ADDIS ABABA UNIVERSITY
SCHOOL OF GRADUATE STUDIES

FACULTY OF TECHNOLOGY
DEPARTMENT OF CIVIL ENGINEERING

PRESTRESSED CONCRETE BRIDGE DESIGN AND CONSTRUCTION PRACTICES AND PROSPECTS IN ETHIOPIA

BERUK BERHANE

JULY 2002

PRESTRESSED CONCRETE BRIDGE DESIGN AND CONSTRUCTION PRACTICES AND PROSPECTS IN ETHIOPIA

A Thesis submitted to the school of Graduate Studies, Addis Ababa University in Partial Fulfillment of the Requirements for the Degree of Master of Science in Civil Engineering

By Beruk Berhane

Advisor

Professor Negussie Tebedge

Addis Ababa University

Approved by Board of Examiners

Professor Negussie Tebedge		
Advisor	Signature	Date
Dr. Abebe Dinku		
External Examiner	Signature	Date
Dr. Shifferaw Taye		
Internal Examiner	Signature	Date
Dr. Zelalem Hailu		
Chairman	Signature	Date

Declaration

I, the undersigned, declare that the thesis is my original work, has not been presented for a degree in any other University and that all sources of materials used for the thesis have been duly acknowledged.

Name:	Beruk Berhane
Signature:	______________
Place:	Faculty of Technology,
	Addis Ababa University
Date of Submission:	July 1, 2002

Acknowledgments

I gratefully acknowledge my advisor, Professor Negussie Tebedge for his invaluable guidance and constant encouragements.

I also acknowledge my partners (Bezan Development P.L.C.) giving me the opportunity to attend my postgraduate study with all the support I needed.

The contribution of all organizations and individuals engaged in the design and construction of bridges in the country was also considerable to the completion of this thesis.

At last but not least I am very much tank full to all my friends for their encouragements and also W/t Essey for typing this manuscript.

Forward

We must always work hard to apply our knowledge in which we acquire it through long process of learning towards bringing a change in our living condition, as the peoples of the developed world did.

It is not a matter of coming up with new scientific investigation, but only to put our knowledge together and a little effort in adopting the existing well-developed technology to the best of our local conditions and capacities.

We must therefore prepare ourselves for new systems and be aware of the fact that there is always a better and better ways to solve our problems.

Table of Content

Symbols and Abbreviations

A_g = gross area of concrete section

A_c = net area of concrete section

A_p = area of prestressing steel

A_v = area of transverse reinforcement

a = depth of equivalent rectangular stress block

b = width of member

c = distance from extra compressive fiber to neutral axis

c.g.s =center of gravity of strand

c.g.c = center of gravity of concrete

d_v = effective shear depth

d = depth of member to c.g.s

e = eccentricity of strand centroid

E_p = elastic modulus of prestressing steel

E_c = elastic modulus of concrete

$f_c^{'}$ =concrete compressive strength at 28 days

f_{pu} = proof ultimate strength of prestressing steel

f_t = net top fiber stress

f_b = net bottom fiber stress

f_b = net bottom fiber stress

f_{ps} = allowable stress in prestressing steel

f_y = yield strength of reinforcing steel

f_p^t = top fiber stress due to prestressing

f_p^b = bottom stress due to prestressing

$f_{ci}^{'}$ = concrete strength at transfer

f_G^b, f_s^b, f_L^b = stress at bottom fiber due to girder load, slab load and live load respectively

f_G^t, f_s^t, f_L^t = ditto but at top fiber

I_c = moment of inertia of the net concrete section

I_g = gross moment of inertia of concrete

IM = dynamic load allowance

K = effective length factor for compressive member

L = length of bridge center to center of bearing

l = clear distance between girders

M_{DL} = dead load moment

M_{LL} = live load moment

M_{SS}=dead load from slab only

M_{SD} = dead load from diaphragm

M_r = ultimate moment resistance of a member

M_n = factored ultimate moment capacity of section

Ms or M_T = total moment at service

M_o = total moment at transfer

M_p = prestressing moment

PPR = partial prestressing ratio

P_{jack} = prestressing force at jack

P or F = prestressing force at service

P_i or F_o = prestressing force at transfer

S = center to center distance of girders or stirrups

V_c = concrete shear capacity

V_p = vertical component of prestressing force

V_n = nominal shear resistance

w_{DL} = dead load per meter length

y_b = centroid distance from bottom

Z_b = bottom section modulus of concrete section

Z_t = top section modulus of concrete

φ = resistance

ε = concrete strain

β = factor relating effect of longitudinal strain on the shear capacity of concrete

θ = angle of inclination of diagonal compressive

stresses(degree)

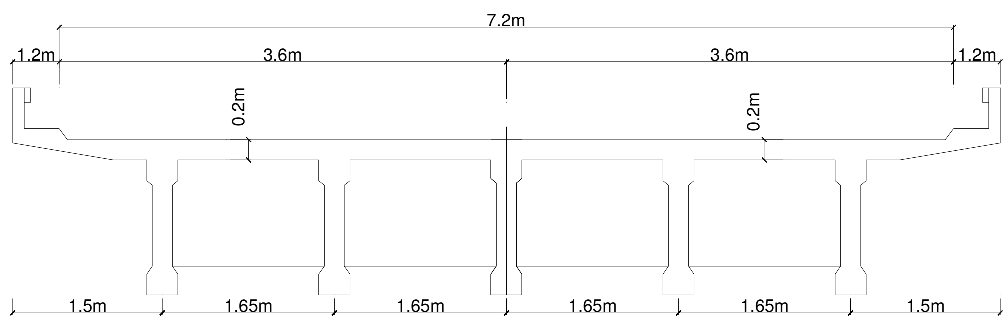
α = angle of inclination of transverse reinforced

Appendix

Short Span Pre-cast Prestressed Concrete Simply Supported Bridges

The following tables are prepared to give a guide in the design of pre-cast prestressed concrete short span bridges.

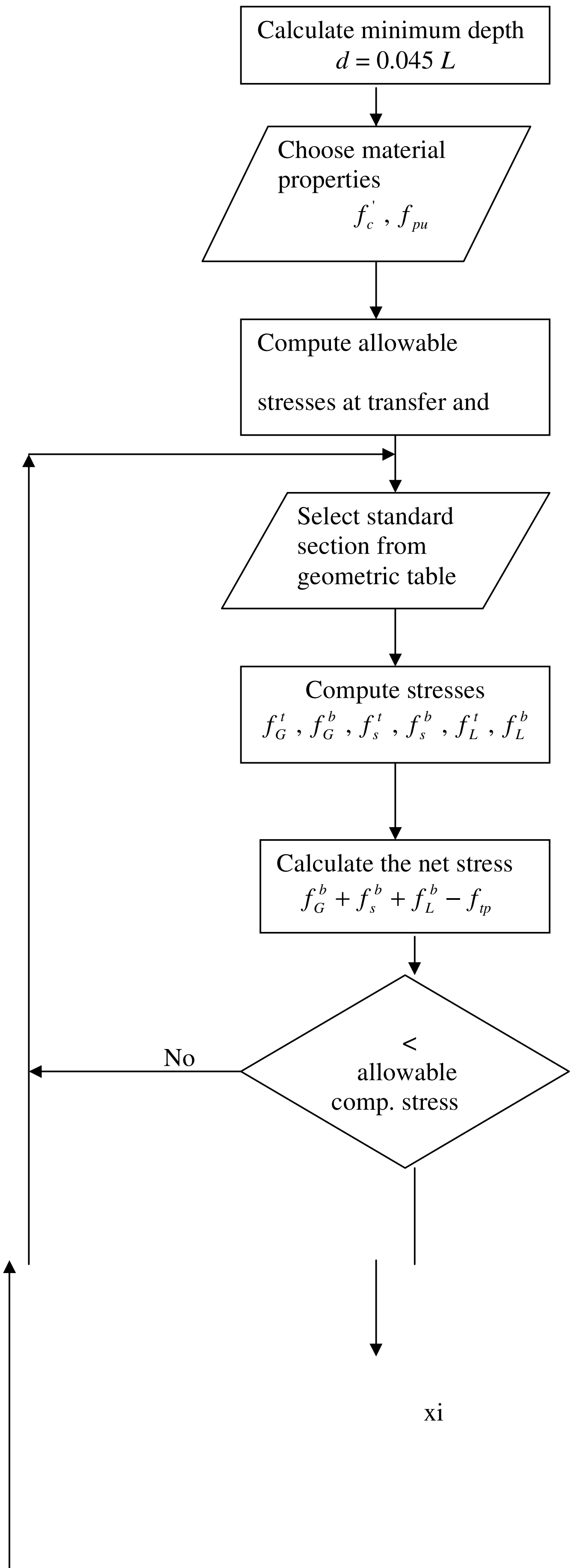
A typical common bridge carriageway width is adopted to establish the load distribution and strength of a single girder.

