Addis Ababa Institute of Technology (AAIT)
School of Graduate Studies

School of Civil and Environmental Engineering
Stream: Rail way Engineering.

Analysis and Design of Pre-stressed Concrete (PSC) Girder Railway Bridge.

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An Independent Project Submitted to school of civil and environmental Engineering in Partial Fulfillment of the Requirements for the Degree of Masters of Engineering in Railway Engineering.

Advisor: - Dr. Abrham Gebre.

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The undersigned have examined the project entitled "**Analysis and Design of Pre-stressed Concrete (PSC) Girder Railway Bridge**'' presented by Temesgen Dola, a candidate for the degree of **Master of Engineering** and here by certify that it is worthy of acceptance.

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Abstract

A bridge is a structure that crosses over a river, bay, or other obstruction, permitting the smooth and safe passage of vehicles, trains, and pedestrians.

Bridge construction today has achieved a worldwide level of importance. Bridges are the key elements in any road network. Use of concrete girder is gaining popularity in railway bridge engineering because of its better stability, serviceability, economy, aesthetic appearance and structural efficiency. The structural behavior of box girder is complicated, which is difficult to analyze in its actual conditions by conventional methods. In this project a simply supported Box Girder Bridge made of pre stressed concrete is analyzed for moving loads following different standards and specifications.

The Ethiopian Government has constructed Addis Ababa light rail transit (AALRT) project and also has started the construction of national railway project for different corridors of the country is under construction with CREC (China Railway Engineering Company) and another foreign (international) contractors.

The railway bridge commonly contains two types of tracks which are ballasted and slab track.

This project principally emphases on the superstructure of bridge supporting track rails attached to the bridge to support trains.

Thus, this project has attempted to develop the Micro soft Excel program to analyze and design the Pre-stressed Concrete (PSC) Girder Railway Bridge for any span length.

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List of abbreviations

AREMA = American Railway Engineering Maintenance of Way Association.

DL = Dead load.

EBCS = Ethiopian building code standard.

LL = Live load.

LRT= Light rail transit.

PSC = Pre-stressed concrete.

RC = Reinforced concrete.

SDL= Superimposed dead load.

ULS = Ultimate limits state.

ACI = American concrete institute.

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1.0 INTRODUCTION

BACK GROUND

A bridge is a structure that crosses over a river, bay, or other obstruction, permitting the smooth and safe passage of vehicles, trains, and pedestrians. An elevation view of a typical bridge is A bridge structure is divided into an upper part (the superstructure), which consists of the slab, the floor system, and the main truss or girders, and a lower part (the substructure), which are columns, piers, towers, footings, piles, and abutments. The superstructure provides horizontal spans such as deck and girders and carries traffic loads directly. They support the horizontal spans, elevating above the ground surface.

Pre-stress concrete is ideally suited for the construction of medium and long span bridges. Ever since the development of pre-stressed concrete by Freyssinet in the early 1930s, the material has found extensive application in the construction of long-span bridges, gradually replacing steel which needs costly maintenance due to the inherent disadvantage of corrosion under aggressive environment conditions. One of the most commonly used forms of superstructure in concrete bridges is precast girders with cast-in-situ slab. This type of superstructure is generally used for spans between 20 to 40 m. In this project the AREMA Rail Road Loading considered for design of bridges, also factor which are important to decide the preliminary sizes of concrete box girders. Also considered the IRRC: 18-2000 for “Pre stressed Concrete Road Bridges” and “Code of Practice for Pre stressed Concrete” Indian Standard. Analyze the Concrete Box Girder Rail Road Bridges for various spans, various depth and check the proportioning depth.

Box-girder and T-girder Bridge have been a dominant bridge type in most of world’s country for railroad and highway for a long time because of their simple geometry, low fabrication cost, easy erection or casting and smaller dead loads. At present the China Railway Engineering Company (CREC) has been constructing the pre-stressed Box-girder bridge structures for Addis Ababa city Light rail Transit (LRT) and for National Rail road projects.

1.1. Statement of the problem

Railway overpasses are used to replace level crossings (at-grade crossings) as a safer alternative. Using overpasses allows for unobstructed rail traffic to flow without conflicting with vehicular and pedestrian traffic. Rapid transit systems use complete grade separation of their rights of way to avoid traffic interference with frequent and reliable service. Flyovers are important constructions that allow for more efficient and faster transport. They become a necessity when roads are congested because of heavy traffic and people of one locality find it difficult to move to another having to take diversions instead of getting a straight road that would save a lot of time and effort.

For the last decade the Ethiopian economy has been rapidly growing; to have continued this economic growth of the country fast and efficient transport system is a must. Due to this the government of Ethiopian has currently constructing the LRT for Addis Ababa city and national railway lines.

In Addis Ababa at the moment the main challenging problem to have continued this economic development is the traffic congestion due to high population growth and also high number of vehicles and pedestrian in the city. Constructing overpass (Rail Bridge) structure is the best option to solve the traffic congestion at grade level, to reduce traffic accidents at grade level and to overcome the topographic elevation difference. To construct such overpass (Rail Bridge) structure needs effective design and analysis and also needs to develop potential software.

The main focus of this project is to do develop the Micro soft Excel program for analysis and design of pre-stressed concrete rail Girder Bridge for any span length.

1.2. OBJECTIVE OF THE PROJECT

1.2.1. General Objective

The main objective of this project is to develop the Micro soft Excel program for analysis and design of the pre-stressed concrete girder railway bridge for any span length.

1.2.2. Specific Objective

The specific objective of this project is:-

- To analysis the pre-stressed concrete railway bridge by using Microsoft excel spread sheet.
- To develop the Microsoft excel to design the pre-stressed concrete girder railway bridge.
- To determine the overall size (dimension) of the girder.
- To come up with meaning full conclusion and recommendations for the future study.

1.3. METHODOLOGY

The methodology carrying out for this project will focusing on reviewing different journals from the Internet in addition to the locally available books especially AREMA and A. Adamu, 2012. Pre-stressed Concrete Railway Bridges, Addis Ababa, Ethiopia teaching materials on Pre-stressed concrete Rail Bridge and other journals that are related to this project.

This project has tried to included Critical assessment of the different analysis and design methods, will trying to determine depth of the girder and other cross sectional dimensions of the girder which is most economical.

This project basically focused on developing analysis and design of simply supported pre stressed precast-cast in-situ concrete railway bridges.

This project used micro soft Excel sheet, Auto CAD to draw the sections of the Girder and other necessary soft wares. AALRT bridge site visit conducted to have better understanding of the girder.

1.4. SCOPE OF THE PROJECT

The scope of this project has been limited to the preparation of analysis and design of pre-stressed precast-cast in-situ concrete railway bridges Also this study considered only single track, not covered dynamic analysis of bridge as well as track but the dynamic effect considered through impact factor, study based on only superstructure, the bridge is assumed to have a linear behavior, pre-stressed concrete is assumed to have an elastic behavior and to be un-cracked and lateral forces, lateral accelerations or displacements are not considered, some load effects, such as snow, water pressure, wind and centrifugal force are not considered.

1.5. STUDY AREA.

The study area of this project is conducted in view of the Addis Ababa light rail transit over pass rail Road Bridge. The total length of AALRT is 34.25 km including elevated girder bridge and at grade level tracks (north-south line 16.9 km and east-west line 17.35 km) two lines with double track. I.e. north-south and east-west lines use common track of about 2.7km (from Meskel square to St. Mariam) and it is standard gauge (1.435 meters).

CHAPTER -TWO

2. Literature Review

Concrete is very strong in compression but weak in tension. In an ordinary concrete beam the tensile stress at the bottom are taken by standard steel reinforcement: But we still get cracking, which is due to both bending and shear.

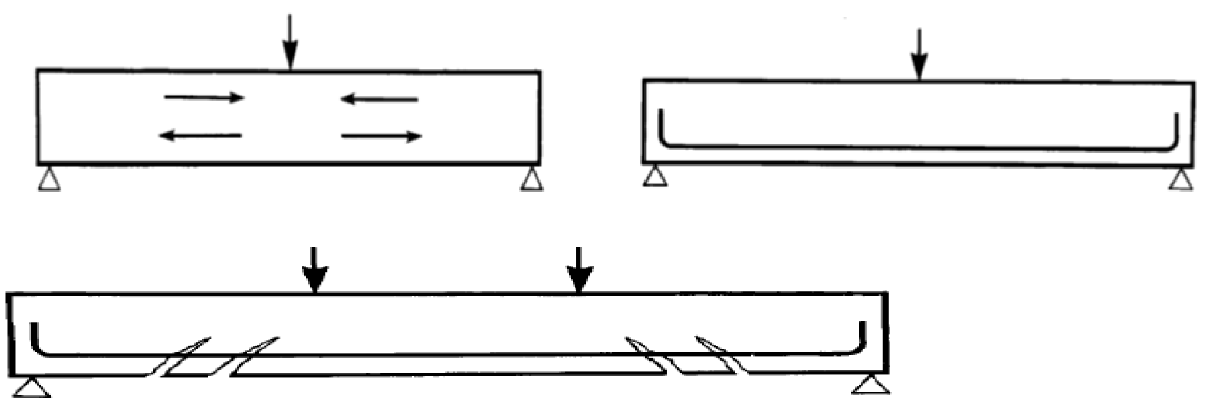


Fig 2.1 reinforced concrete

In pre-stressed concrete, because the pre-stressing keeps the concrete in compression, no cracking occurs. This is often preferable where durability is a concern.

Pre-stress is defined as a method of applying pre-compression to control the stresses resulting due to external loads below the neutral axis of the beam tension developed due to external load which is more than the permissible limits of the plain concrete. The pre-compression applied (may be axial or eccentric) will induce the compressive stress below the neutral axis or as a whole of the beam resulting either no tension or compression.

Pre-stressed concrete is basically concrete in which internal stresses of a suitable magnitude and distribution are introduced so that the stresses resulting from the external loads are counteracted to a desired degree.

Methods of Pre-Stressing

There are two methods of pre-stressing:

Pre-tensioning: Apply pre-stress to steel strands before casting concrete; and

Post-tensioning: Apply pre-stress to steel tendons after casting concrete.

1. Pre-Tensioning In the pre-tensioning systems, the tendons are first tensioned between rigid anchor-blocks cast on the ground or in a column or unit-mold types pre tensioning bed, prior to the casting of concrete in the mold. The tendons comprising individual wires or strands are stretched with constant eccentricity or a variable eccentricity with tendon anchorage at one end and jacks at the other. With the forms in place, the concrete is cast around the stressed tendon. The system is shown in Figure below.

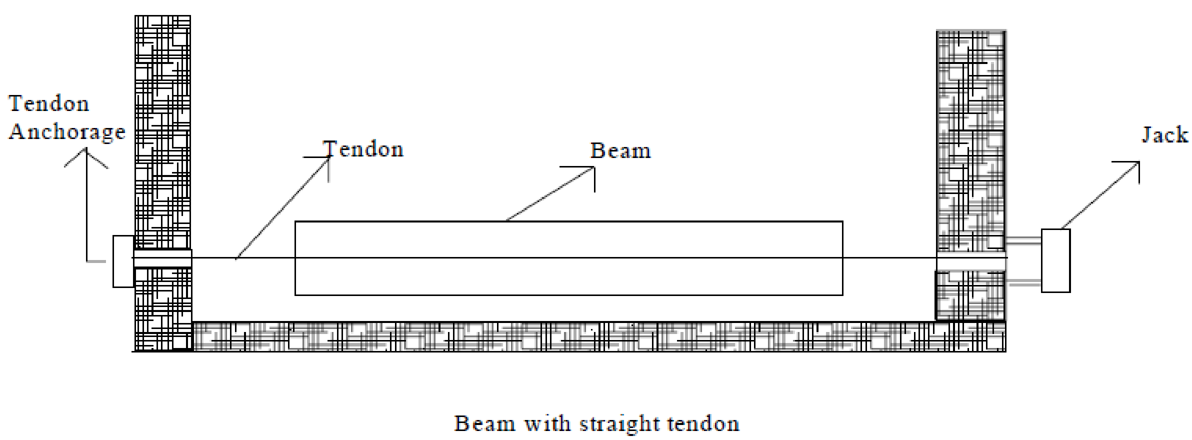


Fig. 2.2 pre- tensioning of tendon.

Pre-tensioning is the most common form for precast sections. In Stage 1, the wires or strands are stressed; in Stage 2, the concrete is cast around the stressed wires/strands; and in Stage 3, the pre-stressed in transferred from the external anchorages to the concrete, once it has sufficient strength.

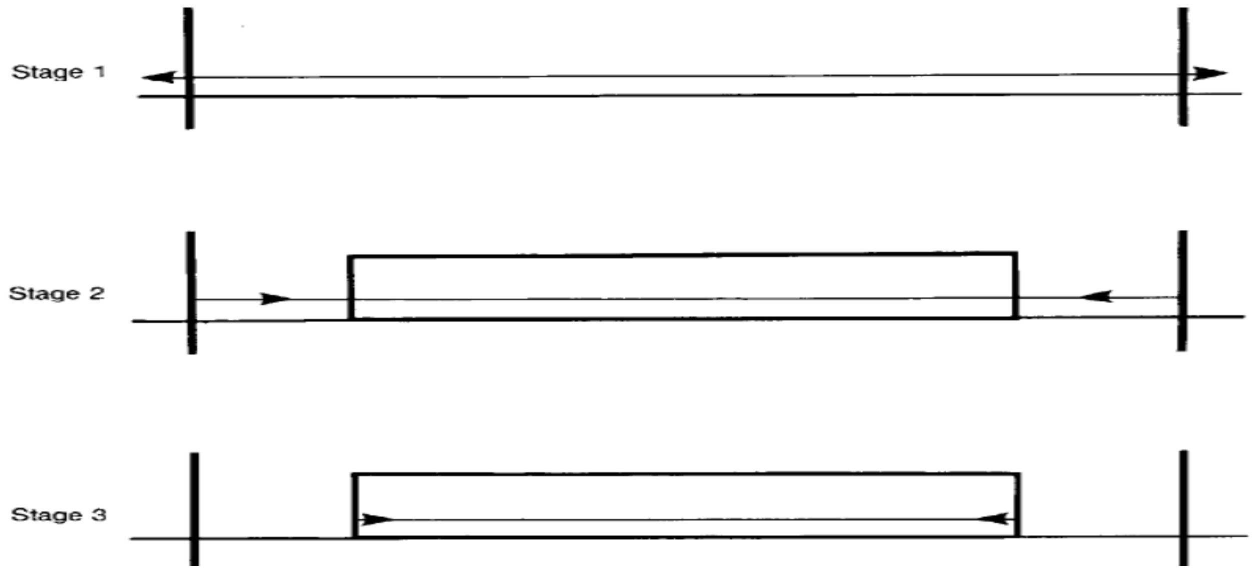


Fig. 2.3 stages of pre-tensioning.

In pre-tensioned members, the strand is directly bonded to the concrete cast around it.

