-6	-193.6	FE500
2	24	1519.	-597.6	-967.9E-6	-193.6	FE500

Maximum stresses in Steel *(=193.6Mpa)* < 0.5fy *(0.75*500 = 375 Mpa).* **O.K.**

- **Crack Width Results (SLS Factors)**

Case	Face	Point	Coordinates y [mm]	Coordinates z [mm]	Strain E_m	Strain E_1	b_t	Control Bar	a_{cr} [mm]	Cover c_{min} [mm]	Cover From	h [mm]	x [mm]	Crack Width [mm]
Maxima														
2	2	2	1564.	-642.6	-0.001019	-0.001019	0.4000	24	53.65	52.00	Face 2	1230.	306.6	0.1576

Maximum Crack Width *(=0.157mm)* < 0.250 mm. **O.K.**

- **ULS Check**

Case	Eff. Centroid (y)	Eff. Centroid (z)	N [kN]	M [kNm]	M_u [kNm]	M/M_u	Governing Condition
Maxima							
2	66.61	-2.650	0.0	619.0	1133.	0.5462	B: Node 1
Minima							
2	66.61	-2.650	0.0	619.0	1133.	0.5462	B: Node 1

Ratio of M/Mu *(=0.562)* < 1.000. **O.K**

4.12 ULTIMATE SHEAR & TORSION

4.12.1 Torsion Reinforcement (links)

To IRS CBC, Sections 16.4.5.2 & 15.4.4.

If v_t> 0.42 MPa (Table 17) torsion reinforcement shall be provided as follows :

$$\frac{A_{st}}{s_v} = \frac{T}{1.6\, x_1 y_1 0.87\, f_{yv}}$$

Where A_{st} is the area of **one** leg of a closed stirrup

x_1 and y_1 are the smaller centre line dimensions of the stirrups.

Note:

As per IRS CBC, Section 15.4.4.5, we are allowed to reduce the links area by (up to) 20% provided the longitudinal steel is increased by 25% (such that the product remains constant).

4.12.2 Longitudinal Torsion Reinforcement

According to IRS CBC, Sections 16.4.5.2 & 15.4.4 longitudinal torsion reinforcement shall be

$$\frac{A_{sL}}{s_L} = \frac{A_{st}}{s_v}$$

provided as (Since same f_y for closed links & longitudinal rebars)

This can also be written as :

$$(A_{sL})_{TOTAL} = \frac{A_{st}}{s_v} \cdot perimeter$$

4.12.3 Shear Stresses (ULS)

We check the total shear stresses due to bending & torsion (□+□ₓ) at ULS.

For shear stress calculation the maximum possible factored shear stress is calculated corresponding to the ULS- GI load combination : -1.25*DL +2*SIDL+1.75*LL

To IRS CBC, Sections 16.4.5.2 & 15.4.3.1, the shear stress is calculated from

$$\zeta = \frac{V}{bd}$$

Where,

b = Minimum breadth of the section, and

d = effective depth of the section (max of 0.8h, effective depth to cable CG)

For torsional shear stress calculation refers to IRS CBC, Sections 16.4.5.2. According to IRS CBC, Sections 15.4.4.4 (b)

$$\zeta_x = \frac{2T}{h^2_{min} . (h_{max} - \frac{h_{min}}{3})}$$

Where T is the torsional moment due to ultimate load, h_{min} is the smaller dimension of the section, h_{max} is the larger dimension of the section

We then check for (refer to IRS CBC, Sections 15.4.4. Table 17)

$\square_x + \square_y < 4.75$MPa

4.12.4 Verification of Ultimate Shear and Torsion

Ultimate shear checks is accordance with IRS Concrete Bridge Code, 1997,§ 15.4.3.

Ultimate Torsion checks is accordance with IRS Concrete Bridge Code, 1997, § 15.4.4.

4.12.5 **Ultimate shear & Torsion check**

SHEAR CHECK (As per IRS-CBC, 1997, § 15.4.3)

Ultimate Shear Force, **V** = 37.8 T

		B		**D**	
Dimension of Cross-Girder	=	0.4	m	1.389	m
Effective dimension	=	0.400	m	1.343	m

Shear stress, **v** = 70 T/m2 0.704 N/mm2 **OK**

%Tension reinforcement = 0.94

Ultimate shear stress, v_c 0.724 N/mm2

Depth factor, **s** **0.781** (According to table 16 of IRS-CBC 1997)

Shear Reinforcement required **(v > s*vc)**

SHEAR REINFORCEMENT

1 If $v \le sv_c$ A_{sv} = $0.4 \cdot b \cdot s_v / 0.87 \cdot f_y$

2 If $v > sv_c$ A_{sv} = $b \cdot s_v \cdot (v+0.4-sv_c) / 0.87 \cdot f_y$

Where

A_{sv} =Cross-sectional area of all the legs of the stirrup/links at a particular cross section

s_v = Spacing of the stirrups along the member

b = Breadth of the section

s_v = 100 mm

f_y = 415 N/mm2

Required A_{sv} = 60 mm2

TORSION CHECK (As per IRS-CBC, 1997, § 15.4.4)

From equation 9a, IRS - CBC, 1997, § 15.4.4.4

Vt = 2 * T/(hmin^2 * (hmax - hmin/3))

Where,

hmin = smaller dimension of the section

hmax = larger dimension of the section

T = Ultimate TorsionalMoment in the section

Ultimate Torsion Moment , **T** = 12.4 Tm

		hmin		**hmax**	
Dimension of Cross-Girder	=	0.4	m	1.389	m
		x1		**y1**	
Dimension of Stirrups	=	0.318	m	1.307	m (Refer Drawings for details)