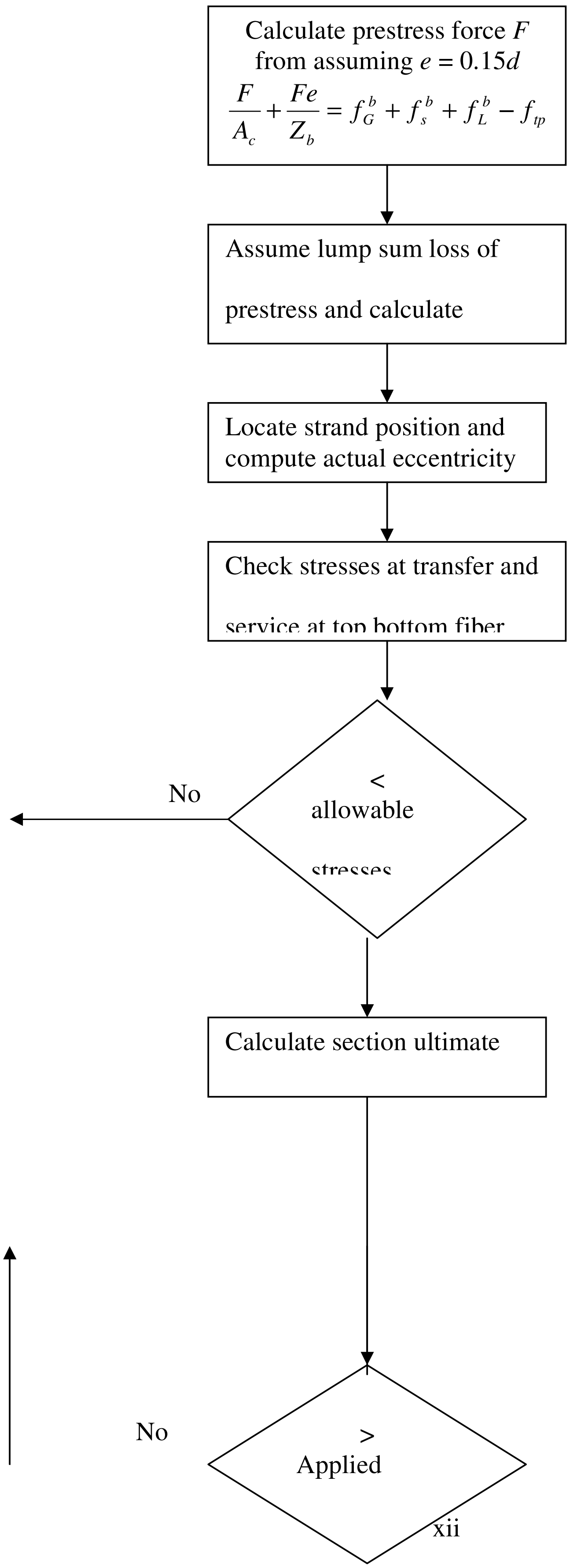


The standard I-girder sections are adopted from the Prestressed Concrete Institute of U.S.A (PCI). And the design is carried out based on

- AASHTO SI unit version 1998 2nd edition
- Concrete quality of 45 MPa minimum compressive strength at 28 days
- Prestressing steel quality of Grade 270 with tensile strength of1860 MPa
- Nominal diameter of 15.24 mm strand is used.

Flow chart
For the Design of Simply Supported Prestressed Concrete Girder Bridge




Calculate prestress force F from assuming $e = 0.15d$
$\frac{F}{A_c} + \frac{Fe}{Z_b} = f_G^b + f_s^b + f_L^b - f_{tp}$
Assume lump sum loss of
prestress and calculate
Locate strand position and compute actual eccentricity
Check stresses at transfer and
service at top bottom fiber
No
<
allowable
stresses
Calculate section ultimate
No
>
Applied

- Establish path of strands
- Design for shear
- Calculate deflection

Standard section

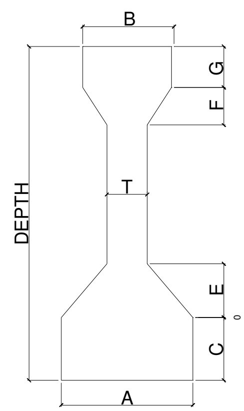


Table 1 Section Dimensions (m)

Type	Depth	A	B	C	E	F	G	T
II	0.91	0.46	0.3	0.16	0.16	0.08	0.16	0.16
III	1.14	0.055	0.40	0.18	0.19	0.12	0.18	0.18
IV	1.37	0.66	0.51	0.21	0.23	0.15	0.20	0.20

Table 2 Section Properties

Type	Depth (m)	Weight (kN/m)	Area (m^2)	I_c (m^4)	y_b (m)	Z_b (m^3)	Z_t (m^3)
II	0.91	5.72	0.238	0.0212	0.40	0.0528	0.0414
III	1.14	8.64	0.3598	0.05198	0.515	0.101	0.0832
IV	1.37	12.22	0.509	0.1085	0.63	0.173	0.146

Table 3 Recommended Section Type & No of Strands

Span (m)	**16**	**18**	**20**	**23**	**25**	**28**	**30**
Section Type	II	II	III	III	III	IV	IV
No of Strands	12	16	16	20	22	26	30

Abstract

The problems and achievements on the design and construction of prestressed concrete bridges in Ethiopia is assessed through distributing Questionnaires to be filled by professionals currently working on design and construction of bridges. A seminar is also organized to collect ideas from the participants and also to convey the basic concepts of prestressed concrete bridge design and construction.

The basic concepts, principles and advantages of prestressed concrete bridge design are discussed. The different construction methods of prestressed concrete bridges with the detail of anchorages are included.

Design example based on simple procedures is shown including its cost comparison with normally reinforced concrete bridge. Standard sections are prepared based on PCI (prestressed Concrete Institute) for different simply supported spans with design flow chart.

1. Introduction

1.1 General

Prestressing is the creation within a material of a state of stress and strain that will enable it to better perform its intended function.

About 1890, Henry Jackson, a San Francisco engineer, reportedly "invented" prestressed concrete. The world's first prestressed concrete bridge was built in Aue, Germany in 1937 using Dischinger's system. In spit of careful design and construction, the bridge was unsuccessful after 25 years, losing 75% of its initial prestress through creep and shrinkage.[10]

Modern development of prestressed concrete is credited to E. Freyssnet of France, who in 1928 started using high-strength steel wires for prestressing.[5]

High strength materials are essential to prestressed concrete for reasons of efficiency and economy of performance, but other properties are essential to the long-term stability and performance.

In prestressed concrete high tensile strength steel is used, which will have to be elongated a great deal before, its strength is fully utilized. If the high tensile steel is simply buried in the concrete as in ordinary concrete reinforcement, the surrounding concert will have to crack very seriously before the full strength of the steel is developed.

Prestressed concrete is not a different type of design rather it is an extension and modification of the applications of reinforced concrete to include steels of higher strength.

In most part of the world, prestressed concrete bridge construction is common practice.

The considerable interest that has been aroused by prestressed concrete would hardly be conceivable if this material did not offer substantial advantages. These advantages can be briefly summarized as follows: -

1. Greater durability because there are no cracks in the concrete, so that the embedded steel is well protected.
2. The full co-operation of the tensile zone of the concrete, saving in concrete is effected in comparison with ordinary reinforced concrete.
3. The self-weight of prestressed concrete beam is significantly lower than its equivalent reinforced concrete beam of the same carrying capacity.
4. The deformations of prestressed concrete structures are very small.
5. Prestressed concrete has a high resilience i.e. a very considerable capacity for recovering completely from the effect of a substantial degree of overloading, without suffering any serious harm. Cracks, which temporarily develop, will close up completely.
6. The fatigue strength of a prestressed concrete structure is a good deal higher than that of structures built from other materials, it is even higher than that of normal structural steel work.
7. The possibility of assembling blocks and separate elements and; in consequence, the possibility of the employment of prefabrication on a large scale.

1.2 Problems Identification

As it is mentioned in the above section, the prestressed concrete bridge construction system is widely used in most part of the world.

Whenever one system is dominant over the other there always exists an advantage in that system. To achieve that advantage in that system, it will not be an easy task in the beginning.

It is tried here to indicate the problems, which hindered the wide application of prestressed concrete bridges construction in the Ethiopia.

Questionnaires were distributed to main bodies that are involved in the design and construction of bridges in the country. The full content of the questionnaire and the summary of the answers are summarized as follows.

Questionnaire

This questionnaire is prepared to investigate the problems hindering the wide application of prestressed concrete bridge construction in Ethiopia. The responses of the questionnaire will be summarized and included in the thesis on “Prestressed Concrete Bridge Construction Practice and Prospect in Ethiopia”. The main objective of the thesis is to promote the widely application of prestressed bridge construction in the country.

1) How much do you know about prestressed concrete bridge design and construction?
2) Who do you think knows better about it?
3) How much is done in the design and construction of prestressed concrete bridge in the country to date?
4) Two major bridges namely the Abbay Bridge and the Awash Bridge are built by using this system. What was Ethiopian’s participation in the design and construction of these bridges?
5) What do you propose for the wide application of the prestressed concrete bridge design and construction in the country in the near future?

Professionals, who are currently working at the major organizations namely Ethiopian Roads Authority, TCDE, DANA Consult P.L.C., Blue Nile Construction, Berta Construction, and others responsible for the design and construction of bridges in the country, were selected to complete the questioner. Their responses may be summarized as follows: -

1) The majority of them did not do any design of prestressed concrete bridge, except some of them knew only the basic principles. Similarly, a few of them have the exposure and the general knowledge regarding the advantages of prestressed concrete construction method.

2) All did not clearly specified which organizations or individual groups have a potential knowledge of prestressed concrete bridge.

3) An appreciable work is not done in the design and construction of prestressed concrete bridge, except the construction of two bridges built and some new designs under proposal.

4) The participation of Ethiopians in the design of the two bridges built namely Awash and Abay is literally insignificant. In the construction, Blue Nile Construction Enterprise, a local contractor, has fully participated in the construction work of the Abay Bridge.

5) In respect to the future proposal, many of them agree that at least the following has to done: -

    - The merits and demerits of the prestressed concrete bridge construction have to be assessed.
    - Engineers have to be given more training.
    - The main organizations (e.g. The Ethiopian Roads Authority) responsible for the realization of bridge construction throughout the country have to take the major task in assessing the possibilities

and means for the application of the system. Such as by giving training to local labor force, preparing standards to give guide lines for design and construction,

- Continuing the effort started with respect to giving design and construction works in association with the local consultants and contractors in the sense of technology transfer in prestressed concrete systems.