In pre tensioned concrete components, the pre-stressing steel is tensioned before the concrete is placed, as illustrated in figure below. The pre-stressing tendons are typically un-sheathed so that the concrete bonds to the tendons. (A. Adamu, 2012. Pre-stressed Concrete Railway Bridges. Addis Ababa, Ethiopia)

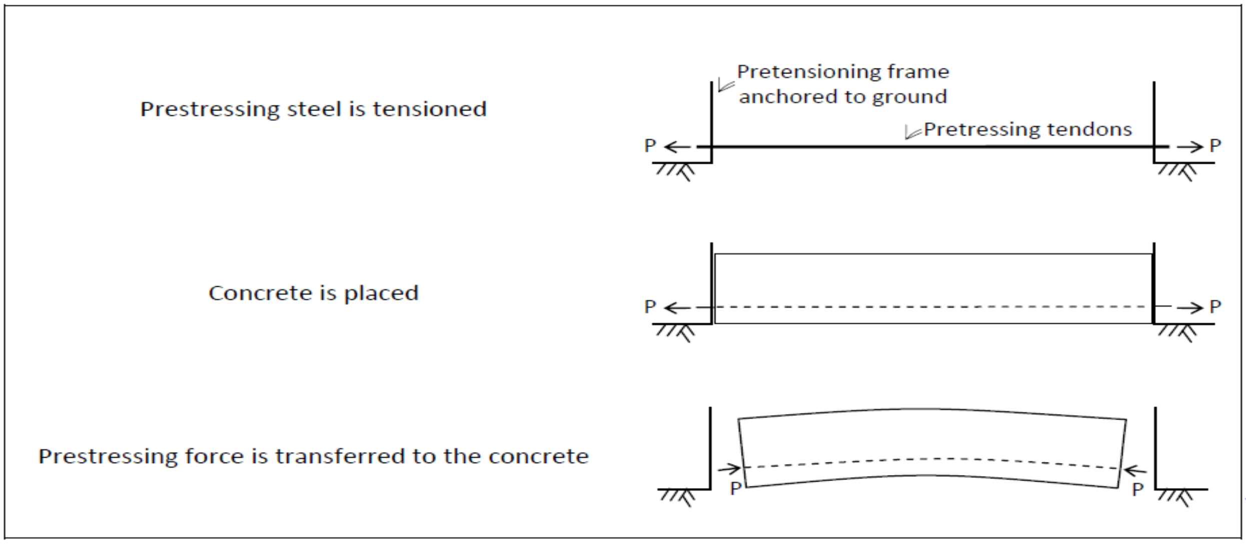


Fig. 2.4 Construction sequence for pre-tensioned concrete beam

2. Post-Tensioned In this method, the concrete has already set but has ducts cast into it. The strands or tendons are fed through the ducts (Stage 1) then tensioned (Stage 2) and then anchored to the concrete (Stage3) (A. Adamu, 2012. Pre-stressed Concrete Railway Bridges. Addis Ababa, Ethiopia)

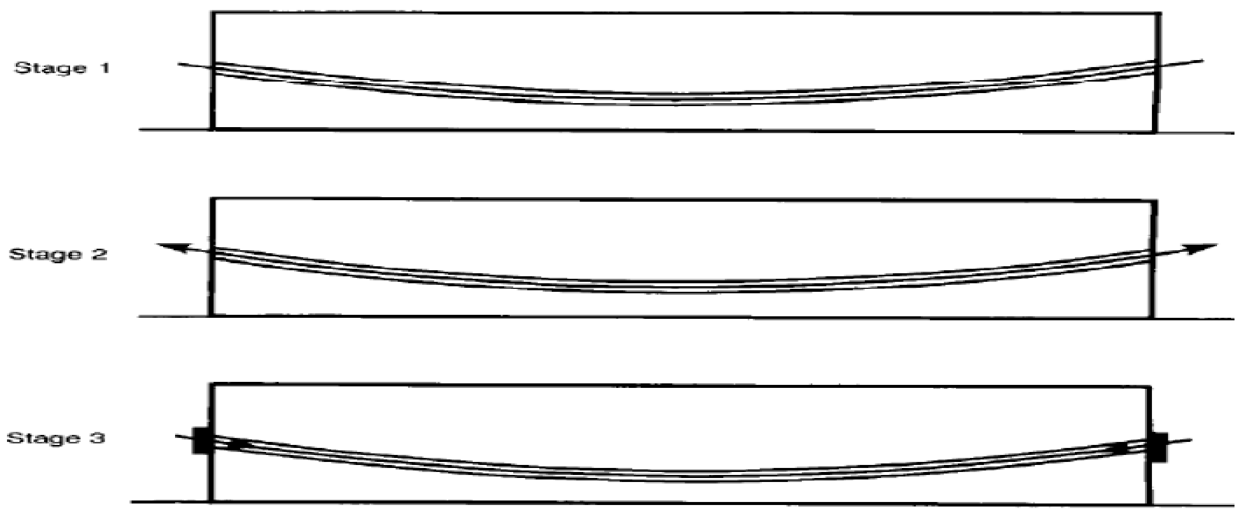


Fig.2.5 Stages of post tensioning

In post-tensioned concrete, the concrete is pre-stressing steel is tensioned after the concrete is placed, as illustrated in figure below. The pre-stressing tendons are sheathed or placed in ducts so that the concrete and tendons are unbounded. Ducts can be grouted after post-tensioning to protect the tendons from moisture and corrosion. (A. Adamu, 2012. Pre-stressed Concrete Railway Bridges. Addis Ababa, Ethiopia)

In post-tensioning the concrete unit are first cast by incorporating ducts or grooves to house the tendons. When the concrete attains sufficient strength, the high-tensile wires are tensioned by means of jack bearing on the end of the face of the member and anchored by wedge or nuts. The forces are transmitted to the concrete by means of end anchorage and, when the cable is curved, through the radial pressure between the cable and the duct. The space between the tendons and the duct is generally grouted after the tensioning operation.

Most of the commercially patented pre-stressing systems are based on the following principle of anchoring the tendons:

1. Wedge action producing a frictional grip on the wire.
2. Direct bearing from the rivet or bolt heads formed at the end of the wire.
3. Looping the wire around the concrete.

Advantage of Pre-Stressed Concrete

- The use of high strength concrete and steel in pre-stressed members results in lighter and slender members than is possible with RC members.
- In fully pre-stressed members the member is free from tensile stresses under working loads, thus whole of the section is effective.
- In pre-stressed members, dead loads may be counter-balanced by eccentric pre-stressing.
- Pre-stressed concrete member possess better resistance to shear forces due to effect of compressive stresses presence or eccentric cable profile.
- Use of high strength concrete and freedom from cracks, contribute to improve durability under aggressive environmental conditions.

- Long span structures are possible so that saving in weight is significant & thus it will be economic.
- Pre-stressed members are tested before use.
- Pre-stressed concrete structure deflects appreciably before ultimate failure, thus giving ample warning before collapse.

Disadvantages of Pre-Stressed Concrete

- The availability of experienced builders is short.
- Initial equipment cost is very high.
- Availability of experienced engineers is short.
- Pre-stressed sections are brittle.
- Pre-stressed concrete sections are less fire resistant.

Pre Stressing required Materials

The materials which are required for pre-stressing concrete girder are the followings:

i. Concrete

The main factors for concrete used in PSC are:

- Ordinary Portland cement-based concrete is used but strength usually greater than 50 N/mm2;
- A high early strength is required to enable quicker application of pre-stress;
- A larger elastic modulus is needed to reduce the shortening of the member;
- A mix that reduces creep of the concrete to minimize losses of pre-stress

ii.Steel

- The steel used for pre-stressing has nominal yield strength of between 1550 to 1800N/mm2.

The different forms the steel may take are:

Wires: individually drawn wires of 7 mm diameter;

Strands: a collection of wires (usually 7) wound together and thus having a diameter that is different to its area;

Tendon: A collection of strands encased in a duct – only used in post tensioning;

Bar: a specially formed bar of high strength steel of greater than 20 mm diameter.

iii. A Typical Tendon Anchorage

The anchorages to post-tensioned members must distribute a large load to the concrete, and must resist bursting forces as a result. A lot of ordinary reinforcement is often necessary.

iv. **Post-Tensioning Ducts**

Post-tensioning ducts for longitudinal post-tensioning tendons in precast spliced I-girders shall be made of rigid galvanized spiral ferrous metal to maintain standard girder concrete cover requirements. The radius of curvature of tendon ducts shall not be less than 20 feet except in anchorage areas where 12 feet may be permitted.

2.1. Precast Concrete

Pre-casting involves the casting of concrete away from the site of final position. It can also be produced near the site. The member of precast element produced either in a permanent plant or temporary arrangement and eventually erected at the final position. Pre-casting permits better control in mass production. It is also economical. Precast concrete is a construction product produced by casting concrete in a reusable form which is then cured in a controlled environment, transported to the construction site and lifted into place. The precast girder supports the cast in place part of the slab and standard precast girders may be grouped together or spread apart to accommodate heavy or light loads, respectively. This precast is post tensioned or pre tensioned is known as precast pre-stressed concrete.

2.2. Cast in place

Concrete cast in situ is poured into site-specific forms and cured on site. Cast in place concrete require more formwork and false work per unit of product but save the cost of transportation and erection, and it is better used for large and heavy members. The cast in place top slab ties the structure together and gives a uniform continuous surface. Therefore, the combination of precast pre-stressed concrete and concrete cast in situ is called composite precast pre-stressed - in situ concrete. As shown in following figure 4.6 below; by using composite construction, it is possible to save much form work and false work as compared to fully cast in place construction.

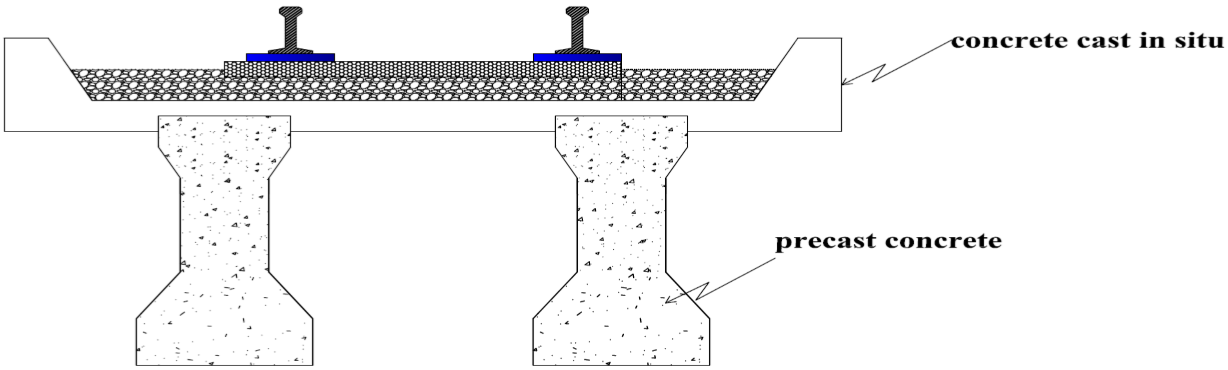


Figure 2.6:- composite precast pre-stressed Girder.

2.3. Railroad Bridges

General

- **Railway tracks** provides a permanent facility for movement of railway trains and consist of:
    - Rails
    - Sleepers
    - Ballast, and
    - Fastening

  (These track components are used in this study for dead load combination in addition to self-weight of bridge and other loads).
- **Rails** are continuous girders (high carbon steel girder) which carry the axle loads and serve the following functions:-

- Provide smooth and uniform surface,
- Bear vertical loads as well as lateral stresses due to braking & thermal stresses
- Transmit the load to the formation through sleepers and ballast

➢ Rail sections weigh **25 to 55 kg/m** and longer rails are preferred to reduce maintenance cost of joints and also reduce discomfort. Though it is proposed to have about 19 m for economic reasons, those available are about 12 m length.

➢ **Gauge of Tracks** – the clear horizontal distance between the inner faces (or running faces) of rails of same track measured near their tips and may be one of the following

- Wide gauge 2.130 m
- Broad gauge 1.676 m
- Standard gauge 1.430 m
- Metro gauge 1.000 m
- Narrow gauge 0.850 m to 0.782 m
- Light Gauge 0.610 m (this study will use the standard gauge = 1.430m)

➢ **Sleepers** support two rails at gauge distances and may be of wooden, metallic or concrete member and function to:

- Support rails firmly and to proper tilt
- Maintain proper gauge
- Transfer load safely to sub-grade or cross-structure (may be Bridge or Culvert) through ballast packing
- Act as elastic medium between the rail section and ballast
- Keep rails at proper level at turn outs and crossings

➢ **Ballast** – hard and strong granular material packed under sleepers or spread over the formation of a railway track which transfer load down over a large area of the earthwork, and can be

- broken stones,
- bricks, slag or

- sand, (this study will use the broken stones (granular)

Where, the size may be 2 to 5 cm is considered as best.

Large size do not provide good interlocking

Railroad Bridge Types

- Railroad bridges are nearly always simple-span structures. Listed below in groupings by span length are the more common types of bridges and materials used by the railroad industry for those span lengths.
- Short spans to 5 m Timber stringers

  Concrete slabs

  Rolled steel beams
- To 10 m Conventional and pre-stressed concrete box girders and beams

  Rolled steel beams
- To 15 m Pre-stressed concrete box girders and beams

  Rolled steel beams, deck and through girders
- Medium spans, 25 m to 40 m Pre-stressed concrete beams

  Deck and through plate girders
- Long spans Deck and through trusses (simple, cantilever, and arches)
- Suspension bridges are not used by freight railroads due to excessive deflection.

Bridge Deck is that portion of a railway bridge that supplies means of carrying the track rails

Railroad bridges typically are designed as either open deck or ballast deck structures. Some bridges, particularly in transit applications, use direct fixation of the rails to the supporting structure.

Open Deck: Open deck bridges have ties supported directly on load-carrying elements of the structure (such as stringers or girders). The dead loads for open deck structures can be significantly less than for ballast deck structures. Open decks, however, transfer more of the dynamic effects of live load into the bridge than ballast decks.

Open decks are less costly and are free draining, but their use over streets and highways requires additional measures such as canopies, plates or wooden flooring to protect highway traffic from falling objects, water or other materials during the movement of trains**.**

Ballast Deck Bridges have the track structure supported on ballast, which is carried by the structural elements of the bridge. Typically, the track structure (rails, tie plates, and ties) is similar to track constructed on grade. Ballast deck structures offer advantages in ride and maintenance requirements.

In ballast deck designs, an allowance for at least **150 mm** of additional ballast is prudent. Specific requirements for additional ballast capacity may be provided by the railroad. In addition, the required depth of ballast below the tie should be verified with the affected railroad. Typical values for this range from **200 mm to 300 mm** or more. The tie length used will have an effect on the distribution of live-load effects into the structure. Ballast decks are also typically waterproofed. The weight of waterproofing should be included in the dead load.

2.4. Bridge Design Loads

The following loads and forces shall be considered in the design of railway concrete structures supporting tracks (AREMA, 2010):

D = Dead Load F = Longitudinal Force due to Friction or Shear Resistance at Expansion Bearings

L = Live Load

I = Impact

CF = Centrifugal Force EQ = Earthquake (Seismic)

E = Earth Pressure SF = Stream Flow Pressure

B = Buoyancy ICE = Ice Pressure

W = Wind Load on Structure OF = Other Forces (Rib Shortening, Shrinkage, Temperature and/or Settlement of Supports)

WL = Wind Load on Live Load

LF = Longitudinal Force from Live Load

Each member of the structure shall be designed for that combination of such loads and forces that can occur simultaneously to produce the most critical design condition as specified in (AREMA 2010 Volume-2), Article 2.2.4.

Dead Load

(1) The dead load shall consist of the estimated weight of the structural member, plus that of the track, ballast, fill, and other portions of the structure supported thereby.

(2) The unit weight of materials comprising the dead load, except in special cases involving unusual conditions or materials, shall be assumed as follows (AREMA, 2010):-

• Track rails, inside guardrails and fastenings – 200 lb per linear foot of track. (3kN/m)

• Ballast, including track ties – 120 lb per cubic foot. (1900 kg/m3)

• Reinforced concrete – 150 lb per cubic foot. (2400 kg/m3)

• Earth filling materials – 120 lb per cubic foot. (1900 kg/m3)

• Waterproofing and protective covering – estimated weight. (AREMA)

Live Load

(1) The recommended live load for each track of main line structure is Cooper E 80 (EM 360) loading with axle loads and axle spacing as shown in the following Figure 4.7. On branch lines and in other locations where the loading is limited to the use of light equipment, or cars only, the live load may be reduced, as directed by the engineer. For structures wherein the material in the

primary load-carrying members is not concrete, the E loading used for the concrete design shall be that used for the primary members.

(2) The axle loads on structures may be assumed as uniformly distributed longitudinally over a length of 3 feet (900 mm), plus the depth of ballast under the tie, plus twice the effective depth of slab, limited, however, by the axle spacing.

(3) Live load from single track acting on the top surface of a structure with ballasted deck or under fills shall be assumed to have uniform lateral distribution over a width equal to the length of track tie plus the depth of ballast and fill below the bottom of tie, unless limited by the extent of the structure.

(4) The lateral distribution of live load from multiple tracks shall be as specified for single tracks and further limited so as not to exceed the distance between centers of adjacent tracks.

The Cooper E80 load is currently the most common design live load. The 80 in E80 refers to the 80 kip **(360 KN)** weight of the locomotive drive axles. An E60 load has the same axle locations, but all loads are factored by **60/80**. Some railroads are designing new structures to carry E90 or E100 loads.

The designated steel bridge design live load also includes an “Alternate E80” load, consisting of four 100-kip (4500 KN) axles. This is also shown in the Figure below. This load controls over the regular Cooper load on shorter spans.

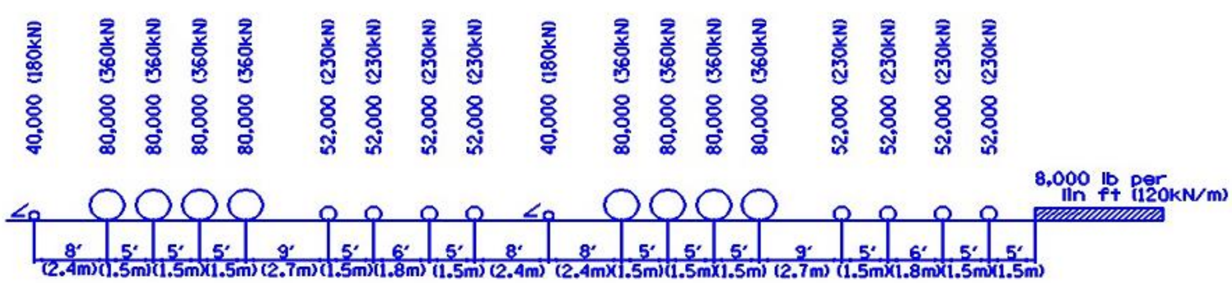
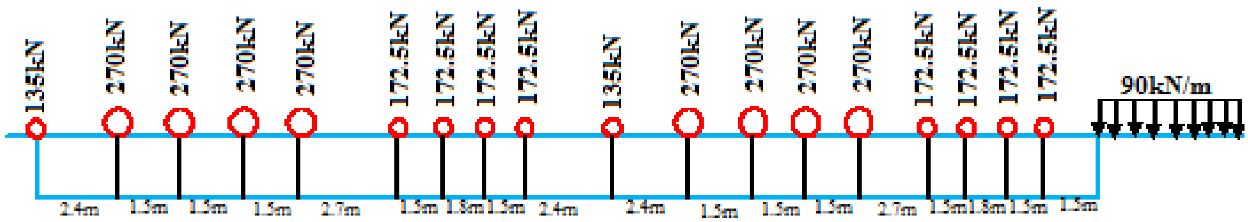


FIGURE: 2.7. Cooper E8O live load



Cooper E 60 Live Load

(This project use cooper E80 & E60 live load for moment and shear force read from AREMA table 15.1.1) in the following table 2.1

(5) In calculating the maximum live loads on a structural member due to simultaneous loading on two or more tracks, the following proportions of the specified live load shall be used:

• For two tracks – full live load,

The following table is used to directly read (to determine) the Maximum bending moment and Shear force due to live load of the train on the structure having different span lengths.