Torsional stress, **Vt** = 123 T/m2 1.210 N/mm2 **OK**

Total Ultimate stress, **Vt + V** = **1.911** N/mm2

Permissible Ultimate Stress, **Vtu =** 4.750 (According to Table 17 of IRS-CBC 1997)

OK **(V+Vt < Vtu)**

TORSIONAL REINFORCEMENT

s_v = 100 mm

f_y = 415 N/mm2

From equation 10a, IRS - CBC, 1997, § 15.4.4.4

Ast/Sv >= T/(1.6* x1 * y1 * (0.87*fy))

Where,

A_{st} =Cross-sectional area of all the legs of the stirrup/links at a particular cross section

s_v = Spacing of the stirrups along the member

x1 = smaller centre line dimension of the stirrups

Required **Ast** = 50.7 mm^2

Therefore, Total Reinforcement Reqd, = **Asv + Ast** = 161 mm^2 (Shear + Torsion)

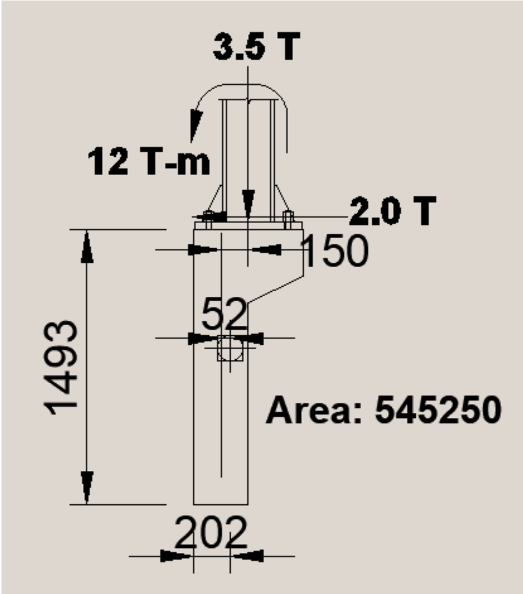
We assume 2 x **T 12** mm

Provided $A_{sv} + A_{st}$ 226 mm2

OK

4.13 CALCULATION FOR OHE PEDESTAL

The OHE are critical and following is considered in design :-



- Vertical load of OHE Mast = 3.5 Ton
- Horizontal load of OHE Mast = 2.0 Ton
- Moment due to OHE Mast = 12 Ton-m
- Moment due to Self-Weight of Parapet

M = 0.546 x 0.6 x 2.55*0.052 = 0.044 T-m

- Moment due to OHE-Mast

M = (3.5*0.15 + 2*1.493 + 12)
= 15.511 T-m

Total Moment, M = 0.044 + 15.511 = 15.555 T-m

Width b	0.600	m	n=Ea/Ec	10	
Depth h	0.600	m	Axial force N	-0.1	T (+ in comp)
			Bending moment M	15.555	T.m

STRESS CALCULATION

	Number	Diameter (mm)	Area (m²)	di (m)	□s (MPa)	*Stress limit*
As1	6	16	0.0012	0.544	**-256.14**	375
As2	0	10	0.0000	0.300	**-105.55**	375
As3	0	10	0.0000	0.056	**45.04**	375
As4	0	0	0.0000		**79.60**	375
As5	0	0	0.0000		**79.60**	375
As6	0	0	0.0000		**79.60**	375
As7	0	0	0.0000		**79.60**	375
As8	0	0	0.0000		**79.60**	375
			Alpha= d_c/d_1	0.2371	□c (Mpa)	
			Depth of concrete in compression d_c (m)	0.1290	**7.96**	*22.50*
			Capable bending moment M (T.m)	15.46		

CRACK WIDTH CALCULATION

INPUT PARAMETERS

h	0.600	m	: Depth of section
bt	0.600	m	: Breadth of section
dc	0.129	m	: Depth of concrete in compression
d1	0.016	m	: Diameter of main reinforcement
d2	0.012	m	: Diameter of stirrup
As	0.0012	m²	: Area of tension reinforcement
cmin	0.040	m	: Clear cover to the outermost reinforcement
cnom	0.025	m	: Nominal cover for crack width calculation
□s	0.001281		: Maximum strain in steel

OUTPUT PARAMETERS

Mq/Mg	1		: Moment due to LL / Moment due to permanent loads
e	0.100	m	: Spacing between main reinforcement bars
a'	0.585	m	: Depth to the surface where cracking is estimated
acr	0.057	m	: Distance from cracking surface to nearest main bar
□1	0.001407		: Strain at face where cracking is estimated (+ : tensile strain)
□m	0.001407		: Strain allowing the stiffening effect of the concrete
w	**0.21**	**mm**	**: Design crack width**

CHAPTER 5

CONCLUSION

1. Bending moments and Shear force for PSC T-beam girder are lesser than RCC T-beam Girder Bridge.
2. PSC T-Beam Girder has less heavier section than RCC T-Girder for 37 m span
3. Shear force resistance of PSC T-Beam Girder is more compared to RCC T- Girder.
4. Deflection for PSC T-beam Girder is less than RCC T-Beam Girder Bridge.
5. T- Girder is having a simple shuttering and not required more skilled labours to carry out that task.
6. We have concluded that long term durability and strength wise PSC Girder is much strong than RCC Girder.

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