A seminar on the subject matter was also organized on May 9, 2002 at the faculty of Technology inviting different organizations working on the design and construction bridges and the academic members.

During the seminar, highlights on the practice and prospects of the prestressed concrete bridge construction in Ethiopia were presented to the participants. Then discussion was held on the following main points: -

1. On the experience of the participants on the design and construction of the prestressed concrete bridges.
2. On the problems that hindered the wide application of prestressed concrete system in Ethiopia.
3. On measures that should be taken for the wide application of the prestressed concrete system.

After the discussion the participants pointed out the following points on the problems hindering the wide application of the system. The points were: -

- The prestressed concrete teaching is not fully covered in the under graduate Civil Engineering Carriculum.
- Ethiopian Roads Authority (ERA) does not give emphasis to the technology.
- Skilled manpower and equipments are not presently available locally.

In its concluding remarks the seminar forwarded the following main points.

- ERA should be open minded to accept this technology and take measures for its application.
- Local consultants should challenge ERA by submitting prestressed concrete bridge design, associated with the construction techniques.
- Higher institutions are expected to incorporate courses in prestressed concrete systems in their curriculum.
- Short-term training should be organized by universities to all civil engineers involved.
- Ethiopian Civil Engineers Association shall serve to disseminate the knowledge to its members, arranging like workshops, etc.
- Local contractors and consultants shall make efforts to learn and implement the technology from their foreign counterparts.

1.3 Scope of the Thesis

The objective of this thesis is to identify in the first place the problems (reasons) why prestressed concrete bridge construction system is not widely used, in spite of the fact that many normally reinforced concrete bridges were built through out the country in the past years.

It is tried in this thesis to summarize the basic concepts and principles in the design and construction of prestressed concrete bridges. The advantages of the system over the others are also demonstrated using examples.

The techniques and equipments involved in the construction of the two bridges built in the country using the prestressed concrete system with overview of their design are also included.

In general it is tried to give more information in the possibilities on the wide application of the prestressed bridge design and construction in the country and to entertain the advantages of the system.

It is believed that, once the application of this system is started in the country, it will open the gate for further research in the adaptation of new construction systems of the developed world to the local condition.

2. Past and Present Status of Prestressed Concrete Bridge Design and Construction in Ethiopia

2.1 Review of the Design and Construction of Prestressed Concrete Bridges Built

2.1.1 South Awash River Bridge

2.1.1.1 General

The bridge is constructed over Awash River on the road from Awash to Mille Towns at around 5.4 km from Awash Town.

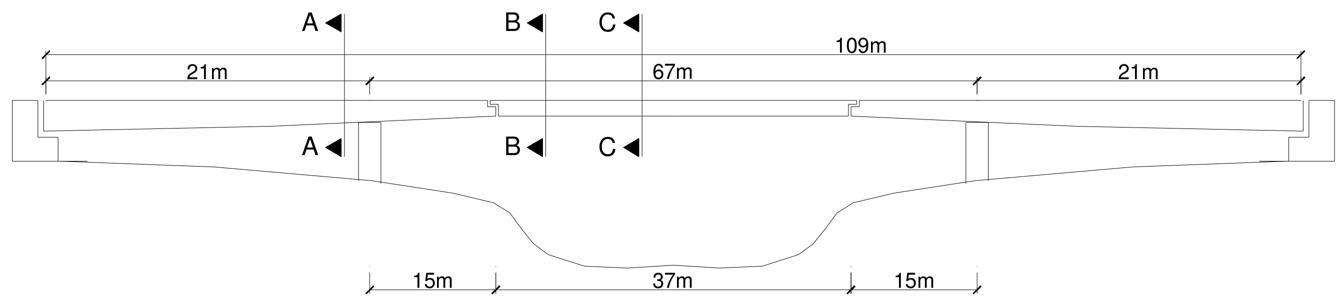
Foreign consulting firm Deleuw, Cather Int.S.A International Engineering Company, designed the bridge in March 1966 according to the title block of the drawings. [12]

The general arrangement of the bridge is composed of three spans consisting 67 meters central span and two 21 meters side spans. The total span of the bridge center to center of bearings is 109 meters.

The bridge is composed of two structural systems, the middle pre-cast girder simply supported and the single spans with cantilever over hang on either side.

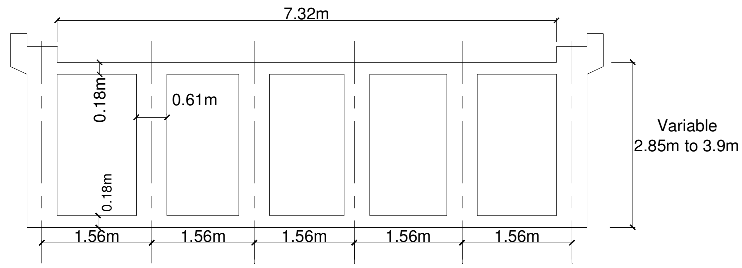
The portion with cantilever over hang has got a variable height that varies from a height 2.1meters at the cantilever end to 3.9 meters at the abutment end. The bridge cross-section in this part of the bridge has got slab with six girders in the cantilever over hang portion and six girders with top and bottom slab in the remaining portion. The middle portion of the bridge has got pre-cast T-girders with cast-in place slab.

The bridge has two lanes totaling 7.32 meters carriageway width and 0.8meters side walkway on both sides. (Refer to Fig 2.1)

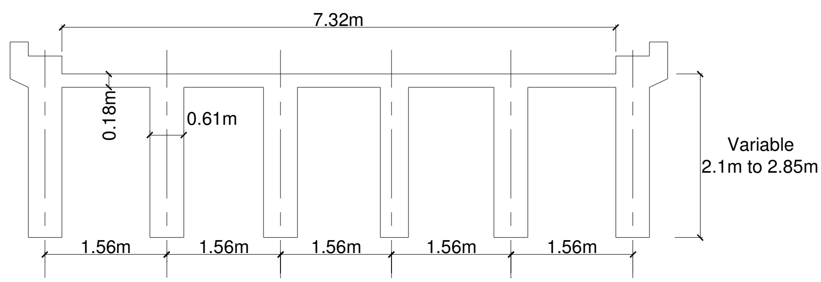


Longitudinal section

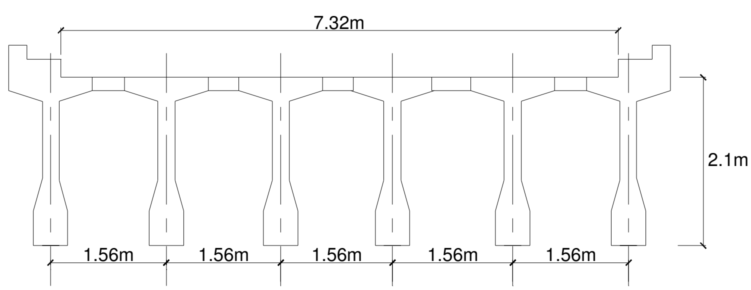
Fig. 2.1 Awash River Bridge



SECTION A-A



SECTION B-B



SECTION C-C

Fig. 2.1 (Continued)

2.1.1.2 Review of Awash Bridge Design

AAASHTO 1973 edition Article No

A) Design Data

1.2.5

- AASHTO Bridge design standard of the 1973 edition is used.
- HS20-44 standard truck loading, concrete quality of class-D, G-60 reinforcement steel bars and prestressing steel quality of proof strength 1655.8MPa were used according to the data obtained from the drawings.
- And the bridge design review is based on dimensions taken directly from the drawings.

B) Theoretical Background

- The analysis is carried out in two parts. In the first part the simply supported middle portion is considered. In the second part the end portion with the maximum reaction from the middle span is considered. (Refer to Fig.2.2)
- The stress at the critical sections are checked i.e. the maximum positive moment stresses at center of the middle portion and the maximum negative moment at the interior support of the end portion with cantilever overhang. (Refer to Fig.2.3)

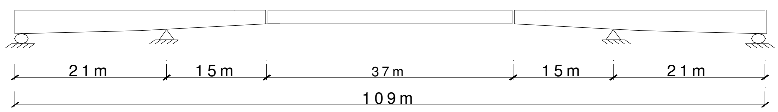


Fig.2.2 Structural Lay Out

2.1.1.2.1 Middle Portion

Dead load moment: - $w_{dl} = (Ac \times 25)\, KN/m$

$w_{dl} = 0.0943 \times 25 = 23.6\, KN/m$

$$M_{dl} = \frac{wL^2}{8}$$

$$M_{dl} = \frac{23.6 \times 37^2}{8} + \frac{0.144 \times 25}{2} \times \frac{37}{2}$$

$$= 4038.6 + 33.3$$

$$= 4072\, kN/m$$

Live load moment: -

- The maximum live load moment for 37m simply supported span comes from standard truck. 1.2.8

1.3.1

- Wheel load distribution

$$= \frac{S}{6} = \frac{1.56}{0.3048(6.0)} = 0.86$$

1.2.12

- Live load impact factor

$$I = \frac{50}{L+125} = \frac{50}{(37/0.3048 + 125)} = 0.203 < 0.3$$

- Live load moment

$$M_{LL} = 0.86(1276.7) \times 1.203$$

$$= 1321 kNm$$

Design calculations: -

Step 1. Calculate geometric and material properties. (Refer to Fig.2.3)

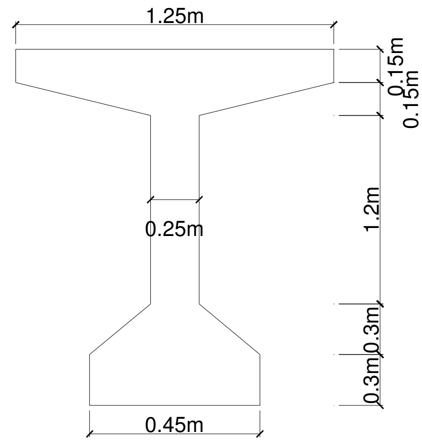


Fig.2.3 Girder Cross-Section

Geometric properties

$A_c = 0.84\,\mathrm{m}^2$

$I_c = 1.6648$ m^4

$Z_t = 1.877$ m^3

$Z_b = 1.213$ m

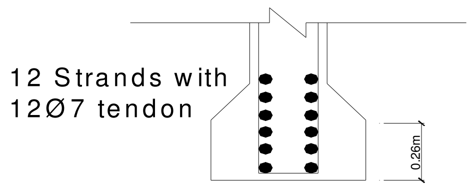
Material properties

- Concrete strength $f_c^{'} = 31$ MPa
- Prestressing strand $f_{pu} = 1655.8$ MPa

Step 2. Check stresses.