Table 2.1 Maximum moments, and shears of LL for one rail (one - half track load)

span length (Ft)	maximum moment (Ft-Kips)		maximum moment Quarter point (Ft-Kips)		maximum shears (Kips)						Maximum pier reaction (Kips) (2)	
					At End		At Quarter Point		At center			
	E-80	Alt.	E-80	Alt.	E-80	Alt.	E-80	Alt.	E-80	Alt.	E-80	Alt.
5	50.00	62.50	37.50	46.88	40.00	50.00			20.00	25.00	40.00	50.00
6	60.00	75.00	45.00	56.25	46.67	58.33	30.00	37.50	20.00	25.00	53.33	58.33
7	70.00	87.50	55.00	68.75	51.43	64.29	31.43	39.29	20.00	25.00	62.86	71.43
8	80.00	100.00	70.00	87.50	55.00	68.75	35.00	43.75	20.00	25.00	70.00	81.25
9	93.89	117.36	85.00	106.25	57.58	72.22	37.78	47.23	20.00	25.00	75.76	88.89
10	112.50	140.63	100.00	125.00	60.00	75.00	40.00	50.00	20.00	25.00	80.00	95.00
11	131.36	164.20	115.00	143.75	65.45	77.27	41.82	52.28	21.82	27.28	87.28	100.00
12	160.00	188.02	130.00	162.50	70.00	83.33	43.33	54.17	23.33	29.17	93.33	108.33
13	190.00	212.83	145.00	181.25	73.84	88.46	44.61	55.76	24.61	30.76	98.46	115.39
14	220.00	250.30	165.00	200.00	77.14	92.86	47.14	57.14	25.71	32.14	104.29	121.43
16	280.00	325.27	210.00	250.00	85.00	100.00	52.50	62.50	27.50	34.38	113.74	131.25
18	340.00	400.24	255.00	318.79	93.33	111.11	56.67	68.05	28.89	36.11	121.33	138.89
20	412.50	475.00	300.00	362.50	100.00	120.00	60.00	72.50	28.70	37.50	131.10	145.00
24	570.42	668.75	420.00	500.00	110.83	133.33	70.00	83.33	31.75	41.67	147.92	154.17
28	730.98	866.07	555.00	650.00	120.86	142.86	77.14	92.86	34.29	46.43	164.58	
32	910.85	1064.06	692.50	800.00	131.44	150.00	83.12	100.00	37.50	50.00	181.94	
36	1097.30	1262.50	851.50	950.00	141.12	155.56	88.90	105.56	41.10	55.56	199.06	
40	1311.30	1461.25	1010.50	1100.00	150.80	160.00	93.55	110.00	44.00	60.00	215.90	
45	1601.20	1710.00	1233.60	1287.48	163.38	164.44	100.27	114.45	45.90	64.45	237.25	
50	1901.80	1959.00	1473.00	1481.05	174.40		106.94	118.42	49.73	68.00	257.52	
55	2233.10		1732.30		185.31		113.58	120.91	52.74	70.91	280.67	
60	2597.80		2010.00		196.00		120.21	123.33	55.69	73.33	306.42	
70	3415.00		2608.20		221.04		131.89		61.45	77.14	354.08	
80	4318.90		3298.00		248.40		143.41		67.41	80.00	397.70	
90	5339.10		4158.00		274.46		157.47		73.48	82.22	437.15	
100	6446.30		5060.50		300.00		173.12		78.72	84.00	474.24	
120	9225.40		7098.00		347.35		202.19		88.92		544.14	
140	12406.00		9400.00		392.59		230.23		101.64		614.91	

Span Length Ft	Maximum Moment Ft–Kips		Maximum Moment Quarter Point Ft–Kips		Maximum Shears Kips						Maximum Pier Reaction Kips (2)	
					At End		At Quarter Point		At Center			
	E-80	Alt.	E-80	Alt.	E-80	Alt.	E-80	Alt.	E-80	Alt.	E-80	Alt.
160	15908.00 (1)		11932.00		436.51		265.51		115.20		687.50	
180	19672.00 (1)		14820.00		479.57		281.96		128.12		762.22	
200	23712.00 (1)		17990.00		522.01		306.81		140.80		838.00	
250	35118.00 (1)		27154.00		626.41		367.30		170.05		1030.40	
300	48800.00 (1)		38246.00		729.34		426.37		197.93		1225.30	
350	65050.00 (1)		51114.00		831.43		484.64		225.51		1421.70	
400	83800.00 (1)		65588.00		933.00		542.40		252.44		1619.00	

Note (1) - Values for Cooper E-80 Live Load. Moment values taken at center span.
Note (2) - Maximum pier reactions are for equal span lengths.

Impact: is the dynamic amplification of the live-load effects on the bridge caused by the movement of the train across the span. The design impact values are based on an assumed train speed of 60mph (96.56 km/h). It should be noted that the steel design procedure allows reduction of the calculated impact for ballast deck structures. Different values for impact from steam and diesel locomotives are used. The steam impact values are significantly higher than diesel impact over most span lengths.

The impact shall be equal to the following percentages of the live load:

$$\begin{cases} L \leq 4m, \; I = 60 \\ 4m < L \leq 39m, \; I = \frac{125}{\sqrt{L}} \\ L > 39m, \; I = 20 \end{cases}$$

Where L is the span length in meters (for this study different span length will be checked to determine the cost effective span length).

- For continuous structures, the impact value calculated for the shortest span shall be used throughout.
- Impact may be omitted in the design for massive substructure elements which are not rigidly connected to the superstructure.
- For steam locomotives with hammer blow, the impact shall be increased by 20%

Centrifugal Force: is the force a train moving along a curve exerts on a constraining object (track and supporting structure) which acts away from the center of rotation. Although the centrifugal action is applied as a horizontal force, it can produce overturning moment due to its point of application above the track. Both the horizontal force and resulting moment must be considered in design or evaluation of a structure. For all bridge types, the bearings and substructure must be able to resist the centrifugal horizontal force.

The train speed required for the force calculation should be obtained from the railroad. The centrifugal force corresponding to each axle load may be applied horizontally at a point 2.45m above the top of rail measured along a line perpendicular to the line joining the tops of the rails and equidistant from them.

This force shall be the percentage of the live load computed from the formulas below.

$$C = 0.000452S^2 D$$

$$E = 0.0068S^2 D-75$$

$$S = \sqrt{\frac{E + 75}{0.0068D}}$$

Where:

C= Centrifugal force in percentage of the live load

D= Degree of curve (based on 30m chord)

E= Actual super elevation in mm

S= Permissible speed in km/hr

Seismic Loads: In regions where earthquakes may be anticipated, structures may be designed to resist earthquake motions by considering the relationship of the site to active faults, the seismic response of the soils at the site, and the dynamic response characteristics of the total structure.

Railroad bridges have historically performed well in seismic events, due to the following factors:

- The track structure serves as an effective restraint (and damping agent) against bridge movement.
- Railroad bridges are typically simple in their design and construction.
- Trains operate in a controlled environment, which makes types of damage permissible for railroad bridges that might not be acceptable for structures in general use by the public.

Seismic force primarily affects bridge substructure components. The earth quake forces can be described as a function of the acceleration coefficient, Soil type, the fundamental period. The effect of earthquake in Ethiopia is grouped in to four different zones. Bridge in seismic zone 1-3 need not be analyzed for seismic loads, regardless of their importance and geometry. Seismic analysis is not required for single-span Bridge, regardless of the seismic zone .Since the study area is Addis Ababa (zone-2), and no need of design of seismic load in this zone.

2.5. Load Combinations:

A variety of loads can be applied to a structure at the same time. A bridge may experience dead load, live load, impact, centrifugal force, wind, and stream flow simultaneously.

The following groups represent various combinations of loads and forces to which a structure may be subjected. Each component of the structure, or the foundation on which it rests, shall be proportioned for the group of loads that produce the most critical design condition.

Load Factor Design.

(1) The group loading combinations for LOAD FACTOR DESIGN are as shown in the following table 2.2

Table 2.2 Group Loading Combinations – Load Factor Design

Group	Item
I	1.4 (D+5/3(L+I) + CF + E+B+ SF)
IA	1.8(D + L + I + CF + E + B + SF)
II	1.4(D + E + B + SF + W)
III	1.4(D + L + I + CF + E + B + SF + 0.5W+ WL + LF+ F)
IV	1.4(D + L + I + CF + E + B + SF + OF)
V	Group II + 1.4 (OF)
VI	Group III + 1.4 (OF)
VII	1.0 (D+ E + B+ EQ)
VIII	1.4 (D+ L+ I + E + B + SF+ ICE)
IX	1.2 (D + E + B + SF + W + ICE)

Where;

D = Dead Load F = Longitudinal Force due to Friction or

Shear Resistance at Expansion

Bearings

L = Live Load

I = Impact

CF = Centrifugal Force, EQ = Earthquake (Seismic)

E = Earth Pressure, SF = Stream Flow Pressure

B = Buoyancy, ICE = Ice Pressure

W = Wind Load on Structure, OF = Other Forces (Rib Shortening, Shrinkage, Temperature and/or Settlement of Supports)

WL = Wind Load on Live Load

LF = Longitudinal Force from Live Load

For this project Load Factor Design load combination Group 1 = 1.4(D+5/3(L+I)) is applied. B/c the combination has almost includes all types of loads on the girder.

2.6. Design Criteria: Strengths, Deflections and Shears

Material Properties

To satisfy the design objective of a structural member, the design strength for ULS of both concrete and steel are required. The instantaneous and time dependent properties of concrete and steel at typical in-service stress level are also required.

A) Concrete

High strength concrete are used, concrete grade larger than C-30 class I works high compressive strength at a reasonably early age, & comparatively higher tensile strength as compared with ordinary RC member, low shrinkage, minimum creep characteristics and high young‟s modulus are necessary.

Concrete grade larger than C-40, for pre tensioned,

C-30, for post tensioned members.

The stress permitted in concrete at the stage of transfer and service loads are defined in terms of the corresponding compressive strength of the concrete at each stage.

Allowable stresses in concrete.

At transfer

$f_{ct} \leq 0.6\ f_{ck(t)}$ in compression due to bending.

$f_{tt} \leq 0.21\ f_{ck(t)}^{2/3}$ in tension due to bending.

Under working loads

$f_{cw} \leq 0.50\ f_{ck}$ in compression due to bending.

$f_{tw} \leq 0.00$ in tension for fully pre-stressing.

b) Pre-Stressing Steel

Three types of high strength steel termed as tendons are used with $f_{pu} \geq 1000$ MPa. For design purpose the following idea stress – strain curve is adopted.

In design the stress in pre stressing steel at ULS limited to $0.9f_{pk}/\Upsilon_s$ and strain $_u \leq 0.01$.

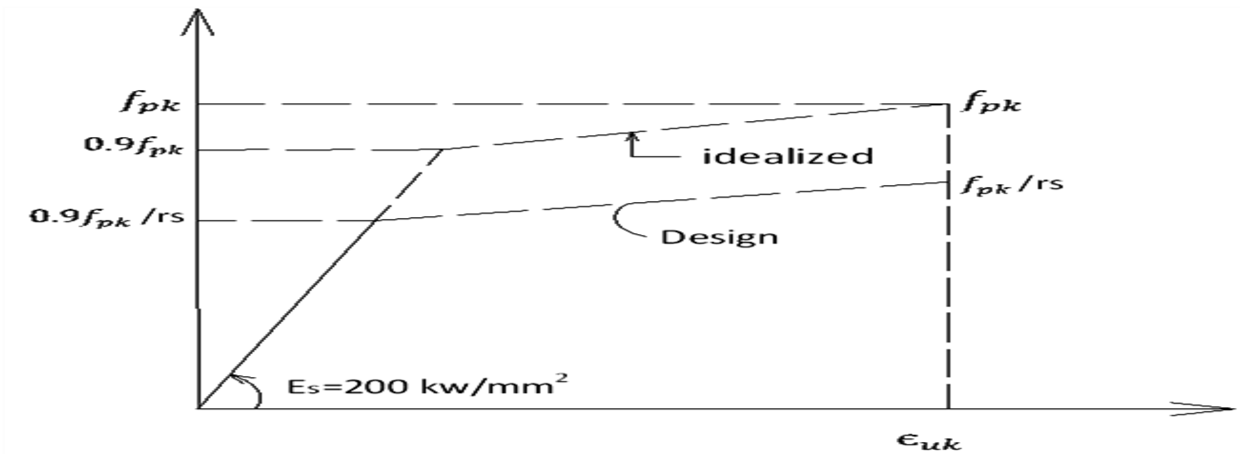


Figure 2.8 Stress-strain curves for typical high strength steel

Immediately after pre stress transfer

fpi ≤ 0.75fpk

Stress in tendons after all losses (under working)

fpe ≤ η*(0.75fpk) = 0.6 fpk

Where, η- is loss factor which is 0.78 or 0.82 for pre tension and post tension respectively.

Pre-Stress Losses

Lump Sum Estimate of Pre Stress Losses

For average steel and concrete properties, cured under average air condition, the following values may be taken as representative of average losses.

Loss due to	% of loss	
	Pre- tensioning	Post - tensioning
Elastic shortening and bending concrete	4	1
Creep of concrete	6	5
Shrinkage of concrete	7	6
Steel relaxation	5	6
Total loss	22	18

Table 2.3 pre – stress losses.

These values are based on the assumption that over tensioning has been applied to overcome friction and anchorage losses. An average of 22% loss in case of pretension and 18% in post-tension may be assumed with condition of over tensioning has been applied to overcome friction and anchorage losses.

Deflection

Deflections can take place: upwards due to pre stressing (-) & down wards due to transverse loads (+). The deflection can be calculated using the ordinary deflection formulas because there are no cracks Deflections due to loads: ordinary formula can be applied depending on:-

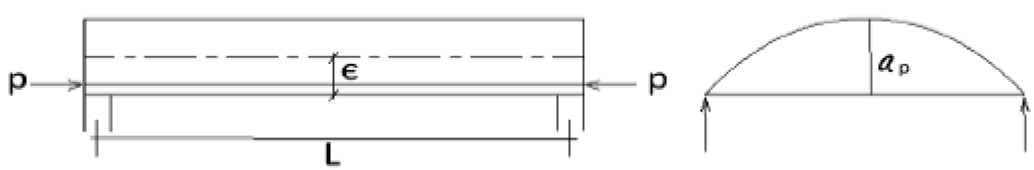
- The intensity of loading
- Restraint condition and span

Deflection due to Self-Weight or uniformly distributed Superimposed Load

$$\Delta = \frac{5 \times W \times L^4}{384 \times E_c \times I}$$

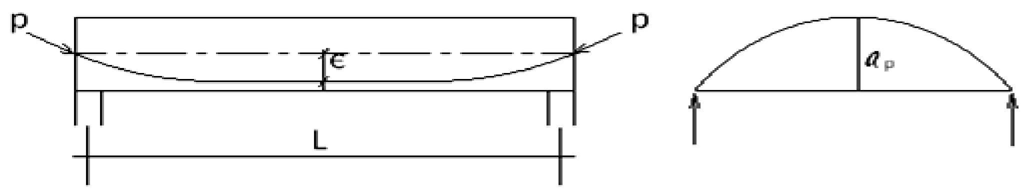
Deflection due to Pre Stressing Force

i) Straight Tendons



$$\Delta_p = \frac{-P \times e \times L^2}{8EL}$$

ii) Parabolic Tendons (Central Anchors)



$$\Delta = \frac{-5 \times P \times e \times L^2}{48 \times E_c \times I}$$

Shear Two major modes of shear cracks may be observed namely web shear and flexure shear cracks.

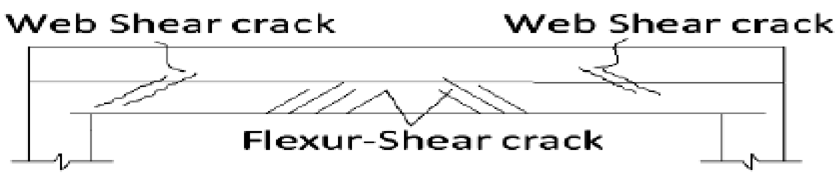


Fig.2.10 Web and flexural shear crack.

Web Shear Crack: - it is governed by limiting value of the principal tensile stress as developed in concrete. It is more likely to occur in highly pre stressed members with thin webs.

Flexure Shear Cracks: - primarily initiated by flexural cracks and are developed when the combined shear and flexural tensile stresses produce a principal tensile stress exceeding the design tensile strength of concrete. When the shear force exceeds the limiting value against diagonal compression the section must be changed.

When Vc > Vd, only minimum shear reinforcement shall be provided which may be expressed as:-

$$S_{max} = \frac{a_v}{\rho_w \times b} = \frac{a_v \times f_{yk}}{0.4 \times b}$$

The shear force sustained by stirrups may be computed as:-

$$V_s = \frac{a_v \times d \times f_{yd}}{S}$$

CHAPTER - THREE

3.0. Analysis and Design of Pre-Stressed Concrete Bridge.

Conventions:- Concrete is homogeneous elastic material and within the range of working stresses both materials concrete & steel behave elastically

Plane sections before bending remain plane.

Sign convention: - in the next computation

- (+) is for compression
- (-) is for tensile stress

An average of 22% loss in case of pretension and 18% in post-tension may be assumed with condition of over tensioning has been applied to overcome friction and anchorage losses.

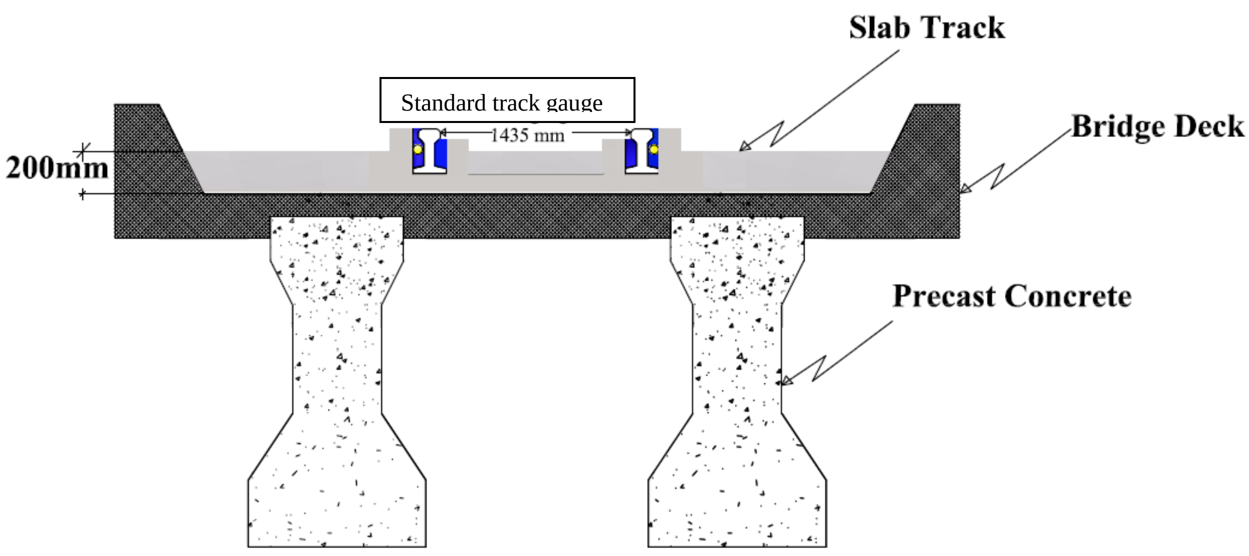


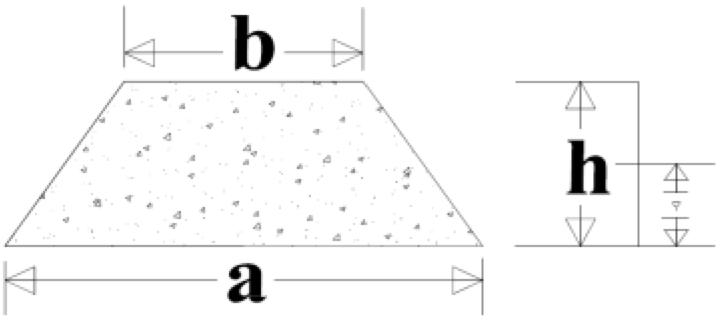
Fig. 3.1 The cross – section of pre-stressed girder.

For Slab track; slab is continuously casted over the girder and the rail is placed on it. But for ballasted track the sleeper is spread over the ballast with in constant space and the rail placed over it.

The minimum depth of slab track is 200mm

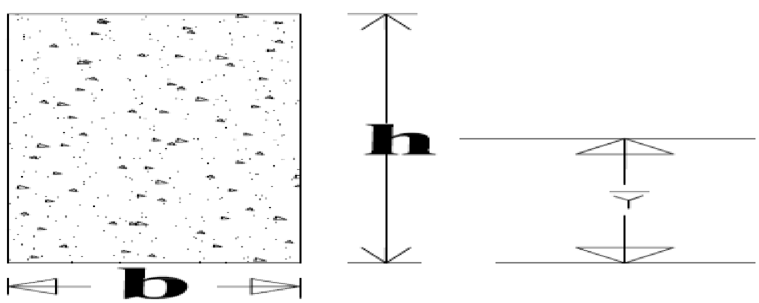
3.1. Section Properties of Pre-Stressed Precast Girder.

Centroid of trapezium (form top of trapezium):- $Y = \frac{h(2a+b)}{3(a+b)}$



Second moment of inertia at center of trapezium is given as:-

$$\bar{I} = \frac{h^3(a^2+4ab+b^2)}{36(a+b)}$$



Centroid of rectangle = $\bar{y} = \frac{h}{2}$

Second moment of inertia at center of rectangle = $\bar{I} = \frac{b \times h^3}{12}$

Section modulus, Z is (Top and Bottom respectively)

$$Z_t = \frac{\bar{I}}{h-\bar{y}}$$

$$Z_b = \frac{\bar{I}}{\bar{y}}$$

Where, $\bar{y}$ is measured from base or bottom of rectangle.

3.2. Section Properties of composite Girder

Centroid of composite structure is

$$\bar{y} = \frac{\sum A_i \times \bar{y}_i}{\sum A}$$

Where, A_i - Area of components of composite structure

$\bar{y}_i$ - Centroid of components of composite structure

A - Area of composite structure

The moment of inertia about the desired axis by the transfer of axis theorem is $\bar{I} + A \times d^2$.

The entire section the desired moment of inertia becomes $I = \Sigma\, \bar{I} + \Sigma A \times d^2$.

The moment of inertia for the composite area about the x- and y- axes is given as:-

$$I_x = \sum \bar{I}_x + \sum A \times {d_x}^2 ,$$

$$I_y = \sum \bar{I}_y + \sum A \times {d_y}^2$$

3.3. Analysis Procedure for Different Stages of Loading

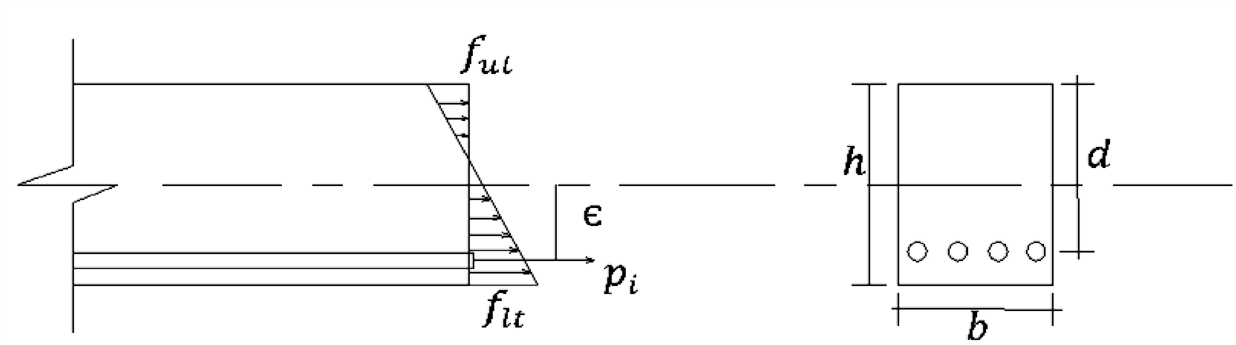
1. For precast

Bending stress analysis of a PC member has two cases and calculation may be made:

a)At transfer

- Self-weight
- Pre-stress force

Stresses Immediately after Pre Stressing



fut = Pi/Apc - Pi × e/Ztp + M(gp) /Ztp ≥ - ftt

flt = Pi/Apc + Pi × e/Zbp - M(gp) /Zbp ≤ fct

Where;

Pi = pre stressing force immediately after pre stressing

Apc = b×h, area of concrete,

Ztp = section modulus top,

Zbp = section modules bottom

Mgp = bending moment caused by the own weight of precast.

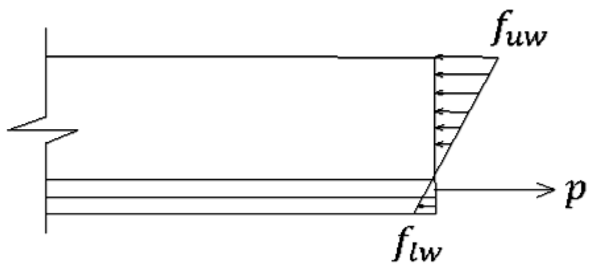
fct = permissible compressive stresses in concrete at the moment of pre stressing.

ftt = permissible tensile stresses in concrete at the moment of pre stressing.

b)**Under Working (at Service)**

- Self-weight
- Cast-in situ deck
- Pre-stressing force

Stresses after all Losses of Pre Stress



flw = Pe/Apc +Pe × e/Zbp – M(gp+gin-situ) /Zbp ≥ -ftw

fuw = Pe/Apc - Pe × e/Ztp + M(gp+gin-situ) /Ztp ≤ fcw

Where;

Pe - is the effective pre stressing force after all losses.

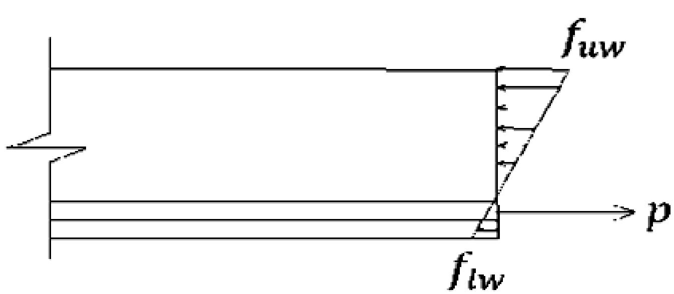
M (gp+gin) = Bending moment caused by maximum total load of precast and cast-in situ.

2. For Precast-Cast-in Situ Composite Structure

Under Working (at Service)

- Self-weight (precast plus cast-in situ deck)
- Superimposed dead load
- Live load & Pre-stressing force

Stresses after all Losses of Pre Stress and the full Working Load acts on the Member



flw = Pe/Apc +Pe × e/Zbp – M(g1+g2 +q) /Zbc ≥ -ftw

But -ftw is zero for fully pre-stressed

fuw = Pe/Apc - Pe × e/Ztp + M(g1+g2 + q) /Ztc ≤ fcw

Where, Pe - is the effective pre stressing force after all losses.

M (g1+ g2+q) = bending moment caused by maximum total load (self-weight, superimposed load and live load).

Ultimate Limit State of a Cross Section

Consider the PC member when stressed nearly to failure.

NP = AP × fpyd,

Where,

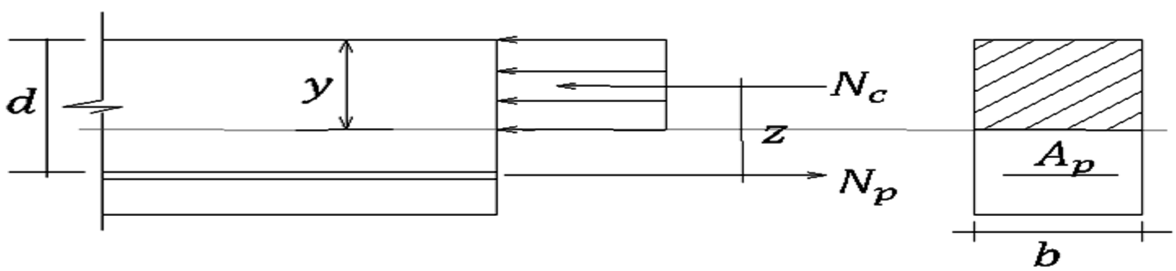
fpyd - is the design strength of tendon

fpyd = 0.9×fP

fP = fpk/ϒs,

Where, ϒs is partial safety factor

AP = area of tendons cross section.



ΣFx = 0 Np = Nc = y×b×fcd (summation of Force at horizontal = 0)

$y = \frac{Np}{b * fcd}$, Z =d-y/2

ΣM = 0 (summation of moment = 0)

Mu = Nc × Z = Np × Z

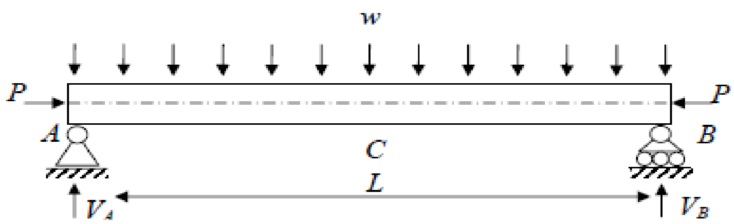
Design moment must be less than ultimate moment ($M_d \leq Mu$)

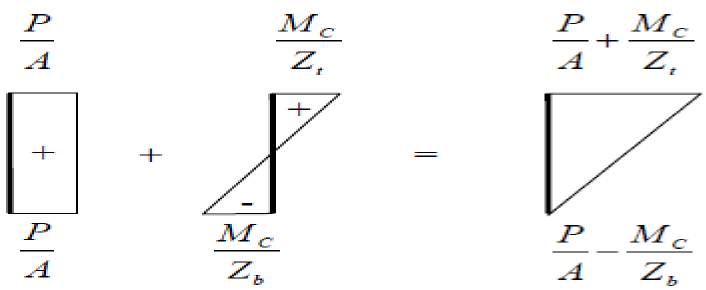
Consider a simply-supported beam subjected to a uniformly distributed load of w kN/m

The maximum mid-span moment is given as:-

$$M = \frac{WL^2}{8}$$

Situation 1:- Consider the beam with centroidal axial pre-stress (eccentricity equals zero)



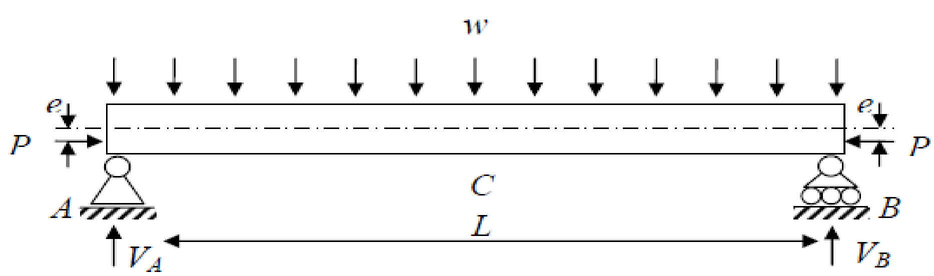


Failure to occur, the moment caused by the load must bring a tensile stress greater than P/A . Therefore, just prior to failure, we have:

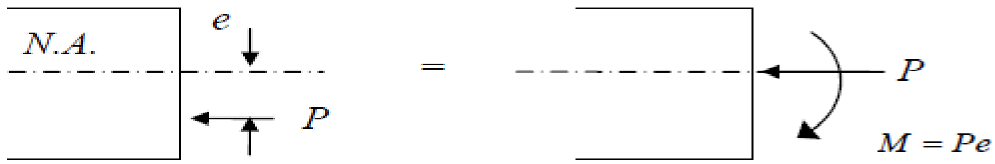
$$\frac{M_c}{Z_b} = \frac{P}{A} = \frac{WL^2}{8Z_b}$$

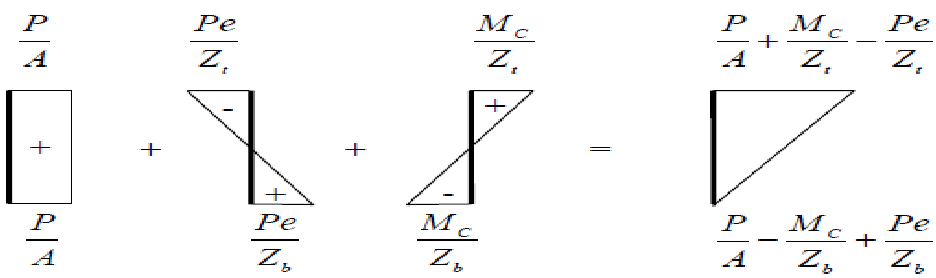
$$W_I = \frac{8P \times Z_b}{A \times L^2}$$

Situation 2:- Consider the beam with centroidal axial pre-stress (eccentricity not equals to zero pre-stress force at an eccentricity.



We have the following stress:-





Therefore, the failures we have are:

$$\frac{Mc}{Zb} = \left(\frac{P}{A} + \frac{Pe}{Zb}\right)$$

But $M_c = \frac{WII*L^2}{8}$

$$W_{II} = \frac{8*Zb}{L^2}\left(\frac{P}{A} + \frac{Pe}{Zb}\right)$$

If we take e =d/6, then:

$$W_{II} = \frac{8*Zb}{L^2}\left(\frac{P}{A} + \frac{Pd/6}{(bd^2)/6}\right) = \frac{16*Zb}{L^2} * \frac{P}{A} = 2 * W_I$$

So the introduction of a small eccentricity has doubled the allowable service load.

3.4. Design Consideration for Shear

i. Web Shear Crack: - it is governed by limiting value of the principal tensile stress as developed in concrete. It is more likely to occur in highly pre stressed members with thin webs.

$$V_{cw} = 0.67 * b_w * h * \sqrt{fctd^2 + fcp * fctd}$$

Where;

fcp is compressive pre stress at the centroid of a section

bw is the breath of web,

h is gross depth.

ii.Flexure Shear Cracks: - primarily initiated by flexural cracks and are developed when the combined shear and flexural tensile stresses produce a principal tensile stress exceeding the design tensile strength of concrete.