- Stresses at service condition after all losses.
- Prestressing tendons provided

1.2.8



- The top fiber stress due to service load assuming 15% total loss

$$f^t = \frac{M_T}{Z_t} = \frac{(4072+1321)}{1.877} = 2.87 \text{ MPa}$$

- The allowable compressive stress in concrete after all losses

1.5.6A&B

$$= 0.4 f_c^{'} = 0.4(4500) \times \frac{4.448}{(0.0254)^2} = 12.4 MPa$$

and the allowable tension in precompressed zone

$$= 6 \times \sqrt{f_c^{'}} = 6 \times \sqrt{4500} \times \frac{4.448}{(0.0254)^2} = -2.277 MPa$$

- The steel allowable stress after all losses

$$0.8 f_y = \frac{0.8 \times 765}{12 \times 38.5} = 1325 MPa$$

- The top fiber stress due to prestressing, considering 15 % loss

$$f_p^t = \frac{F}{A_c} - \frac{Fe}{Z_t} = 0.85\left(\frac{612 \times 12 \times 10^3}{0.84}\right) - 0.85\left(\frac{7.344 \times 10 \ \ \times (1.213 - 0.26)}{1.877}\right)$$

$$= 7.43 - 3.17 = 4.26 \text{ MPa}$$

- The net top fiber stress at service load

$$= 2.87 + 4.26 = 7.13 \text{ MPa} \quad < \quad 12.4 \text{ MPa} \qquad \text{OK!}$$

- The bottom fiber stress due to service load

$$f^b = \frac{M_T}{Z_b} = \frac{(4072 + 1321)}{1.373} = -3.93\, N/mm^2 \qquad \cong \quad \text{-2.277 MPa}$$

- The bottom fiber stress due to prestressing force

$$f_p^b = \frac{F}{A} + \frac{Fe}{Z_b}$$

$$= \frac{0.85 \times 7.344 \times 10^6}{0.84} + \frac{0.85 \times 7.344 \times 10^6 \times (1.213 - 0.26)}{1.373}$$

$$= 7.34 + 4.34 = 11.77 \text{ MPa}$$

- The net stress at bottom fiber due to service loading

$$= -3.393 + 11.7 = 7.84 \text{ MPa} \qquad \text{OK !}$$

2.1.1.2.2 Cantilever Portion

- Live load moment

$$M_{LL} = 0.86 \times 4500 \times 1.203$$

$$= 4655.61 \text{ kNm}$$

- Dead load moment

$= 5850$ kNm from self weight

$= 6549$ kNm from central span

$$M_{DL} = 12399 \text{ kNm}$$

<u>Design calculations</u>

Step1. Geometric properties

$$A_c = 2.0782 \text{ m}^2$$

$$I_c = 2.172 \text{ m}^4$$

$$Z_b = 1.95 \text{ m}^3$$

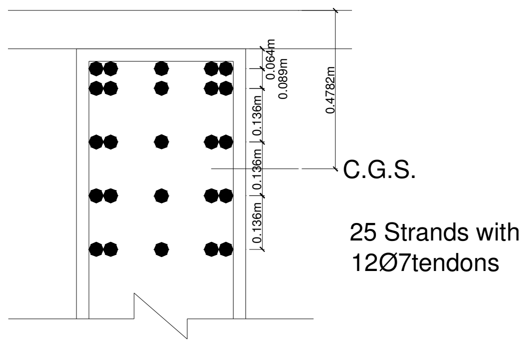
$$Z_t = 1.29 \text{ m}^3$$

$$y_b = 1.682 \text{ m}$$

Step2. Check stress

- Stress at service condition after all loss

- The prestressing tendons provided



- The prestressing force, $F = 25\times462\times1325 = 15304$ kN

- The top fiber stress due to service loads

$$f_t = \frac{M_T}{Z_t} = \frac{(12399+4656)}{1.29}\times10^3 = -13.22 \text{ MPa}$$

- The top fiber stress due to prestressing, assuming 15% loss

$$f_p^t = \frac{F}{A} + \frac{Fe}{Z_t}$$

$$= \frac{0.85\times15304\times10^3}{2.782} + \frac{0.85\times15304\times10^3}{1.29}$$

$$= 15.74 \text{ MPa}$$

- The net top fiber stress

$$-13.22 + 15.74 = 2.52 \text{ MPA} \quad > 0 \quad \text{OK!}$$

- The bottom fiber stress due to service loads

$$f^b = \frac{M_T}{Z_b} = \frac{17055\times10^3}{1.95} = 8.75 \text{ MPa}$$

- The bottom fiber stress due to prestressing

$$f_p^b = \frac{F}{A_c} - \frac{Fe}{Z_b} = \frac{130084 \times 10^3}{2.0782} - \frac{130084 \times 10^3 \times 0.9398}{1.95}$$

$$= -0.099 \text{ MPa}$$

- The net bottom stress $= 8.75 - 0.099 = 8.651$ MPa < 12.4 MPa

  OK!

2.1.1.3 Overview of the Construction

A very important factor to be considered in the design of bridge structures is the system of construction that could give the most economical and suitable solution depending on the site conditions.

From this point of view the designer of the Awash Bridge may considered the site conditions and used a very suitable and economical construction solution.

Over the main course of the river perhaps due to diversion problem and height of bridge from river bed level, a 37meter simply supported pre–cast I-girders were provided at middle portion. The prestressed pre-cast I girders gave them the chance of lifting and transporting the beams easily because of lesser dead weight due to prestressed sections. Otherwise 37 meters span normally reinforced concrete pre-cast beams with tremendous dead weight might be impossible to transport and lift.

Over the river course with the relatively shallow depth, they provided cast- in-situ box section with variable depth decreasing towards the intermediate piers and rectangular girders over hanging about a span of 15 meters beyond the piers might be with the intension of reducing the middle pre-cast girder span length. This 15 meters cantilever portion may not be possible in normally reinforced concrete.

The middle portion with pre-cast I-girders that were post-tensioned at casting yard and transported and placed in position after the two end portions of the bridge were cast-in place on false works that were positioned on the safe and shallow bank of the river course. The cast-in place portion were post tensioned after the pre-cast girders were placed in position. The final work was to connect the pre-cast girders together with thin concrete slab.

2.1.2 Abay River Bridge

2.1.2.1 General

This bridge is constructed over Abay River on the route from Bure to Nekemte Towns. A foreign consultant BCEOM of France in 1983 GC designed the bridge.

The main part of the bridge is composed of a three continuous spans, 85meter central spans and 52meter end spans totaling 189meter overall length.

The bridge has got two lanes with sidewalk ways. The cross-section of the bridge is single cell box with flares at each corner.

The bridge is located in the deep gorge of Abay River, which in turn requires the bridge to be part of curved approach on both directions, and to be raised 45 m above the average ground level. (Refer to Fig.2.4)

2.1.2.2 Overview of the Design

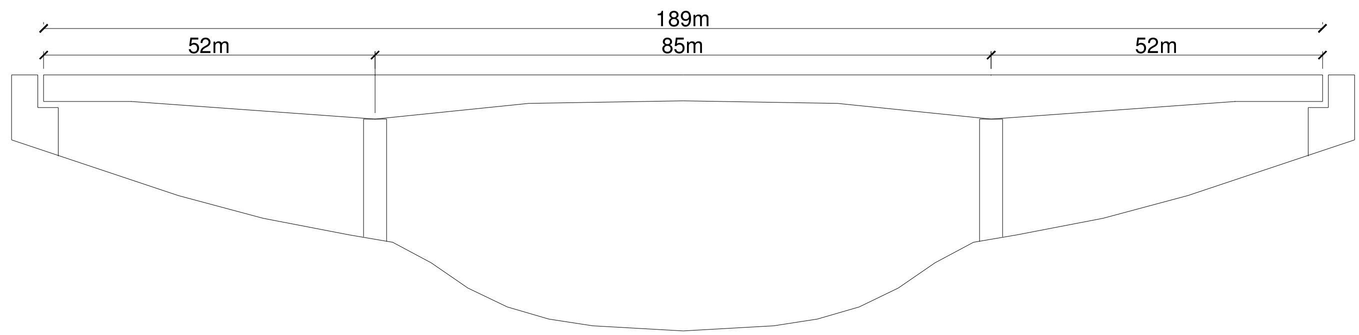
Theoretical Background

The design of such type of bridge involves a number of loading at different construction stages. It is tried here only to show briefly the material characteristics, structural geometry, loading and design considerations.