The section capacity in this case on the basis of experiments is given by

$V_c = ß_1 * 0.05 * b * d * f_{ctd}$

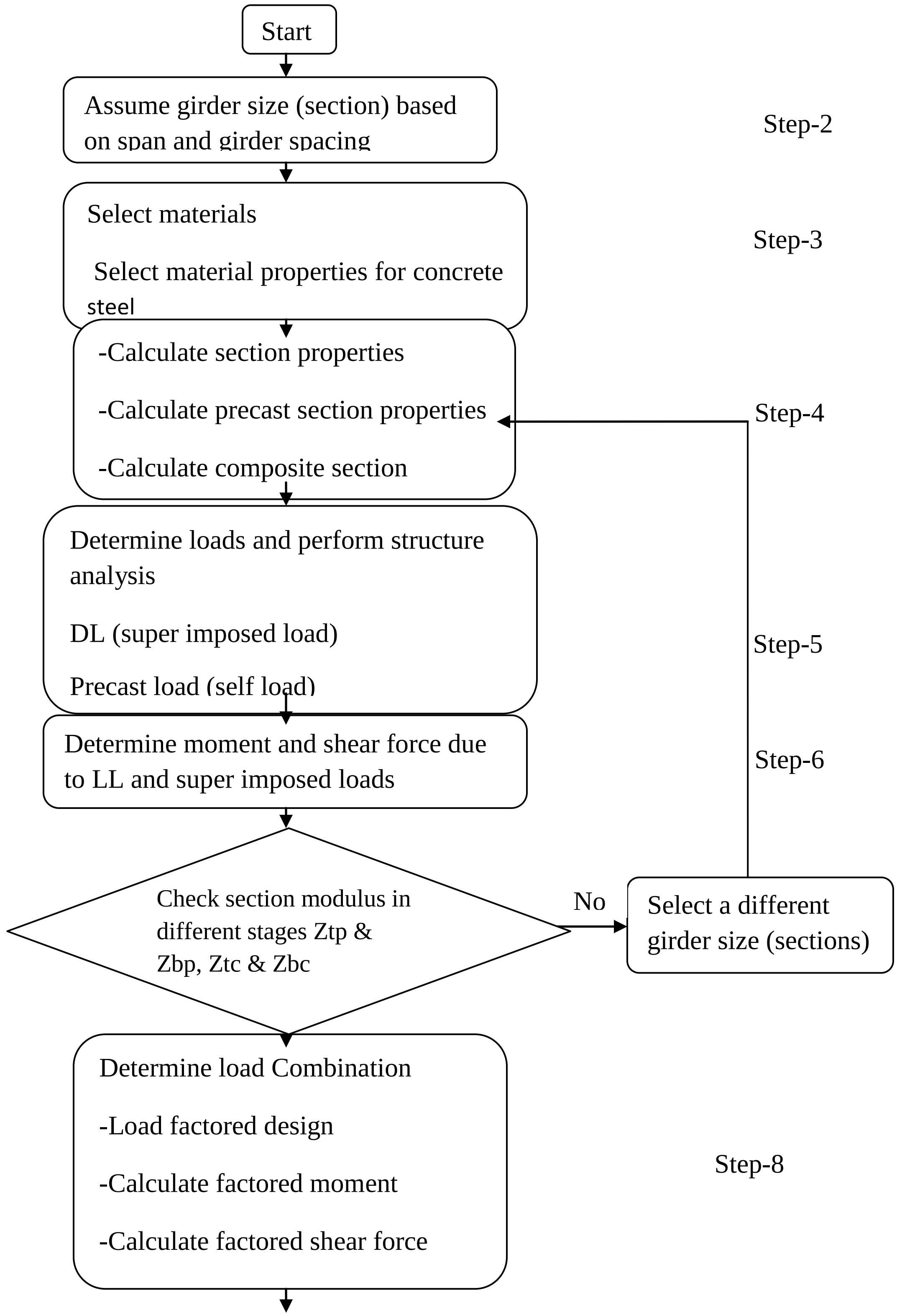
Where, $ß_1 = 1 + \frac{Mo}{Md} \leq 2.0$

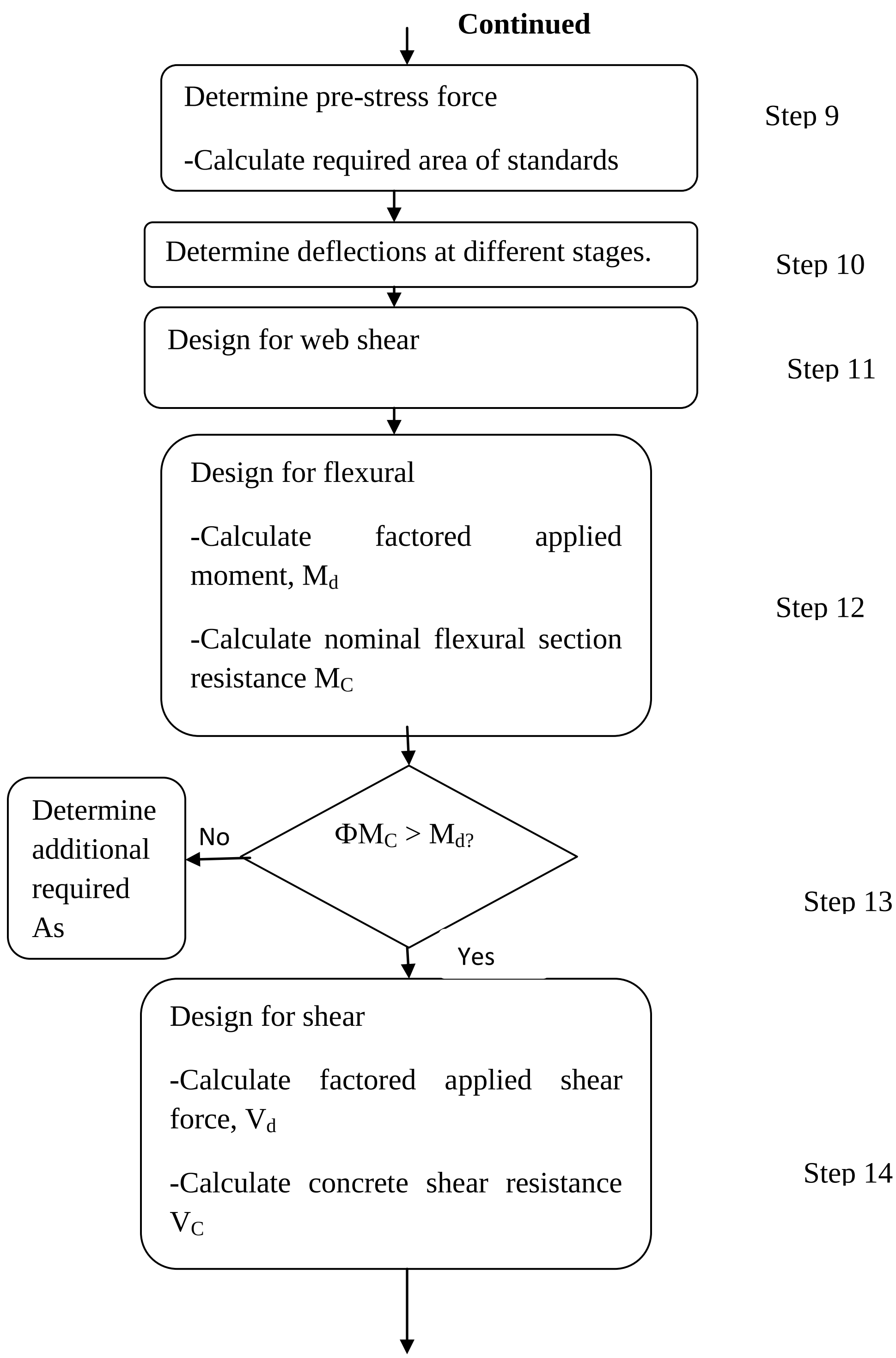
Md is the bending moment at the cross section where shear capacity is calculated. Mo = P(Z/Ac + e) , Z is section modulus where flexure shear likely to occur , P is pre-stressing force at the cross section however, when the shear force exceed the limiting value against diagonal compression the section must be changed. When Vc > Vd, only minimum shear reinforcement shall be provided.

CHAPTER FOUR

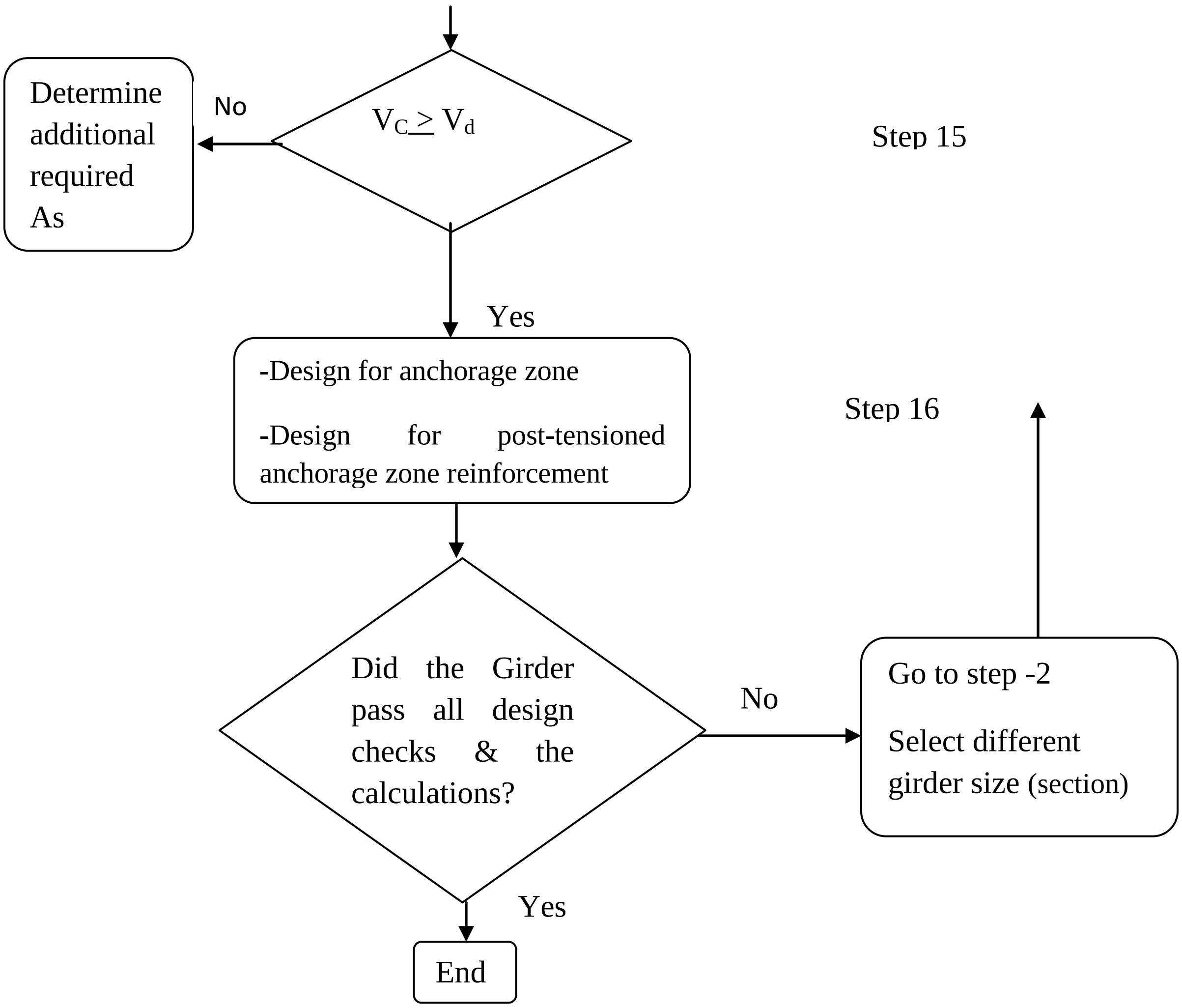
4.0. DESIGN FLOW CHART

The following flow chart shows the typical steps for designing precast, pre-stressed concrete girders. The example in the next section closely follows this flow chart.


Start
Assume girder size (section) based on span and girder spacing
Step-2
Select materials
Select material properties for concrete steel
Step-3
-Calculate section properties
-Calculate precast section properties
-Calculate composite section
Step-4
Determine loads and perform structure analysis
DL (super imposed load)
Precast load (self load)
Step-5
Determine moment and shear force due to LL and super imposed loads
Step-6
Check section modulus in different stages Ztp & Zbp, Ztc & Zbc
No
Select a different girder size (sections)
Determine load Combination
-Load factored design
-Calculate factored moment
-Calculate factored shear force
Step-8


Continued
Determine pre-stress force
-Calculate required area of standards
Step 9
Determine deflections at different stages.
Step 10
Design for web shear
Step 11
Design for flexural
-Calculate factored applied moment, M_d
-Calculate nominal flexural section resistance M_C
Step 12
$\Phi M_C > M_d$?
No
Determine additional required As
Yes
Step 13
Design for shear
-Calculate factored applied shear force, V_d
-Calculate concrete shear resistance V_C
Step 14

Continued



Where; Φ is section capacity moment deduction factor.

4.1. DESIGN EXAMPLE

50m Span Bridge

A simple span precast post tensioned pre-stressed concrete girder bridge is to be designed for a railway bridge with cast in-situ deck slab, cross section of which shown in Figure, below. The cast in place concrete deck slab is of 300 mm thick, while the ballast below the sleepers is 250 mm in thickness. Additional ballast above the bottom of the sleepers may be assumed of 150 mm. The bridge is to serve for the Cooper E-80 Rail load configuration and the following design data may be used.

<u>Materials</u>

- Concrete C-50
- Mild steel S-500
- Pre-stressing Tendons f_{pk} = 1800 MPa
- Unit weight of plain concrete 24 kN/m^3
- Unit weight of RC 25 kN/m^3
- Unit weight of Steel 78.5 kN/m^3
- Unit weight of sand, gravel and ballast 19 kN/m^3

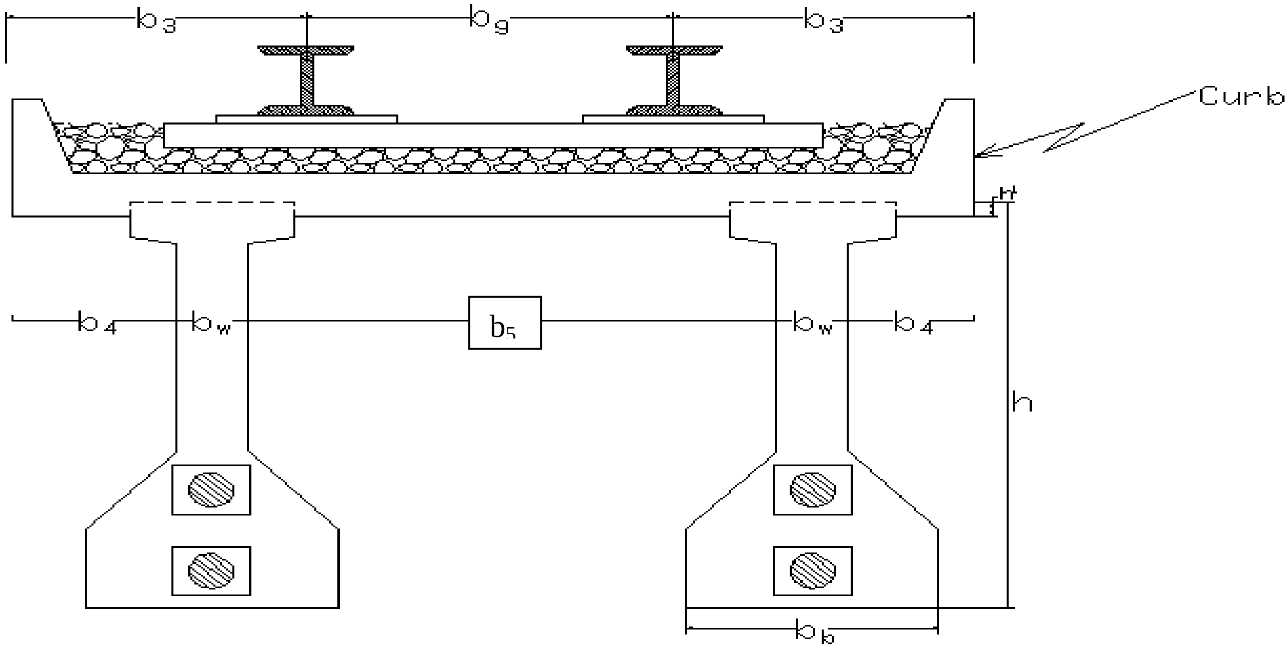


Figure: 4.1 Cross section of a Prestressed Concrete Girder Railway Bridge.

Table 4.1 Dimensions of the bridge.

Span length (mm)	Standard Rail gauge b_g(mm)	Girder clear spacing b_5(mm)	Live load configuration	Precast web b_w(mm)	b_4(mm)	h'(mm)
50,000	1435	2000	E-80	550	1200	200

The parameters are

C-50 Mpa, S- 500 Mpa, f_{pk} = 1800 Mpa, Υ_c = 24 Mpa, Υ_{RC} = 25 Mpa, Υ_{steel} = 78.5 Mpa, $\Upsilon_{ballast}$ = 19 Mpa.