Structural geometry (Ref. to Fig.2.4)

- The main bridge part is composed of a continuous three span with variable height increasing towards the pier supports,

- The depth varies from 2.5 meters at the free end to 5.3 meters at the interior supports and back to 2.5 meters at center of the interior span.
- The bridge cross-section is single cell box with flare at web to slab connections.
-



Longitudinal Section

Fig.2.4 Abay River Bridge

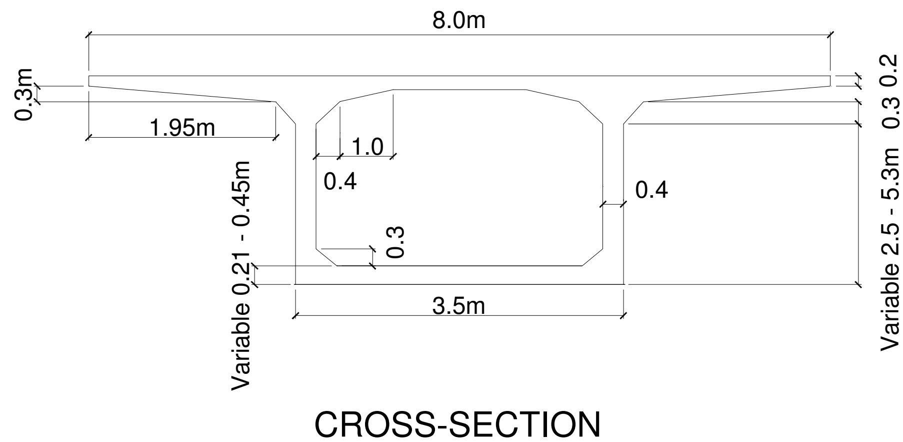


Fig. 2.4 (Continued)

- The bottom slab thickness varies increasing from 0.21meters at free end to 0.45 meters towards the pier support. (Refer to Fig.2.4)

Material Strength

- Concrete quality of 35 MPa minimum compressive strength at 28 days
- Ordinary reinforcement steel quality of G-60 and prestressing steel of G-270

Structural Loading

- The bridge loading as specified on the general note is AASHTO H20-S-16-44
- Dead load of 25 KN/m^3 of concrete which is variable due to height and bottom slab thickness.
- Two lane live load loading
- The analysis of such structure should be carried out with step-by-step consideration of the different loading conditions involved in each construction stage.
- The structural behavior at each stage shall also be considered including material strength developed at the different segments that are casted at different times.
- The loading condition during construction must also consider: -

- Any suspended shuttering, traveler's plat-from and internal scaffoldings.
- Wind loads
- Impact loads from concert buckets
- The work force and relevant machine weights
- After the whole scenario of the constructing is completed, the integrity of the whole components should be checked and analyses must be done for critical line loading for the integrated continuous structure.

2.1.2.3 Overview of the Construction

As mentioned above the construction method applied is the balanced cantilever method, which may be one of the modern ways of long span bridge construction in the world.

Blue Nile Construction Company carried the construction under the supervision of foreign consultants, in duration of two & half years. The construction was completed in 1991 with a little interruption due to government change.

The construction of the superstructure was started from the two main piers by casting concrete, segment-by-segment in both directions away from the pier center. The shuttering for each segment was erected on traveler platform suspended on cables hanging over a truss support, which could roll on rails tied up on the already casted segment. (Ref. to Fig.2.6)

Once the concrete on either of the pier side was casted, it was left for a week time until the concrete gain strength. The post-tensioning was done from both directions to hold the segments together. To avoid any risk of any unbalanced loading the starting middle segment was tied up with the pier on either side of the centerline of the pier. On doing this temporary concrete blocks were used to support the segments in balance.

After the whole segment casting was completed from the center of the two main piers, the remaining portion of the bridge on the two ends of the superstructure were casted on scaffolding from the ground.

Before joining the end segments and the central span middle segment, the balanced cantilevered part was jacked up using 500 tons capacity jacks from the temporary concrete block supports. This was really a fantastic job that requires great care not to loss the balance.

After completion of the whole segment casting, the continuity of the whole bridge system was done by post-tensioning for positive bending and strengthening for the negative bending.

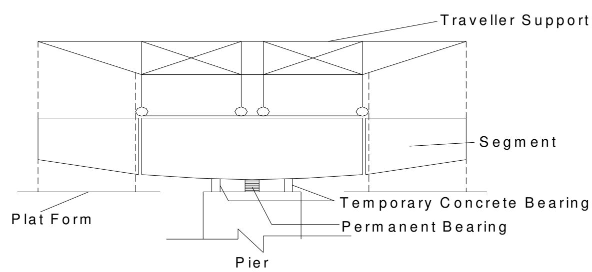


Fig. 2.6 Method of Balanced Cantilever Construction on Abay Bridge

2.2 Present Status of the Design and Construction of Prestressed Concrete Bridges in Ethiopia

The country's present status on prestressed concrete bridge construction is only at the design proposal stage. The design proposals are summarized as follows.

A similar type of bridge as that of the Abbay Bridge on Bure to Nekmete rout is again proposed by foreign consultant over Abay River on the rout from Combolcha to Gendeweyen of Amhara Regional state.

A final design of 20 meters and 30 meters pre-cast prestressed simply supported girders, which could be used repeatedly for twenty seven bridge locations in the Arbaminch Jincka Road Upgrading Project is submitted by a foreign consultant working with local ones.

In the phase III portion of the Addis Ababa Ring Road Project, 30 meters continuous span prestressed concrete box girder viaducts are also proposed.

A prestressed concrete bridge over Akaki River on the old route was also proposed in a design proposal.

3. Design Consideration

3.1 Basic concepts

Prestressing is the imposition of a state of stress on a structural body, prior to its being placed in service, that will enable it to better withstand forces and loads imposed on it in service or to better perform its design functions.

There are two normally used systems of prestressing.

1) Pretensioning is the imposition of prestress by stressing the tendons against external reactions before the hardening of the fresh concrete, then allowing the concrete to set and gain a substantial portion of its ultimate strength, then releasing the tendons so that the stress is transferred into the concrete. Pretensioning is most commonly applied to pre-cast concrete elements manufactured in a factory or plant.
2) Post-tensioning is the imposition of prestressing and anchoring tendons against already hardened concrete. Post-tensioning is most commonly applied to cast-in-place concrete members and to those requiring a curved profile of the prestressing force.

3.2 Basis of Design and Principles

The design of prestressed concrete members is generally governed by limits on tensile and compressive stress in service rather than by their strength at the ultimate limit state.

Normal practice, therefore, is first to produce an initial design, that is to choose an appropriate prestress force and tendon location, which satisfies the limitation on service stresses. This design is then checked at the ultimate limit state to insure that it satisfies strength requirements.

Owing to the techniques employed in the construction of prestressed concrete members, two critical loading conditions arise where the stresses in the concrete must be checked against specified values.

The first condition, known as the transfer condition, occurs immediately on transfer of the prestress force to the concrete. At this stage, the concrete is still relatively young and its compressive strength has not reached its full design value. The stresses, which are acting on a member during the transfer condition, are prestress and stress due to the moment, M_o, induced by the applied loads that are present at transfer.

The second condition, which must be checked, is the service condition. This condition is reached when the concrete has matured to its full strength and the full service loads are being applied to the member. At this stage, the applied prestress force has been reduced from its initial magnitude, P, due to losses, which have taken place in the concrete and the steel. The total loss of prestress force between transfer and service is generally in the region of 10-20 %. [10]

For the design of fully prestressed members, in which no cracking of the members is permitted, the tensile stress may be limited to the tensile strength of concrete. In the case of partially prestressed members, in which cracks are allowed to form at service condition. Instead, limits are placed on the width to which cracks are allowed to open. For this reason it has been found that it is often easier initially to design partially prestressed members for strength at the ultimate limit state and then to check that the transfer and service conditions are satisfied.

Usually tendon is located at an eccentric point from the centroid of a member. The application of this eccentric force is equivalent to applying a concentric axial force, P, and a sag bending moment, M_p, which is the product of the prestress force and eccentricity, e (that is $M_p=Pe$) as shown in Fig. 3.1.

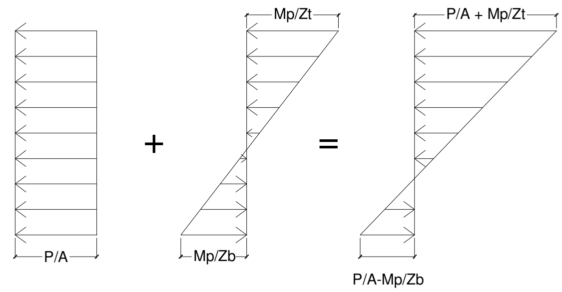


Fig.3.1 Stress Distribution due to Eccentric Prestress.