- Concrete strength at transfer fck=50/1.25=0.8 *50 = 40Mpa
- Pre-stressing Tendons, $f_{pk} = 1800 Mpa$
- Yield strength of tendons, fpi = 0.75 fpk = 0.75 * 1800= 1350Mpa

- Stress limit for Pre-stressing Tendons
- Before transfer, $f_{pi} \leq 0.75 f_{pk} = 0.75*1800 = 1350 Mpa$
- After all losses, $f_{pe} \leq 0.82 fpi = 0.8*1350 = 1107 Mpa$
- fpe = 0.82 *1350Mpa = 1107Mpa
- f_{cd} = (0.85 * f_{ck})/1.5 = (0.85 * 40)/1.5 =
- $f_{ctd} = f_{ctk} / \Upsilon_c$, $f_{ctk} = 0.21 * f_{ck}^{2/3}$
- $f_{ctk} = 0.21 * 40^{2/3} = 2.46$ Mpa
- $f_{ctd} = f_{ctk} / \Upsilon_c = 2.46/1.5 = 1.64$ Mpa

Mild steel reinforcement, S-500

- Yield strength, $fyk = 500 Mpa$
- Modullus of elasticity , $E_s = 200 Gpa$ **(AREMA Art. 2.23.4 b)**
- Modulus of elasticity of cast in place concrete $E_c = 4700\sqrt{f_{ck}} = 4700\sqrt{40} = 29725.41 Mpa$

  or **(AREMA Art. 2.23.4 a)**

  $E_c = \gamma_c^{1.5} * 0.043\sqrt{f_{ck}} = 2400^{1.5} * 0.043\sqrt{40} = 31{,}975.35 Mpa$
- For pre cast girder at transfer, $E_{ci} = 4700\sqrt{37.5} = 28781.5 Mpa$
- For pre cast girder at service, $E_c = 4700\sqrt{40} = 29{,}725.41 Mpa$
- Modular ratio between slab and girder, $n = \frac{E_c(slab)}{E_c(girder)} = 1$

For S-500(Mpa), fyk = 500Mpa.

fyd = fyk / ϒs = 500 / 1.15 = 434.78 Mpa

Permissible stresses immediately at transfer

fct = 0.6* fck = 0.6 *40 = 24MPa

$F_{tt} = 0.21* f_{ck}^{2/3} = 0.21 * 40^{2/3} = 2.46$ Mpa

Permissible stresses under working condition (after all losses)

fcw = 0.5 * fck = 0.5 * 40 = 20MPa

$F_{tw} = 0.75 * 0.21* f_{ck}^{2/3} = 0.75 * 0.21 * 40^{2/3} = 1.84$ Mpa

Permissible range of stresses at the top of the cross section

ftr = (fcw + η ftt) ; Where, η-is pre-stress loss factor = 0.85

ftr = (20+0.85 * 2.46) = 22Mpa

Permissible range of stresses at the bottom of the cross section

fbr = η fct + ftw

fbr = 0.85* 24 +1.84 = 21.5 Mpa

Geometric properties

Span length $L = 50m$

Depth of girder $h = 390cm$

Clear spacing of girder $S = 2m$

Center to center girder spacing $S = 2.55m$

Deck thickness $t_s = 30cm$

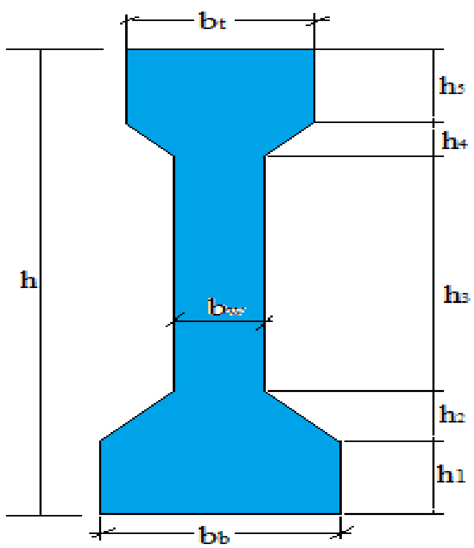
Assume the required cross sectional dimensions

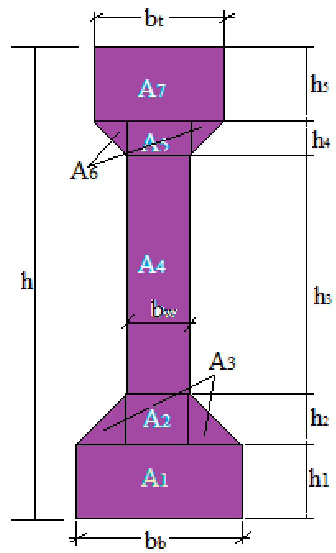
Recommended minimum depth of girder is h = 0.07S **(AASHTO Art. Table 8.9.2)**

Where, S is span length of girder in meter

For 50m span, h = 0.07*50m = 3.5m

b_b	100cm
b_t	80cm
b_w	55cm
h_1	45cm
h_2	60cm
h_3	200cm
h_4	40cm
h_5	45cm





Centroid of the pre-cast is calculated as:-

Let centroid of $A_1 = \check{y}_1$, $A_2 = \check{y}_2$, $A_3 = \check{y}_3$, $A_4 = \check{y}_4$, $A_5 = \check{y}_5$, $A_6 = \check{y}_6$ and $A_7 = \check{y}_7$

$A_P * \check{y}_p = A_1 * \check{y}_1 + A_2 * \check{y}_2 + A_3 * \check{y}_3 + A_4 * \check{y}_4 + A_5 * \check{y}_5 + A_6 * \check{y}_6 + A_7 * \check{y}$

$$\check{y}_p = \frac{A1 * \check{y}1 + A2 * \check{y}2 + A3 * \check{y}3 + A4 * \check{y}4 + A5 * \check{y}5 + A6 * \check{y}6 + A7 * \check{y}}{AP}$$

Moment of inertia of precast is calculated as:-

Let D_i = is centroid of each area from pre-cast N.A

$\bar{I} = (\bar{I}1+ \bar{I}2+ \bar{I}3+ \bar{I}4+ \bar{I}5 + \bar{I}6 + \bar{I}7) + A_1 D_1^2 + A_2 D_2^2 + A_3 D_3^2 + A_4 D_4^2 + A_5 D_5^2 + A_6 D_6^2 + A_7 D_7^2$

$\bar{I}x = \Sigma \bar{I}i + \Sigma Ai * d_i^2$

Section modulus of precast structure

$Z_{bp} = \frac{Ix}{\check{y}p}$ $Z_{tp} = \frac{I}{(h - \check{y}p)}$

Section properties of pre-cast girder form Excel sheet is in the table below.

	A_p (cm2)	$\check{Y}_i$= h_b (cm) centroid of each area from bottom	$A*h_b$ (cm3)	Ix (cm4) moment of inertia	$A*(h_b-y_b')$^2
A1	4500	22.5	101,250	759,375	1.19E+08
A2	3300	75	247,500	990,000	39987204
A3	1350	65	87,750	270,000	19465528
A4	11000	205	2,255,000	36,666,667	4365412
A5	2200	325	715,000	293,333.3	43071494
A6	500	331.67	165,833.33	444,44.44	10744006
A7	3600	367.5	1,323,000	607,500	1.2E+08
∑	**26,450**		**4,895,333**	**39,586,875**	**3.6E+08**

Table - 4.2. Section properties of pre-cast girder

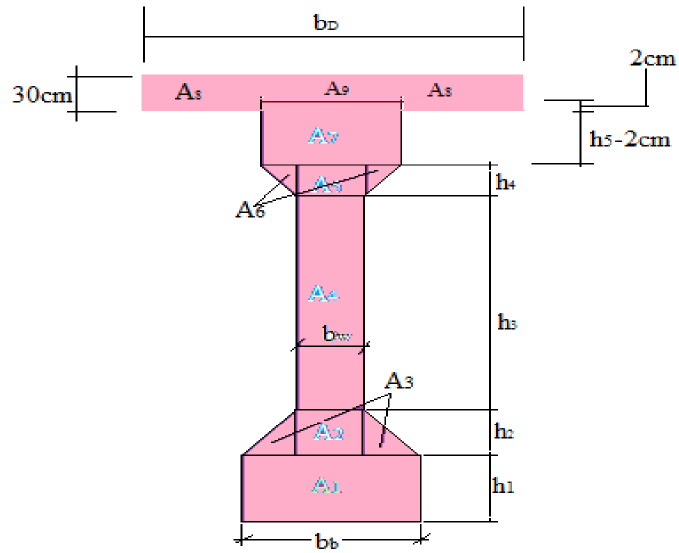
pre-cast girder

A_p	26,450 cm^2
$Ў_p$	185.078 cm
I_p	396,007,336 cm^4
y_t	204.92124 cm
Z_{bp}	2139,669.2 cm^3
Z_{tp}	1932485.6 cm^3

Effective flange width (AREMA Art. 17.10.1 b)

$$\text{The lesser of } \begin{cases} one\ \ fourth\ of\ \ girder\ \ span\ length\ = \frac{1}{4} * 50m = 12.5m \\ 6t_s\ \ on\ each\ side\ of\ \ the\ effective\ \ web\ width + effective\ \ web\ width\ = 4.1m \\ \frac{1}{2} clear\ \ dis\tan ce\ on\ each\ side\ of\ the\ effective\ \ web\ width + effective\ \ web\ width = 2.5m \end{cases}$$

Effective flange width, b_D = 2.5m



Composite section properties

Modular ratio, n = 1

Transformed effective length, 2.5m*1 = 2.5m

Eccentricities (mm)	1750.8
Concrete area, Ac (mm^2)	3.379
self-weight (KN/M3)	84.475
I (moment of inertia)=sum(Ii+Aidi^2) (m^4)	6.701579475
$Z_{tc}=I/y_t$	3.612376083
$Z_{bc}=I/y_b$	2.882613201

Regions	width	height	Area	Cgs from top	Ai*Yi	di	Ii	Ii+Ai*di^2
A1	1	0.45	0.450	3.955	1.77975	-2.100	0.00759375	1.991768
A2	0.55	0.6	0.330	3.43	1.1319	-1.575	0.0099	0.828327
A3	0.45	0.6	0.135	3.53	0.47655	-1.675	0.0027	0.381382
A4	0.55	2	1.100	2.13	2.343	-0.275	0.366666667	0.44975
A5	0.55	0.4	0.220	0.93	0.2046	0.925	0.002933333	0.191241
A6	0.25	0.4	0.050	0.86333	0.04316667	0.992	0.000444444	0.049632
A7	0.8	0.45	0.360	0.505	0.1818	1.350	0.006075	0.662342
A8	1.7	0.3	0.510	0.15	0.0765	1.705	0.003825	1.486707
A9	0.8	0.28	0.224	0.14	0.03136	1.715	0.001463467	0.66043
		sum	**3.379**		**6.26862667**			**6.701579**
				Y_b	**1.8552**			
				$y_t = h-y_b$	**2.3248**			

Table 4.3. Section properties of composite girder

Over all of composite section, $h_c = 390 + 30 = 420cm$

Distance from the centroid of composite section to top fiber of pre cast section, $y_{tg} = 1.575m$

Section modulus of Composite Girder

$Z_t c = 3.612376\text{m}^3 = 3.6*10^9\text{mm}^3$

$Z_b c = 2.8826\text{m}^3 = 2.9*10^9\text{mm}^3$

$Z_{tg} = \frac{I_c}{y_{tg}} = 4.2544\text{m}^3 = 4.3*10^9\text{mm}^3$

1. Determine maximum moment and shear force due to live load

From table 2.1 above, the maximum moment due to live load (Cooper E-80) for span length 50m is 17,137.5KN-m and moment due to impact load (20% of live load) = 3,427.5KN-M. The maximum moment due to live load

(Mq) = 17,137.5 KN-m + 3,427.5 KN-m = <u>20,565KN-m</u>

The maximum shear force due to live load for span length 50m is 1,501.78KN and impact load from live load is 20% of live load is = 300.36 KN.

The **<u>maximum shear force</u>** due to live load = **1,501.78KN + 300.36 KN = <u>1,802.14KN</u>**

2. Find maximum moment and shear force due to self -weight.

i. Pre-cast

Area of precast Ap = 2.65m^2 (from above table)

- Self – weight = Υ_p* Ap = 25KN/m^3 * 2.65m2 = **66.25 KN/m**
- Maximum moment = (M_{gl}) = $\frac{w*L^2}{8} = \frac{66.25*50^2}{8}$ = **20,703.13 KN-m**
- Maximum shear force = $\frac{w*L}{2} = \frac{66.25*50}{2}$ = **1,656.25KN**

ii.**Composite structure**

Area of composite section Ac= 3.38 m^2

- Self – weight = Υ_c* Ac = 25KN/m^3 * 3.38m^2 = **84.5KN/m**
- Maximum moment = (M_{gl}) = $\frac{w*L^2}{8} = \frac{84.5*50^2}{8}$ = **26,406.25 KN-m**
- Maximum shear force = $\frac{w*L}{2} = \frac{84.5*50}{2}$ = **2,112.5 KN**

3. Find maximum moment and shear force due to superimposed load.

i. Track rail, inside guardrails and fastenings = 3kN/m **(AREMA Art 2.2.3 b)**

For one girder = **1.5kN/m/girder**

ii. Ballast + track ties = 19kN/m^3*0.4(5+5.3) = 78.28kN/m

For one girder = **39.14kN/m/girder** (10.3= length on w/c ballast distribute. 0.4= thickness of ballast.)

iii. Deck slab = 25kN/m^3*0.3*2.5 = **18.75kN/m/girder**

For the two girders = 37.5kN/m

iv. Concrete curb = 25kN/m^3*1/2(0.188+0.3)0.75 = **7.338kN/m/girder**

For both sides 7.338*2 =14.775kN/m

v. Earth filling materials (sand) = 19kN/m^3*0.15m*(5+5.3)=29.36kN/m

For one girder = **14.68kN/m/girder**

vi. Waterproofing and protective covering = 0.15kN/m

For one girder = **0.075kN/m/girder**

vii.Catenary system = 0.02kN/m

For one girder = **0.01kN/m/girder**

viii. Future utilities = 5lbs/ft2/deck =0.05kN/m

For one girder = **0.025kN/m/girder**

From above; the total super imposed load on girder

=1.5 KN/m + 39.14 KN/m + 18.75KN/m + 7.338KN/m + 14.68 KN/m + 0.075KN/m + 0.01KN/m + 0.025KN/m = **81.52 KN/m**

- Maximum moment = $(M_{g2}) = \frac{w*L^2}{8} = \frac{81.52*50^2}{8}$ = **25,475 KN-m**
- Maximum shear force = $\frac{w*L}{2} = \frac{81.52*50}{2}$ = **2,038 KN**

Checking of Section Modulus in Different Stages

ztp > (Mq+(1-η) *Mg1)/ftr or ztc > (Mq+Mg2+(1-η) *Mg1)/ftr

zbp > (Mq+(1-η)* Mg1)/fbr or zbc > (Mq+Mg2+(1-η) *Mg1)/fbr

Check section modulus at transfer

$[Mq+ (1-\eta) * Mg_1] / ftr$ = [20,565+ (1- 0.85)) × 20,703.13] *1,000,000 / 22 = 1.08×10^8 mm^3

$1.08 * 10^7\ mm^3 \leq 1.93 * 10^9\ mm^3$ ---------Ok!

$[Mq+ (1-\eta) * Mg_1] / fbr$ = [20,565+ (1- 0.85) * 20,703.13] * 1.000,000 / 21.5 = $1.1 * 10^8\ mm^3$

$=1.1 * 10^8\ mm^3 \leq 2.14 * 10^9\ mm^3$ ------Ok!

Check section modulus at working

$[Mq + Mg_2 + (1-\eta) * Mg_1]/ftr$ = [20,565+ 25,475 + (1-0.85) × 26,406.25] * 1,000,000/22

$= 2.2*10^8\ mm^3 = 2.2*10^8\ mm^3 \leq 3.6 \times 10^9$ mm3 ---------Ok!

$[Mq + Mg_2 + (1-\eta) * Mg_1]/fbr$ = [20,565+ 25,475 + (1-0.85) * 26,406.25]×1,000,000/21.5

$= 2.3 \times *10^8\ mm^3 = 2.3 * 10^8\ mm^3 \leq 2.9 \times 10^9$ mm3-------------Ok!

Load combination

For this project Load Factor Design load combination Group 1 = 1.4(D+5/3(L+I)) is applied. B/c the combination has almost includes all types of loads on the girder.

- Self wt = 84.5 KN/m
- Dead Load on Girder = 81.52 KN/m
- Live load + Impact load = 65.81 KN/m
- Total Factored load = 1.4(DL+5/3(L+I)) = **385.98 KN/m**; where D = 84.5+81.52 = 166.02

Factored shear force $Vd = 9{,}649.5KN$

Factored bending moment $Md = 120{,}618.75kN.m$

Initial pre - stressing force to be applied is calculated as:-

Minimum pre - stressing force to be applied on concrete is expressed as

e = ȳ-concrete cover-1/2 Ø where, Ø – is diameter of tendon it is assumed as 100mm.

e =1,850.8- 50 - 0.5 *100 = **1,750.8mm**

$f_{lw} = P_e/A_{cp} + Pe * e/Zb_p - M(g_1+g_2+q)/Z_{bc} \geq -f_{tw}$, but $-f_{tw}$ is zero for fully pre-stressed,

Then $P_e/A_{cp} + P_e * e/Zb_p = M(g_1+g_2+q)/Zbc$

$P_e = M(q+g_2+g_1) * Z_{bc} * A_{cp} * Z_{bp} / (Z_{bp} + A_{cp} * e))$

$$P_e = \frac{M(q+g1+g2)*Acp*Zbp}{Zbc\ (Zbp+Acp*e)}$$

$$P_e = \frac{(20{,}565+26{,}406.25+25{,}475)*1{,}000{,}000*2{,}645{,}000*2{,}139{,}669{,}200}{2{,}882{,}613{,}201*(2{,}139{,}669{,}200+2{,}645{,}000*1750.8)} = \mathbf{21{,}034KN}$$

P_e = 21,034KN (Initial pre - stressing force).

$P = P_e * \eta$ = 21,034*0.85 = **17,878.90KN** (effective pre-stressing force after all losses)

Determination of Number of high strength steel wires

Area of tendon = $\frac{Pe}{fpe}$ = 21,034/1107 = 19,000.90 mm^2

Assume wire diameter is 7mm

Area of one wire = $\frac{\pi * d^2}{4} = \frac{\pi * 7^2}{4}$ = 38.47mm^2

Number of high strength steel wires required (N) = $\frac{Area\ of\ tendon}{Area\ of\ one\ wire}$ = 19,000.90/38.47 = 494

N = 494 wires are required to pre-stress the concrete Girder.

Determination of Deflection

I. Deflection due to pre-stress (for parabolic tendons)

E_c=wc * 1.5 * 0.043 * (f'c) $^{1/2}$

w_c = Unit weight of concrete is 25KN/m = 2500kg/m3

f'_c = Characteristics compressive strength of concrete

f_{ck} =40Mpa,

E_c = 2500 * 1.5 * 0.043 * $40^{0.5}$ *10^{-3}

Ec= **33.99 GPa**

I_p = 396,007,336* $10^4 mm^4$

$$\Delta = \frac{-5 \times P \times e \times L^2}{48 \times E_c \times I}$$

$$\frac{-5 * 17{,}878{,}900 * 1780.8 * 50{,}000^{\wedge}2}{48 * 33.99 * 10^{3} * 396{,}007{,}336 * 10^{4} mm^{\wedge}4}$$

Δp **= -61.59mm**……………. **(Up wards)!**

II. Deflection due to beam self-weight at transfer:

$$\Delta = \frac{5 \times W \times L^4}{384 \times E_c \times I}$$

$$\frac{5 * 66.25 * 50^{\wedge}4 * 10^{\wedge}12}{384 * 33.99 * 10^{3} * 6{,}701{,}579{,}475 * 10^{3} mm^{\wedge}4}$$

Δg**=23.67mm**……………. **(Down wards)!**

III. Deflection due to uniformly distributed superimposed dead load:

$$\Delta = \frac{5 \times W \times L^4}{384 \times E_c \times I}$$

$$\frac{5 * 81.52 * 50^{\wedge}4 * 10^{\wedge}12}{384 * 33.99 * 10^{3} * 6{,}701{,}579{,}475 * 10^{3} mm^{\wedge}4}$$

Δs **=29.12mm**……………. **(Down wards)!**

IV. Deflection due to live load:

W_{eq} is uniformly distributed live load and Impact. From above MLL+I = 20,565KN-m

Hence the value of distributed live load and impact is,

W_{eq} = (8*MLL+I) /L^2 = 8*20,565 / 50^2 = **65.81 KN/M**

$$\Delta = \frac{5 \times W \times L^4}{384 \times E_c \times I}$$

$$\frac{5 * 65.81 * 50^{\wedge}4 * 10^{\wedge}12}{384 * 33.99 * 10^{3} * 6{,}701{,}579{,}475 * 10^{3} mm^{\wedge}4}$$

Δ_{LL+I} **=23.52mm**

But, the maximum allowable deflection due to live load plus impact shall not exceed L/640. Therefore, L/640 = 50,000/640 = 78.13mm, ΔLL+I = **23.52mm < 78.13mm**……………..OK!!

VI. Deflection due to self-weight under working:

$$\Delta = \frac{5\times W\times L^4}{384\times E_c\times I}$$

$$\frac{5 * 84.5 * 50^4 * 10^12}{384 * 33.99 * 10^3 * 6{,}701{,}579{,}475 * 10^3 mm^4}$$

=**30.19mm**

VII. Long time deflection:

Long time deflection is due to creep and shrinkage of concrete and relaxation of stresses in steel. Therefore, the total long time deflection is

Δl = Δc * (1+ Φ) – Δp [(1- ΔP/P) +Φ * (1- ΔP/P)]

Where;

Δc = Initial deflection due to transverse loads

Δl = Long time deflection

Δp = Initial deflection due to pre stress force

Φ= Coefficient which account creep, shrinkage etc. (Φ =2.0)

ΔP = P-Pe = loss of pre stressing force due to relaxation, shrinkage & creep.

$\Delta_c = \frac{5*W*L^4}{384*Ec*Ic}$ Where; W = the sum of live load, composite self-weight and superimposed dead load. W = 65.81 + 84.5 + 81.52 = **231.83 KN/m**

$$\frac{5 * 231.83 * 50^4 * 10^12}{384 * 33.99 * 10^3 * 6{,}701{,}579{,}475 * 10^3 mm^4}$$

Δ_c =**82.82mm ……………. (Down wards)!** This much deflection happened is due to large span length of the Girder as result of large live load, impact and dead load moments.

$\Delta P = P - \eta P = P-Pe$,

Where; η-is pre-stress loss factor = 0.85.

$\Delta P = P- \eta Pe = 17{,}878.90 - 0.85\times21{,}034 =$ **0 KN**

$\Delta l = \Delta c \times (1+ \Phi) – \Delta p$(deflection due to pre-stress) * $[(1- \Delta P/P) + \Phi * (1- \Delta P/P)]$

$\Delta l = 82.82\times (1+2) – 61.59\times [(1- 0/17{,}878.90) + 2* (1- 0/17{,}878.90)]$

$\Delta l =$ **63.69mm**………. (Downwards)

Check web shear and design for flexural shear

Are governed by limiting value of the principal tensile stress.

I. Web shear:

fcp = Pe/Apc , but Pe= As × fpe , therefore; fcp =(As fpe)/Apc

fcp = Pe/Apc, = 21,034,000/2,645,000 = 7.95MPa

From above calculated values:

Factored shear force V_d = 9,649.5KN

b_w = 550mm, h = 3,500mm

$V_{cw} = (\eta * b_w * h * \sqrt{fctd^2 + fcp * fctd} * (10^{-3})$

$= (0.85 * 550 * 3{,}500 * \sqrt{1.64^2 + 7.95 * 1.64} * (10^{-3}) = 6{,}489.13KN$

V_d = 9,649.5KN > V_{cw} = 6,489.13KN………………Not okay!!!

Therefore it required to provide stirrups for shear. We have to determine the shear sustain rebar's

$$Vs = av\times d\times fyd /s$$

II. Flexural shear:

Vc (section capacity) = $\beta1 \times0.5\times b_w\times d\times fctd$

Where, $\beta1 = 1+ Mo / Mu$

From above calculated value:-

Md (factored B.M) = 120,618.75KN-M

A_{cp} = 2,645,000mm^2, e = 1750.8mm, P = 24,745,880 N, Z_{bp} = 2,139,669,200mm^3

Mo = P (Z_{bp}/Acp + e).

Mo = 17,878,900 * (2,139,669,200/2,645,000 + 1750.8) * 10^{-6} = 45,765.49KN-M

β1 = 1+ (Mo / Md) $\leq$ 2.0

= 1 + (45,765.49 /120,618.75) = 1.38 $\leq$ 2.0 ………..okay!!!!

Vc = β1 ×0.5×b_w×d×fctd

β1 = 1.89, b_w = 550mm, d = hc – cover- 1/2 diameter of tendon = 4200 – 50-0.5×100= 4,100mm

fctd = 1.64 MPa.

Vc = (1.89 ×0.5×550×4,100×1.64) × (10-3) = 3,494.79 KN

Vc = 3,494.79 KN < V_d = 9,649.5KN ……………..Not Okay!!

Therefore it requires to provide stirrups for shear. We have to determine the shear sustain rebar's

Vs = av×d×fyd /s

Assume that spacing of stirrup is S =150mm, Vs = Vd -Vc =5,383.3- 3,494.79 =1,888.51 kN and

av = (Vs × S / d × fyd) = 1,888.51*1000 × 150 / (4,100× 434.78) = 157.83mm^2.

Assume that stirrup diameter is Ø10.

A = $\frac{\pi * d^2}{4}$ = $\frac{\pi * 10^2}{4}$ = 78.54mm^2

Number of stirrups (N) = av/A = 157.83mm^2/78.54mm^2 = **3**

VRd = 0.25 × fcd × bw ×d = 0.25×22.667×550×4100 × (10^{-3}) =**12,778.52 kN > Vd =5,383.3KN** ……Ok! The section is adequate.

Bearing and anchorage zone design

Anchorage bearing stress $= Min.\begin{cases} 21Mpa \\ 0.9\, f_{ci}^{'} \end{cases}$ **(AREMA 17-16.2.4)**

0.90f'ci=0.90*0.75*50=33.75

So take Anchorage bearing stress, fc= 33.75

For P=24,745.88 KN Aa=P/fc=24,745.88/33750=0.733 m^2

<u>= square of 830mm*830mm</u>

Post tensioning anchorages and couplers **(AREMA Art. 17.5.6)**

Shall develop not less than 95% of the ultimate tendon strength and avoid located in high stress areas.

Tendon anchorage zone **(AREMA Art. 17.5.7)**

Provide reinforcement at anchorage zone to prevent bursting, splitting and spalling. Post tensioning anchorage should be checked the resistance of maximum jacking force.

4.2 Excel Template used for Analysis and Design of pre- stressed concrete Girder Railroad Bridge.

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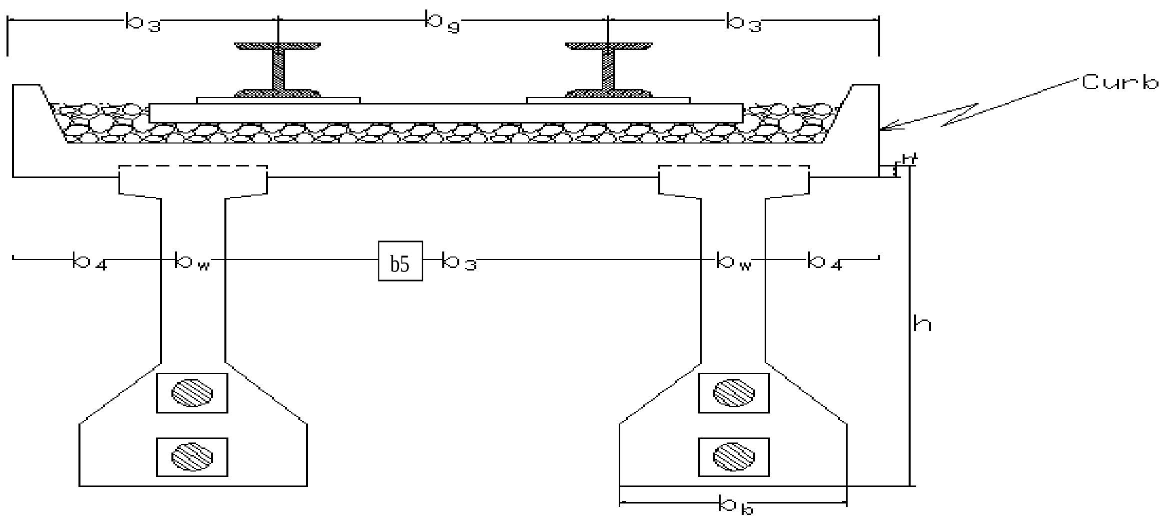
MICCRO SOFT EXCEL PROGRAM FOR DESIGN OF PRE-STRESSED CONCRETE GIRDER RAIL ROAD BRIDGE.

Sample Example : For 50m Span Bridge

A simple span precast post tensioned pre-stressed concrete girder bridge is to be designed for a railway bridge with cast in-situ deck slab, cross section of which shown in Figure, below. The cast in place concrete deck slab is of 300 mm thick, while the ballast below the sleepers is 250 mm in thickness. Additional ballast above the bottom of the sleepers may be assumed of 150 mm. The bridge is to serve for the Cooper E-80 Rail load configuration and the following design data may be used.



Materials

*	Concrete C-50	50	**50**
*	Mild steel S-!	500	**500**
*	Pre-stressing Tendons fpk = 1800 MPa		**1800**
*	Unit weight of plain concrete 24 kN/m3		**24**
*	Unit weight of RC 25 kN/m3 (ϒp)		**25**
*	Unit weight of Steel 78.5 kN/m3		**78.5**
*	Unit weight of sand, gravel and ballast 19 kN/m3		**19**

Girder Dimensions

Span length (mm)	Rail gauge (bg)	Girder clear spacing (b3)	Live load configuration	Pre-cast web(bw)	b4	h'
50000	1435 mm		E-80			

As we can see; as we have insert the **span length (first step)** of a Girder all the necessary cross sectional dimensions, Areas, moment of inertia, Centeriod and estimated minmum depth of the Girder are calculated once.

second step is fill all un input data;s (Track rail, inside guardrails and fastenings, Ballast + track ties, Deck slab, Concrete curb, Earth filling materials (sand) , Waterproofing and protective covering , * Catenary system , Future utilities, Wind load) loads. Then moments and shear forces are calculated automatically. then we are going to check the section capacity and deflection of the section at different stages.

The parameters are

C-50 Mpa, S- 500 Mpa, fpk = 1800 Mpa, ϒc = 24 Mpa, ϒRC = 25 Mpa, ϒsteel = 78.5 Mpa, ϒballast = 19 Mpa.

Concrete strength at transfer fck 40 Mpa
Yield strength of tendons, fpi = 0.75 fpk 1350 mpa
Stress limit for Pre-stressing Tendons
Before transfer,
fpi <= 0.75* fpk = 1350 Mpa
After all losses,
fpe < = 0.82 *fpi = 1107 Mpa
fcd = (0.85 * fck)/1.5 = 22.67 Mpa
fctd = fctk/ ϒc, fctk = 0.21 * fck^2/3 **2.46 Mpa**
fctd = fctk/ ϒc = **1.64 Mpa**

Mild steel reinforcement, S-500
Yield strength, fyk = **500 Mpa**
Modullus of elasticity ,
$E_s = 200Gpa$ (AREMA Art. 2.23.4 b)

Modulus of elasticity of cast in place concrete (AREMA Art. 2.23.4a)
Ec = ϒ^1.5 * 0.043*squr fck = **31,975.35 Mpa**
For pre cast girder at transfer,
Eci = 4700*squr fck 29,725.41 Mpa
For pre cast girder at service,
Eci = 4700*squr fck 29,725.41 Mpa

For S-500(Mpa), fyk = **500 Mpa**
fyd = fyk / ϒs = **434.78 Mpa**

Permissible stresses immediately at transfer
fct = 0.6* fck = **24 Mpa**
Ftt = 0.21* fck 2/3 = **2.46 Mpa**
Permissible stresses under working condition (after all losses)
fcw = 0.5 * fck = **20 Mpa**
Ftw = 0.75 * 0.21* fck 2/3 = **1.84 Mpa**
Permissible range of stresses at the top of the cross section
ftr = (fcw + η ftt) ; Where, η-is pre-stress loss factor = 0.85
ftr = **22.09 Mpa**
Permissible range of stresses at the bottom of the cross section
fbr = η fct + ftw **22.24 Mpa**

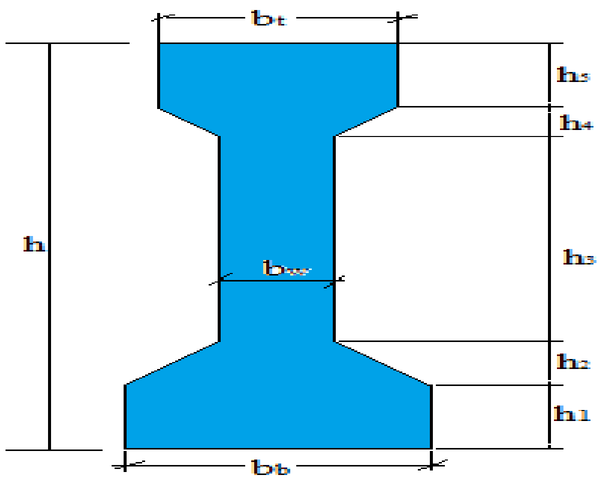
Determine the required crossectional Dimenssions of the Girder

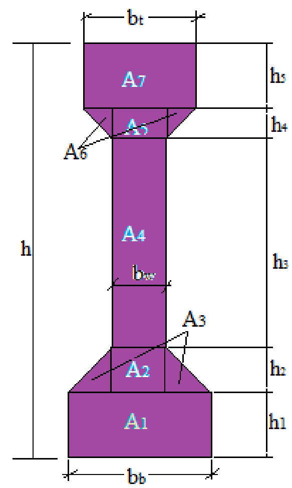
Recommended minimum depth of girder is **h = 0.07S** (AASHTO Art. Table 8.9.2)
Where, S is span length of girder in meter

h = 0.07S **3500** mm

in mm

b_b	1050
b_t	875
b_w	350
h_1	525
h_2	350
h_3	1400
h_4	175
h_5	350





Section properties

Moment of inertia of precast is calculated as:-

Let yb' (Di) = is centroid of each area from pre-cast N.A

$$\bar{I} = (\bar{I}1+ \bar{I}2+ \bar{I}3+ \bar{I}4+ \bar{I}5 + \bar{I}6 + \bar{I}7) + A1\ D1^2 + A2\ D2^2 + A3\ D3^2 + A4\ D4^2 + A5\ D5^2 + A6\ D6^2 + A7D7^2$$