The total stress due to prestress at the top fiber is

$$f^t = \frac{P}{A_c} + \frac{M_p}{Z_t} \qquad [3.1]$$

and the bottom fiber is

$$f^b = \frac{P}{A_c} - \frac{M_p}{Z_b} \qquad [3.2]$$

In the design of prestressed concrete members, it is often necessary to choose an initial trial section in order to estimate the load due to self-weight. The more accurate this initial section size is, the fewer iterations of the design process will be required later. In such cases, a reliable preliminary section can often be determined by a consideration of the stress requirements at transfer and service. Expressions are derived in terms of the minimum values for the section moduli, Z_t and Z_b, which are required to satisfy the stress limits at critical sections. From these minimum section moduli, the minimum dimension, of the member can readily be determined from : -

$$Z_t \geq \frac{M_s - \rho M_o}{P_{s\max} - \rho P_{o\min}}$$

and [3.3]

$$Z_t \geq \frac{\rho M_o - M_s}{\rho P_{o\max} - P_{s\min}}$$

Where P_{omin} = minimum permissible stress at transfer

P_{omax} = maximum permissible stress at transfer

P_{smin} = minimum permissible stress at service

P_{smax} = maximum permissible stress at service

ρ = ratio of prestress force at service to prestress force at transfer (typically $\rho = 0.75$ to 0.9)

M_s = total applied moment at service

M_o = total applied moment at transfer

The above two inequalities represent the two lower limits of the elastic section modulus. The greater of the two values for Z_t from the two expressions should be used to determine an initial section size.

$$|Z_b| \geq \frac{\rho M_o - M_s}{P_{s\max} - \rho P_{o\min}}$$

and [3.4]

$$|Z_b| \geq \frac{M_s - \rho M_o}{\rho P_{o\max} - P_{s\min}}$$

The above two inequalities represent the minimum required absolute value for the elastic section modulus for the extreme bottom fiber. As for Z_t, the greater value from the two expressions should be used to determine an initial section size.

The sign convention used

a) Applied sag moments are considered positive and applied hog moments are negative.

b) The eccentricity, e, of prestressing tendons above the centroid of the member is positive and below the centroid of the member is negative.

c) Compressive stresses are considered positive and tensile stresses are considered negative.

d) The distance from the centroid is positive upwards. Hence, the numerical values for $Z_b(=I/y_b)$ is always negative.

Once a cross-section has been chosen for a prestressed member, which satisfies inequalities given above, the next step is to determine how much prestress force to apply, and where to locate, at each section along the length of the member .The most appropriate value of P and e can be chosen for each critical section with relative ease to fully prestressed members or partially prestressed members where stress limits are specified in place of allowable crack width.

$$\left(\frac{1}{P}\right)f_{t\min} \leq \frac{1}{A}+\frac{e}{Z_t}$$

$$\frac{1}{A_c}+\frac{e}{Z_t} \leq \left(\frac{1}{P}\right)f_{t\max}$$

$$\left(\frac{1}{P}\right)f_{b\min} \leq \frac{1}{A_c}+\frac{e}{Z_b}$$

$$\frac{1}{A_c}+\frac{e}{Z_b} \leq \left(\frac{1}{P}\right)f_{b\max} \qquad [3.5]$$

Each of the inequalities represents a linear relationship between e and $1/P$ that can be illustrated on a plot of e versus $1/P$. This form of graphical representation of the limits on the prestress force and eccentricity is attributed to Magnel and hence it is commonly known as a Magnel diagram [10]. The point where the lines cross the e-axis can be found by setting $1/P$

equal to zero in each inquality. Similarly, the point where each line crosses the *1/P* axis is found by setting *e* equal to zero in each inequality, knowing these two intersection points for each line, the lines can readily be drawn on the plot. The half-plan, which represents the inequality, is the one containing the origin. The zone in which all the stress limits are satisfied and is called the feasible zone. Any combination of y_p and *e* that falls within the feasible zone constitutes a valid solution. Usually, to make saving on prestressing steel, a solution is chosen corresponding to a low value of *P* but sufficiently away from the edge of the feasible zone to allow some latitude in tendon eccentricity. It is often helpful to draw in two further boundary lines on the Magnel diagram, which represents the maximum eccentricities, which are physically possible.

In the above

$$f_{t\min} = \text{greater of } \left(P_{o\min} - {}^{M_o}\!/_{Z_t}\right) \text{ and } {}^{1}\!/_{P}\left(P_{s\min} - {}^{M_s}\!/_{Z_t}\right)$$

$$f_{t\min} = \text{lesser of } \left(P_{o\max} - {}^{M_o}\!/_{Z_t}\right) \text{ and } {}^{1}\!/_{P}\left(P_{s\max} - {}^{M_s}\!/_{Z_t}\right)$$

$$f_{b\min} = \text{greater of } \left(P_{o\min} - {}^{M_o}\!/_{Z_b}\right) \text{ and } {}^{1}\!/_{P}\left(P_{s\min} - {}^{M_s}\!/_{Z_b}\right)$$

$$f_{b\max} = \text{lesser of } \left(P_{o\max} - {}^{M_o}\!/_{Z_b}\right) \text{ and } {}^{1}\!/_{P}\left(P_{s\max} - {}^{M_s}\!/_{Z_b}\right) \qquad [3.6]$$

Where f_{tmin} = allowable minimum stress at top fiber

f_{tmax} = allowable maximum stress at top fiber

f_{bmin} = allowable minimum stress at bottom fiber

f_{bmax} = allowable maximum stress at bottom fiber

In case of typical post-tensioned beams, the critical bending moments will vary from one section to the next within each span. In addition, many post-tensioned beams have a variable depth over their length. When one tendon is to be used throughout the length of the beam, the force, P, will be the same for each section except for small differences due losses. For such members, an appropriate prestress force and tendon profile, which satisfies the prestress requirement at all sections, can be sought by observing the following three-steps.

1. Choose a prestress force, P, which is within the feasible zone of each Magnel diagram. If this not possible, the section size of the member must be revised so that it is possible to find an appropriate value for P.
2. For the chosen prestress force, calculate the two limits e_{min} and e_{max} on the tendon eccentricity for each Magnel diagram. Plot the locations of these eccentricity limits on a longitudinal section through the beam. Lines are then drawn to join the limits for adjacent sections. These tendon limits constitute a longitudinal feasible zone within which the tendon must tie in order to satisfy the stress limits at each section.
3. A tendon profile is such that its centroid lies within the longitudinal feasible zone.

The total number of sections along the length of a member, which are considered, is a matter for engineering judgment. In general, only a few sections (at points of maximum and minimum bending moment and one or two points in between) need to be considered in order to find an acceptable tendon profile.

3.2.1 Losses in Prestress Force

Owing to losses of force that occur in pressing strands and tendons, the effective prestress force, P, which is transferred to the concrete is not generally equal to the applied jacking force, P_{jack}, nor is it constant along the length of the member. These losses could be expressed as follows: -

- Pre-transfer loss of force at section $i = P_{jack} - P_i$
- Post-transfer loss of force at section $i = P_i - \rho_i P_i$

Where $\rho_i P$ is the prestress force at section i reduced to final value after all loses has occurred.

Pre-transfer losses result from elastic shortening of the concrete and, in the case of post-tensioned members, from friction between the tendons and the surrounding ducts. Post-transfer losses are caused by relaxation of the steel and by creep and shrinkage of the concrete. In addition, in the case of post-tensioned members, there is a post-transfer loss due to slippage of the tendons at the anchorage known as draw-in loss.

1. Friction losses

The loss of prestress force due to friction arises only in post-tensioned members, caused by two sources of friction.

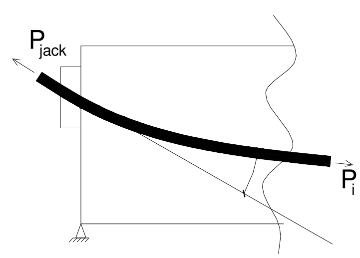
a) Friction due to curvature of the tendon.

b) Friction due to unintentional variation of the duct from its prescribed profile or 'wobble'.

The loss between the jack and the i^{th} section is an exponential function of the change in angle, θ between the two points.[10]

The reduced force, P_i, at section i relative to the jacking force, P_{jack}, is given by:

$$P_i = P_{jack} e^{-\mu\theta} \qquad [3.8]$$



Where μ is a friction coefficient, which depends on the surface roughness of the duct and θ is the aggregate change in slope in radians between the jack and section i. If the tendon slopes first upwards and then downwards the corresponding changes in angle are additive. In the absence of more exact data from manufacturers, that μ for strands can be taken as 0.19.

The magnitude of the wobble loss at a section depends not on the curvature of the tendon but on the distance of the section from the jacking

$$P_i = P_{jack} e^{-\mu K x} \qquad [3.9]$$

where K is a wobble coefficient, which depends on the quality of workman ship, the distance between tendons supports, the degree of vibration used in placing the concrete and the type of duct. And x is the distance from jack the section i. The value for K is generally in a range 0.005 to 0.01.

Therefore the total friction loss in prestress force at section i is equal to

$$\text{Total friction loss} = P_{jack}\left(1 - e^{-\mu(\theta + Kx)}\right) \qquad [3.10]$$

2. Elastic Shortening Losses

As the prestress is transferred to the concrete, the concrete undergoes elastic shortening, which reduces the length of the member. This causes a slackening of the strand, which results in a loss of prestress force.

The loss of prestress force due to elastic shortening in pretensioned members is elastic shortening loss of force given by [10] : -

$$= P_{jack} A_p \frac{E_p}{E_c}\left(\frac{1}{A_g} + \frac{e^2}{I_g}\right) + A_p E_p \left(\frac{M_o e}{E_c I_g}\right) \qquad [3.11]$$

Where E_p is the elastic modulus of the strands and A_p is the total cross-sectional area of the strands.

For the general case of a post-tensioned member with many tendons, the total loss is normally less then the elastic shortening loss, which occurs in pre-tensioned member.