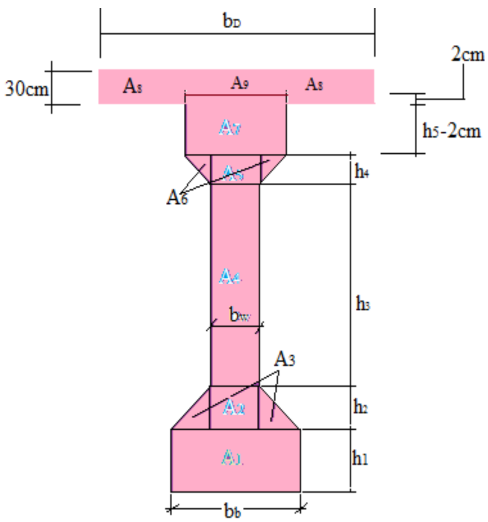
$$\bar{I}xx = \Sigma\ \bar{I}i + \Sigma Ai * di^2$$

	A (mm2)	hb (mm)	A*hb (mm3)	Ix (mm4)	A*(hb-yb')^2
A1	551250	262.5	144703125	12661523438	5.3325E+11
A2	122500	700	85750000	1250520833	36524327064
A3	245000	641.6666667	157208333.3	833680555.6	6.36876E+11
A4	490000	1575	771750000	80033333333	53025757711
A5	61250	2362.5	144703125	156315104.2	76347318715
A6	91875	2391.666667	219734375	78157552.08	1.20583E+11
A7	306250	2625	803906250	3126302083	5.82345E+11
∑	**1868125**		**2327755208**	**98139832899**	**2.03895E+12**

A	1868125	mm2
yb'	1246.038251	mm
Ixx	2.13709E+12	mm4
yt	2253.961749	mm
Zbp	1715109340	mm3
Ztp	948149117.5	mm3

Section properties of Composite girder

Assume thickness of slab deck = (cm) **30**



AREMA Art. 17.10.1 b)

$$\text{The lesser of}\begin{cases} \text{one fourth of girder span length} = \frac{1}{4} * 50m = 12.5m \\ 6t_s \text{ on each side of the effective web width} + \text{effective web width} = 4.1m \\ \frac{1}{2}\text{clear distance on each side of the effective web width} + \text{effective web width} = 2.5m \end{cases}$$

Effective flange width, b_D = 2.5m

Regions	width	height	Area	Yi top	Ai*Yi	di	Ii	Ii+Ai*di^2
A1	1050	525	551250.000	3237.78	1784826225	-1339.259	1.3E+10	1.00139E+12
A2	350	350	122500.000	2800.28	343034300	-901.759	1.3E+09	1.00864E+11
A3	700	350	245000.000	2858.613333	700360266.7	-960.092	8.3E+08	2.26669E+11
A4	350	1400	490000.000	1225.28	600387200	673.241	8E+10	3.02128E+11
A5	350	175	61250.000	437.78	26814025	1460.741	1.6E+08	1.30849E+11
A6	525	175	91875.000	408.6133333	37541350	1489.908	7.8E+07	2.04025E+11
A7	875	350	306250.000	175.28	53679500	1723.241	3.1E+09	9.12554E+11
A8	-872.5	0.3	-261.750	0.15	-39.2625	1898.371	-1.9631	-943297974.9
A9	875	0.28	245.000	0.14	34.3	1898.381	1.60067	882943413.9
		sum	**1868108.250**		**3546642862**			**2.87842E+12**
				yb	**1898.5211**			
				yt=h-yb	**1601.4789**			

Section Modulus

Z_{tc}=I/y_t **1797350837** mm4

Z_{bc}=I/y_b **1516137778** mm4

Determine maximum moment and shear force due to live load

these are **un input data's (read from AREMA mauel) based on the span length of a Girder,**

From AREMA manuel (2003) the Maximum Bending moment and shear force due to Live Load (LL) (Cooper E-80) for different span length has tabulated and we can read the values from the table for required span length. Impact load due to live load is 20% of LL and add to both Moment and shear force.

For 50m span Girder Mq = 17,137.5KN-m and Vq = 1,501.78KN

Mq - is Maximum moment due to live load and 20% of Mq is moment due to Imapact load; hence the maximum moment due to LL = Mq +20%of Mq **20,565.00 KN-m**

* Mq **17,137.50** KN-m

Vq - is Maximum shear force due to live load and 20% of Vq is shear force due to Imapact load; hence the maximum shear force due to LL = Vq +20%of Vq **1,802.14 KN**

* Vq **1,501.78** KN

Find maximum moment and shear force due to self -weight.

i. Pre-cast

Area of precast Ap= **1868125** m2

Self – weight = ϒp* Ap = **46703125** KN/m

Maximum moment (Mgl) = **1.45947E+16** KN-m

$$\frac{w*L^2}{8}$$

Maximum shear force = **1.16758E+12** KN

$$\frac{w*L}{2}$$

i. Composite structure

Area of composite section Ac= **1868108.250** m2

Self – weight = ϒc* Ac = **46702706.25** KN/m

Maximum moment (Mgl) = **1.45946E+16** KN-m

$$\frac{w*L^2}{8}$$

Maximum shear force = **2.33514E+12** KN

$$\frac{w*L}{2}$$

Find maximum moment and shear force due to superimposed load.

these are **un input data's**; they are depends on the design type, interest and materials used.

Let us fill the following un input loads (abrbtirarly)

The following superimposed loads are may applied on the girder bridge

* Track rail, inside guardrails and fastenings	1.5	KN/m
* Ballast + track ties	39.14	KN/m
* Deck slab	18.75	KN/m
* Concrete curb	7.34	KN/m
* Earth filling materials (sand)	14.68	KN/m
* Waterproofing and protective covering	0.075	KN/m
* Catenary system	0.01	KN/m
* Future utilities	0.02	KN/m
* Wind load	0	KN/m
Total superimposed load on Girder =	81.515	KN/m

Maximum moment (Mg2) = **25473437500** KN-mm

$$\frac{w*L^2}{8}$$

Maximum shear force = **2037875** KN

$$\frac{w*L}{2}$$

Checking of Section Modulus in Different Stages

ztp > (Mq+(1-η) *Mg1)/ftr or ztc > (Mq+Mg2+(1-η) *Mg1)/ftr

zbp > (Mq+(1-η)* Mg1)/fbr or zbc > (Mq+Mg2+(1-η) *Mg1)/fbr

Check section modulus at transfer η = 0.85

ztp > (Mq+(1-η) *Mg1)/ftr-----Okay!! **0** !!!

zbp > (Mq+(1-η)* Mg1)/fbr-----Okay!! **0** !!

Key - 0 = Not okay!! 1= Okay

**If this condition is not okay; Revise the Girder size

Check section modulus at working

ztc > (Mq+Mg2+(1-η) *Mg1)/ftr---Okay!! **0** !!!

zbc > (Mq+Mg2+(1-η) *Mg1)/fbr…Okay **0** !!!

Key - 0 = Not okay!! 1= Okay

** If this condition is not okay; Revise the Girder size

Load combination

Load Factor Design load combination Group 1 = 1.4(D+5/3(L+I))

* Self wt =	**46702706.25**	**KN**
* Dead Load on Girder =	**81.515**	**KN**
* Live load + Impact load = Read from AREMA table for different sapn length for live load E-80	**0**	**KN**
TOTAL	**46702787.77**	**KN**

Factored shear force

Vd **1.16757E+12** **KN**

Factored bending moment

Md **1.45946E+16** **KN-m**

Determine Initial pre - stressing force to be applied

$$P_e = \frac{M(q+g1+g2)*Acp*Zbp}{Zbc\ (Zbp+Acp*e)}$$ **2.93011E+31** **KN**

Pe = is the intialpre-stressing force.

where; e = ȳ-concrete cover-1/2 Ø where, Ø – is diameter of tendon **1750.8** **mm**

P = Pe* η **2.4906E+31** **KN**

P = is the effective pre-stressing force after all losses.

Determination of Number of high strength steel wires

* Area of tendon = $\frac{Pe}{fpe}$ **2.64689E+28** mm2

*diameter(d) of wire.(mm) **7** mm

* Area of one wire = $\frac{\pi*d^2}{4}$ **38.465** mm2

d- diameter of the wire.(mm)

Number of high strength steel wires required (N) = **6.88131E+26**

$$\frac{Area\ of\ tendon}{Area\ of\ one\ wire}$$

Determination of Deflection

*** Deflection due to pre-stress**

$$\Delta = \frac{-5\times P\times e\times L^2}{48\times E_c\times I}$$ **-1.66177E+26** mm

Ec=wc * 1.5 * 0.043 * (f'c) 1/2 **31,975.35** Mpa

*** Deflection due to beam self-weight at transfer:**

$$\Delta = \frac{5\times W\times L^4}{384\times E_c\times I}$$ **41294796.98** mm

*** Deflection due to uniformly distributed superimposed dead load:**

$$\Delta = \frac{5\times W\times L^4}{384\times E_c\times I}$$ **72.07537774** mm

*** Deflection due to live load:**

ΔLL+I = (5*Weq * L^4)/(384*Ec * I) **5.81873E-05** mm

Weq = (8* Mq) /L^2 **0.000065808** KN/m

L/640 > ΔLL+I……….Okay!!! **1** !!! **Key - 0 = Not okay!! 1= Okay**

*** Deflection due to self-weight under working:**

Δs = (5*W * L^4)/(384*Ec * I) **41294426.73** mm

*** Long time deflection:**

Δl = Δc * (1+ Φ) – Δp [(1- ΔP/P) +Φ * (1- ΔP/P)]

$$\Delta = \frac{5\times W\times L^4}{384\times E_c\times I}$$ where; W = **46702787.77** KN-m

Δc = **41294498.8** mm

ΔP = P - η Pe = **0** KN

Δl = **#REF!** mm

Check web shear and design for flexural shear

I. Web shear:

Vcw = (η* bw * h * $\sqrt{fctd^2 + fcp * fctd}$
fcp = Pe/Apc , but Pe= As × fpe , therefore; fcp =(As fpe)/Apc
fcp = Pe/Apc **1.56848E+25** Mpa

Vcw = **8.64224E+18** KN
Vcw > Vd------Okay!

1 !!! **Key - 0 = Not okay!! 1= Okay**

If not okay provide stirrups for shear

II. Flexural shear:
Vc = β1 ×0.5×b×d×fctd

Where, β1 = 1+ Mo / Mu ≤ 2.0

Mo = P (Zbp/Acp + e).

Mo = **6.64713E+34** KN-m
β1 = **4.5545E+18**

β1 ≤ 2.0 **0** !!!! **Key - 0 = Not okay!! 1= Okay**

Vc = β1 ×0.5×bw×d×fctd

Vc = **4.43792E+24** KN

Vc > Vd----Okay!!!! **1** **Key - 0 = Not okay!! 1= Okay**

If not okay provide stirrups for shear

CHAPTER FIVE

5.0. CONCLUSION AND RECOMMENDATIONS

5.1. CONCLUSION

Bridge design contains a number of design steps which involves tedious calculations due to the presence of moving loads, dead (superimposed loads), lateral loads and needs choice of parameters and decision. So due to the complexity and time consuming of the design; it will be better if we use simple bridge design Micro soft excel programs to get better and accurate results in a short time.

The following points have been summarized as conclusions for this project.

1. This Project has shown it is possible to produce our own Micro soft excel sheets (programs) to facilitate our daily design works. Subsequently it is possible for preparing such Micro soft excel sheets (programs) not only in the bridge but also in different construction industry in the country.
2. This project work will contribute a little for the designers and consultants by showing how to produce our own user Micro soft excel sheets (programs) to have got the required out puts with in short period of time.
3. Since traditional methods for the analysis and design of bridge is tedious and time consuming, using Computer programs (Micro soft excel) for bridge design offers accuracy and flexibility that cannot be matched with traditional hand calculations.
4. This project will benefit the designers and bridge engineers a lot. For example, getting quick results of dimension and reinforcement detail for different span length of pre-stressed concrete girder Railway Bridge from the output of this product, one can do economic comparison between different spans of bridges.

5.2. RECOMMENDATIONS

The following points have been summarized as conclusions for this project.

1. This project has principally deal with design of simply supported (super structure) pre-stressed concrete Girder Rail Road Bridge. In the future I hope that this work is to be developed into highly potential software which incorporates design of continuous spans and also design of substructures with a more features by providing drawings and other important outputs. Future research could include other bridge types, such as T- Girder Bridge, Slab Bridge and others. Providing a collection of several different design methods, each illustrated with real-life examples would be a valuable source of information in teaching future bridge engineers.

2. It would be better to compare various design methods and thus generate a more extensive view of bridge engineering. Relating currently existing design procedures with new structural concepts would contribute to the knowledge in that current methods are assessed and possibly adjusted to future challenges in bridge engineering.
3. The university should build up students and the engineers to contribute something valuable for our country in the design of complex bridges for Railway and Highway.
4. Government should strength the technology transfer center for railway technology incubation (especially on over pass super Rail road Brides).
5. Ethiopia Railway Corporation should engage the local contractors and consultants in construction of such large over pass rail road bridges to gain experiences and in turn which create high job opportunities for the citizens and decrease the foreign currency burden on the country economy.
6. The ERC must involve and trained his Engineers (professionals) intensively to have technological transfer and experience gaining in design and construction of such overpass structures.

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8. Symbols which are used on Calculations

Apc = *Area of precast concrete.*

As = *Area of tendon.*

e = *Eccentricity of pre-stressing force.*

Es = *Modulus of elasticity of steel.*

Ec = *Modulus of elasticity of concrete.*

fbr = *Permissible range of stresses at the bottom of the cross section.*

fcd = *The compressive design strength of concrete.*

fck = *The characteristic cylinder compressive strength of concrete.*

fcp *= Compressive pre-stress at the centroid of a section.*

fct = *Allowable compressive Stresses in concrete at transfer.*

fctd = *The tensile design strength of concrete.*

fctk = *Tensile strength of concrete.*

fcu = *The value of the cube strength of concrete.*

fcw= *Allowable compressive Stresses in concrete under working.*

fd= *The design strength for a given material.*

fk = *The characteristic strength for all materials has the notation.*

fpe *= Effective stress in tendons after loss.*

fpi= *Stress in tendon at transfer.*

fpk= *Characteristics strength of tendon.*

ftr = *Permissible range of stresses at the top of the cross section.*

ftt = *Allowable tensile stress in concrete at transfer.*

ftw = *Allowable tensile stress in concrete under working.*

fy = *Characteristic strength of reinforcing steel.*

fyd = *The tensile or compressive design strength of reinforcing steel.*

Md = *The factored bending moment.*

Mg1 = *Bending moment due to self –weight.*

Mg2* = *Bending moment due to superimposed load.

Mo* = *Bending moment due to eccentricity and pre-stressing force.

Mq2* = *Bending moment due to the live load.

η = *Reduction factor for loss of pre-stress = 0.85.*

P = *Pre-stressing force at transfer on the cross section*

P_e= *Pre-stressing force under working condition (after all losses) on cross section.*

Vc = *Flexural Shear force.*

Vcw = *Web shear force*

Vd = *Design shear force (factored design shear force)*

Weq = *Equivalent uniform load.*

Z = *Section modulus.*

Zbc = *Section modulus, at bottom fiber for composite structure.*

Zbp = *Section modulus, at bottom fiber for precast structure.*

Ztc = *Section modulus, at top fiber for composite structure.*

Ztp = *Section modulus, at top fiber for precast structure.*

Δ LL+I = *Deflection due to live load and impact load.*

Δg = *Deflection due to self-weight load.*

Δp= *Deflection due to pre-stressing force.*

Δs = *Deflection due to superimposed load.*

DEFINITIONS OF TERMINOLOGY ON THIS PROJECT

Rail = *bar or pair of parallel bars of rolled steel making the railway along which railroad cars or other vehicles can roll.*

Sleeper = *track tie usually made of concrete or wood that is laid across a railroad bed to support the rails.*

Live load= *temporary load for short duration or a moving load. Bridge live loads are produced by vehicles traveling over the deck of the bridge.*

Dead load = *type of load that is relatively constant over time, including the weight of the structure itself and also known as permanent loads.*

Ballast = *Gravel or coarse stone used to form the bed of a railway track or the substratum of a road.*

Anchorage= *device generally used to enable the tendon to impart and maintain pre-stress in concrete.*

Bars = *tendon can be made up of a single steel bar. The diameter of a bar is much larger than that of a wire.*

Strands = *Two, three or seven wires are wound to form a pre-stressing strand.*

Tendons *= group of strands or wires are wound to form a pre-stressing tendon.*

Curb = *rim, especially of joined stones or concrete, along a bridge or roadway, forming an edge for a sidewalk.*

Jacking force = *temporary force applied to tendons by a jacking device to produce the tension in pre-stressing tendons.*

Jacking stress = *maximum stress that occurs during the stressing of a precast concrete tendon.*

Jacking plate *= steel bearing plate used during jacking operations to transmit the load of the jack to the pile.*

Mild steel = *Steel with less than 0.15% carbon.*

Post-tensioning *= method of pre-stressing concrete by tensioning the tendons against hardened concrete. In this method, the pre-stress is imparted to concrete by bearing.*

Pre-stressed concrete *= basically concrete in which internal stresses of a suitable magnitude and distribution are introduced so that the stresses resulting from the external loads are counteracted to a desired degree.*

Pre-stressing wire = *single unit made of steel.*

Pre-tensioning *= method of pre-stressing concrete in which the tendons are tensioned before the concrete is placed. In this method, the concrete is introduced by bond between steel& concrete.*

Cable *= group of tendons form a pre-stressing cable.*