3. *Draw-in losses*

After jacking the post-tensioned member to the required force, P_{jack}, the tendons are released from the jacks and the force is applied through the anchorages to the concrete. When wedge anchors are used, this process results in a loss of prestress due to slippage or 'draw-in' of the strands at the anchors. Fortunately, the friction between the tendons and the ducts ensures that this loss of prestress does not extend very far from the region of the anchor in most cases, particularly in longer members.

$$\Delta P = 2\frac{\left(P_{jack} - P_L\right)}{L}L_d \qquad [3.12]$$

Where L_d = distance from the jack point and is given by

$$= \sqrt{\frac{(\Delta_s E_p A_p)}{\left((P_{jack} - P_2)/L\right)}}$$

Δ_s = draw –in loss total shorting in mm

L = length of the member

4. Time-dependent losses

Losses in prestress, which occur gradually over time, are caused by shortening of the concrete due to creep and shrinkage and due to relaxation of the prestressing steel. Shrinkage is a time-dependent strain, which occurs as the concrete sets and for a period after setting. The shrinkage strain approaches a final value, ε_{sh}, at infinite time and could be found in tables. The loss of prestress force is generally constant over the entire length of the member and can be estimated as:

$$P_{sh} = \varepsilon_{sh} E_p A_p \qquad [3.13]$$

The phenomenon of creep is essentially the same as that of relaxation. Creep is usually measured in terms of creep coefficient, ω, which is the ratio of creep strain to linear elastic strain:-

$$\varphi = \frac{\varepsilon_\infty}{f_c / E_c} \qquad [3.14]$$

Where ε_∞ is the final creep strain and f_c is the elastic stress in the concrete. The magnitude of the creep coefficient depends on the size of the member, the relative humidity and on the age of the concrete when it is loaded. Values of final creep coefficient for the calculation of the total creep strain in prestressed members could be found in tables.

Therefore, the loss in prestress force due to creep is given by [10] : -

$$\text{Creep loss} = A_p E_p \varepsilon_{:} = A_p (E_p/E_c)\ \omega f_c \qquad [3.15]$$

A more conservative estimate of the creep loss can be calculated on the assumption that f_c remains constant at its maximum value.

$$f_c = \frac{P}{A_g} + \frac{(Pe + M_{perm})}{I_g} e \qquad [3.16]$$

Where P is the prestress force in the tendons after pre-transfer losses and M_{perm} is the moment due to permanent loads. This method is conservative and does not require iteration.

Rather than calculate each time-dependent loss separately some codes recommends that the losses be calculated all together using a rather expansive formula.[10]

$$\Delta f_{p,tot} = \frac{\varepsilon_{sh} E_p + \Delta f_{p,r} + m\varphi f_c}{1 + \frac{mA_p}{A_g}\left(1 + \frac{A_g e2}{I_g}\right)(1 + 0.8\varphi)} \qquad [3.17]$$

where $\Delta f_{p,tot}$ = loss of stress due to creep, shrinkage and relaxation

$f_{p,r}$ = loss of stress in the tendons at the design section due to relaxation

$m = E_p/E_c$

When maintained at a constant tensile strain, steel gradually losses its stress with time due to relaxation. The loss of prestress, which is caused by steel relaxation, can range from 3% to12% of the initial force. The precise values for relaxation losses are usually specified by the manufactures of prestressing steel.

The loss of stress in the tendons due to relaxation of the tendons, $\Delta f_{p,r}$ with the ratio of steel stress to the characteristic tensile strength(f_p/f_{pk})

$$f_p = f_{p_i} = \frac{P}{A_p} + \left(\frac{E_p}{E_c}\right)\frac{M_{perm}e}{I_g} \quad [3.18]$$

3.2.2 Secondary Effects of Prestress

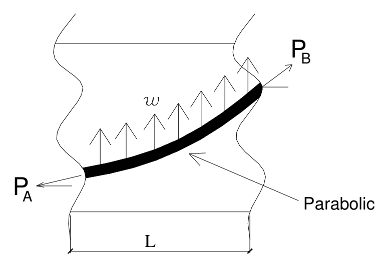
Secondary moments arise from reactions developed at supports during prestressing, which only apply to indeterminate structures. The magnitude of these secondary moments in a given member depends on a number of factors including tendon curvature and geometry changes in non-prismatic members.

For the determination of the prestress moment, M_p, in an indeterminate member, the equivalent load method can be used. In this method, the forces applied to the concrete by the prestress are represented as externally applied loads. The structure is then analyzed to obtain the total (Primary Plus Secondary) moments due to these loads. Once these moments are known, the secondary moments at a section due to prestress can readily be calculated if required by subtracting the primary moment, *Pe*, from the total moment. An added advantage of the equivalent load method is that, once loads have been evaluated, they can also be used to calculate deflections due to prestress.

Calculation of Equivalent Loads

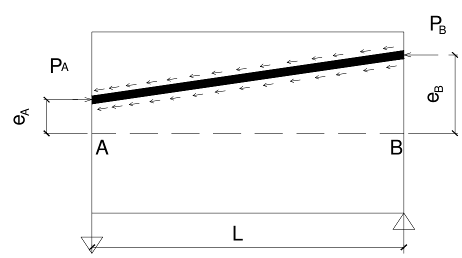
There are essentially four 'sources' of equivalent loading due to prestress in continuous members. These are tendon curvature, friction loss, end forces and moments, and equivalent loading due to geometry changes in beams of variable section. These are illustrated as follows : -

1) Tendon Curvature

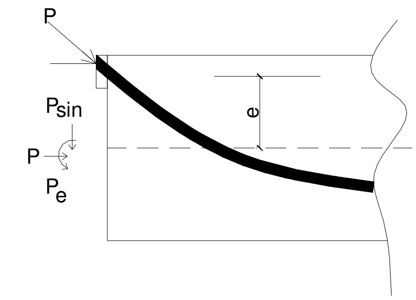


$$\omega = \frac{P_A + P_B}{2\delta L}\left(\sin\theta_B - \sin\theta_A\right) \quad [3.19]$$

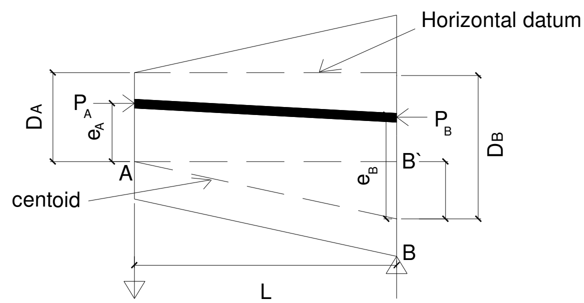
2) Friction Losses



3) End Forces and Moments



4) Geometry Change



3.2.3 Ultimate Moment Capacity of Prestressed Concrete

Once a prestressed member has been designed to satisfy the transfer and service stress limits, it must be checked for other serviceability limit states and the ultimate limit states.

As for ordinary reinforced concrete members, the ultimate moment capacity of a prestresed section is most readily calculated from equilibrium of forces acting at the section. Failure at the section under ultimate loads occurs when the strain in the extreme fiber in compression reaches its ultimate strain value, ε_{ult}. The compressive strain in the concrete due to prestress, ε_{ce}, at the level of the prestressing tendon is given by equation 3.20. [10]

$$\varepsilon_{ce} = \frac{1}{E_c}\left(\frac{\rho P}{A_g} + \frac{\rho M_p e}{I_g}\right) \qquad [3.20]$$

In an indeterminate structure, secondary effects will, in general, change M_p from Pe. However, plastic hinges may have formed in which case the structure could once again be treated as determinate. The strain in the tendon due to the prestress alone at ultimate limit state is (contraction positive): -

$$\text{Tendon strain due to prestress} = \frac{-\rho P}{A_p E_p} \qquad [3.21]$$

The tendon strain, ε_{pu}, is given by

$$\varepsilon_{pu} = \frac{-\rho P}{A_p \varepsilon_p} + \varepsilon_{ct} - \varepsilon_{ce} \qquad [3.22]$$

and the total ultimate strain, ε_{ct} is given by

$$\varepsilon_{ct} = \frac{-\varepsilon_{ult}(d-x)}{x} \qquad [3.23]$$

where x is distance to neutral axis and d is depth to tendon location.

4. Construction of Prestressed Concrete Bridge and Its Economic Consideration

4.1 Methods of Construction

4.1.1. Prestressing Systems

Basically as mentioned earlier there are two methods of prestressing concrete based on when the prestressing wires are stressed.

4.1.1.1 Pretensioning

Tendons are stressed and anchored against external abutments. The concrete is poured and cured so as to have substantial strength in compression and bond. The tendons are then released from the abutments and the "prestress" is transferred into the concrete member. Prestress can be transfered only by elastic shortening of the concrete.

In manufacture by the pretensioned method, the frictional weight of the concrete member, or binding in forms, may prevent longitudinal shortening until the member is lifted. During this period, shrinkage or thermal cracking may occur, unresisted by prestress, since the member has not yet shortened. Similarly, in picking from the forms, the initial lifting force may cause cracks, particularly at the point of pick, since the member has not yet shortened and become prestressed.

Pretensioning requires adequate transfer length to transfer the stress from the tendon to the concrete. Especially short transfer length is desirable when high moment must be resisted a short distance from the end; for example, in cantilevered beams, for which, special efforts must be made to obtain the shortest possible transfer length. Such steps include: -

1. Through consolidation of concrete at ends.
2. Higher compressive strength of concrete at release.

3. Reduced stress level in tendons obtained by increasing steel cross-sectional area.
4. Increased surface area of tendons obtained by using smaller wires or strands.
5. Gradual release, that is, by hydraulic de-jacking.
6. Adequate lateral binding, by spiral or stirrups, at ends.
7. Use of indented strands or wires.
8. Cleaning of the lubricant from the strand used in drawing.

Pretensioned tendons, by the nature of their being stressed before concreting, must be straight between points of support. On the other hand, for many girders the specified tendon profile is a parabolic curve.

To achieve an approximation of such a curve, the most accurate production method is that in which the strands are stressed as straight strands to a pre-computed value, then pulled down (or up) to match the desired parabolic chord profile. Then the longitudinal pretension is checked and adjusted to the design pretensioning value.

Deflection of strand is a critical construction procedure. Should a hold-down fail, the stored energy in the strands is like a bowstring and the hold-down becomes a missile. Positive measures must be taken so that these works can be done from the side.

4.1.1.2 Post-tensioning

Post-tensioning is the system of prestressing by which the tendons are tensioned against the concert after it has hardened and reached a substantial portion of its design strength. The tendons are typically inserted through the concrete element, through holes which have been formed by ducts. Then the tendons are stressed by hydraulic jacks, and anchored against the concrete. The duct is then filled with cement grout or other substance designed to prevent corrosion of the tendons.

In some cases, the tendons are inserted into the ducts before casting. This poses the hazard of accidental grout leakage into the ducts, which would impede the later tensioning.

Post-tensioning is used to create the desired state of initial stress in cast- in- place concrete structures. This requires that the structure be supported in such away that it is free to shorten, either by sliding on its bearing or by deflection of the support.

Post-tensioning can also be utilized to join pre-cast segments while at the same time creating the desired prestress in the structure. For such operations, accuracy is required in the assembly of the pre-cast members, especially to ensure the continuity of ducts.

In post-tensioning if the tendons are grouted after prestressing then the tendon become bonded to the concrete section and is called bonded tendons. But if tendons are prevented from becoming bonded to the concrete then this type of post-tensioning is usually referred to as unbonded construction.

Post-tensioning offers a means of prestressing on the job site. This procedure may be necessary or desirable in some instances. Very large bridge girders that cannot be transported from a precasting plant to the job site (due to their weight, size, or the distance between plant and job site) can be manufactured by post-tensioning on the job site. Post-tensioning is used in pre-cast as well as in cast-in-place construction.

In post-tensioning, it is necessary to use some type of device to attach or anchor the ends of the tendons to the concrete section. These devices are usually referred to as end anchorages. The end anchorages, together with the special jacking and grouting equipment used in accomplishing the post-tensioning by one of the several available methods, are generally referred to as post-tensioning systems.

Post-tensioning systems consist of the following assemblages:

- Single wires or single T-wire strands.

- Groups of wires or multistrand tendons.
- Single wires or single strands pre-encased in a sheath of plastic or paper with corrosion – inhabiting and lubricating material between.
- Single bars.

Anchorages for these systems include:

- Button heads for the wires.
- Wedge anchors for the strands.
- Threaded- nuts for the bars.

In basic systems for direct longitudinal post-tensioning we can mention some: [4]

1. **The Freyssinet system**: - the essential feature of the Freyssinet system is the conical shape of the anchorage, which secures the wires comprising the tendon. (See Fig.4.1)

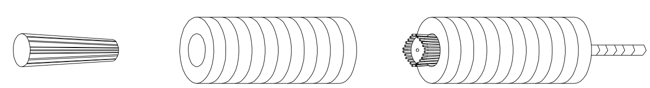


Fig. 4.1 The Freyssinet System

2. **The Magnel- Blaton System**: - instead of tensioning the whole tendon in one operation, as with the Freyssinet system, the Magnel- Blaton system involves the tensioning of well-spaced wires either singly or in pairs. In practice, the wires are always tensioned in pairs. At the point, the tendon emerges; a cast steel anchor plate is bedded in mortar on the end of the beam. Against this rest the " sandwich plates" to which the wires are to be anchored. (See Fig. 4.2)

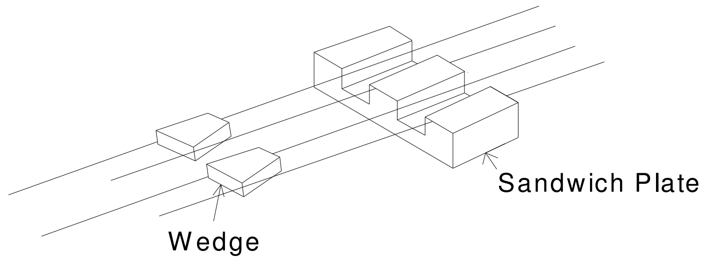


Fig. 4.2 The Magnel- Blaton System

3) **The Macalloy (stressteel) system**: - In the original Lee-Mc Call system the end of the bar had a screw thread cut into it, the bar being anchored to a bearing plate by a nut. (See Fig. 4.3)

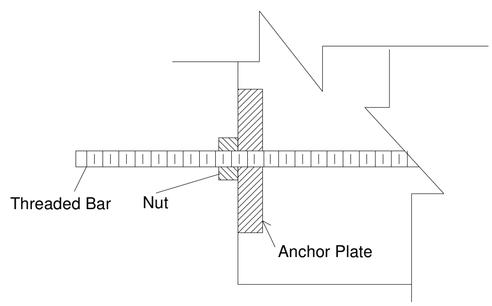


Fig. 4.3 The Maclloy System

4) **The Gifford- Udall System**: - a modification of the Freyssinet system which can provides better facilities for grouting. The convenience of a small stressing jack has given rise to the development of this system to single- wire tensioning system. The system is modified by the provision of anchor plates having tapered holes, each wire being gripped by three conical wedges. (See Fig. 4.4)

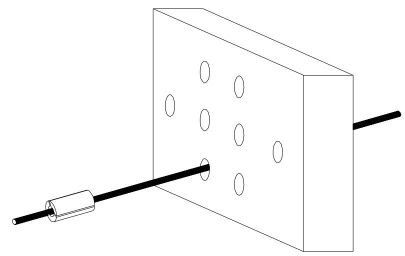


Fig. 4.4 Single-Wire Tensioning

5) **The Prescon System**: - It consists in forming “button heads" (a cold - upsetting or cold -forging process which is also some time called" riveting) at the end of the wires before stressing. With this system the button- heads attach the wires to an anchor head, which is drawn back by the jack, shims being inserted to take up the extension. (See Fig. 4.5)

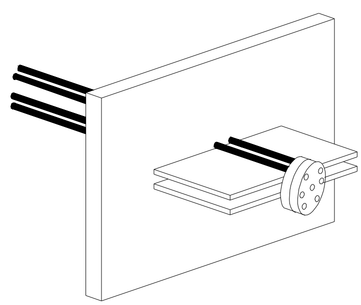


Fig. 4.5 Anchor for the Prescon System.

6) **Klemm Anker or K.A system**: This is based on clamping anchorage. For tendons of up to ten wires they are arranged in pairs on either side of a central bolt, for tendons of from twelve to forty wires they are arranged in layers of four, between two outer bolts. (See Fig. 4.6)

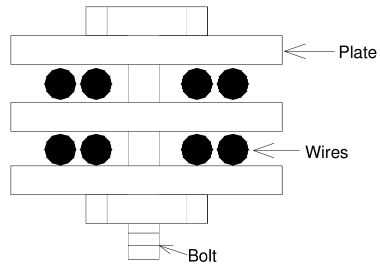


Fig. 4.6 The K.A System (with one bolt)

4.1.2 Prestressed Concrete Bridges

4.1.2.1 Introduction

Many of the most notable successes of prestressed concrete have been in the construction of bridges. Its use offers advantages of low initial cost, low maintenance, high durability, and attractive aesthetics.

A prestressed concrete has been successfully employed for bridges ranging from short span to long spans, in all environmental conditions. It has been widely utilized in crowded city viaducts. It concepts range from simples span slabs to cantilevered segmental construction to cable- stayed spans.

Prestressed concretes superstructure may be composed of pre-cast or cast-in – situ elements. When pre-cast elements used, the segments are joined with other elements for monolithic behavior by cast-in- place concrete or prestressing or both.

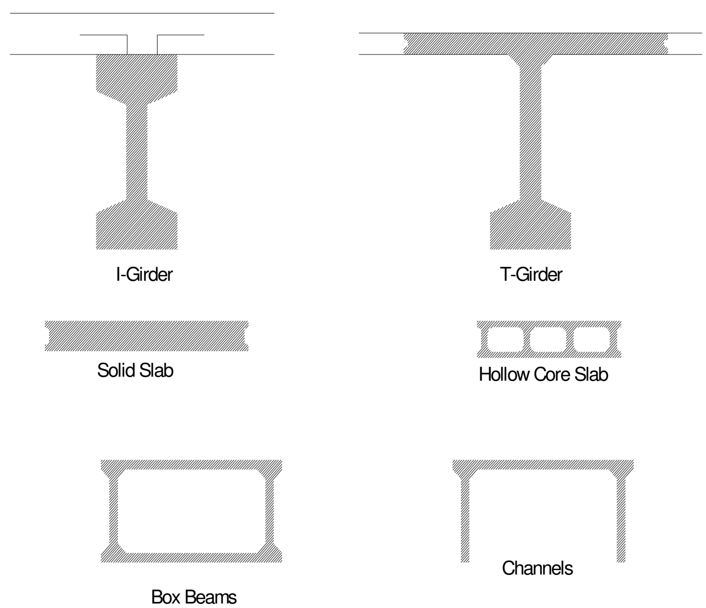
4.1.2.2 Bridge Superstructures Constructed of Pre-cast Elements

4.1.2.2.1 Pre-cast Pretensioned bridge Elements

Pre-cast pretensioned girders have been standardized in some countries. The standard section includes I-girders, Tee-girders, bulb tees, doubles tees, channel section, box girders, hollow-core slabs and solid flat slabs. Precast pretensioned girders are generally employed for spans less than 35meters. When joined by a cast- in-place concrete deck, they act in composite action, as a highly efficient and economical bridge superstructure. (Refer to Fig. 4.6)

Newer sections, such as the bulb tee, have thinner webs, and employ higher strength concrete and shear reinforcement. As a result, they have extended the economical span range to 45 meters.

Combinations of pre-cast pretensioned segments of about 30meter maximum length can be spliced by post-tensioning, for examples at the inflection points, to extend the span range to over 50 meters.



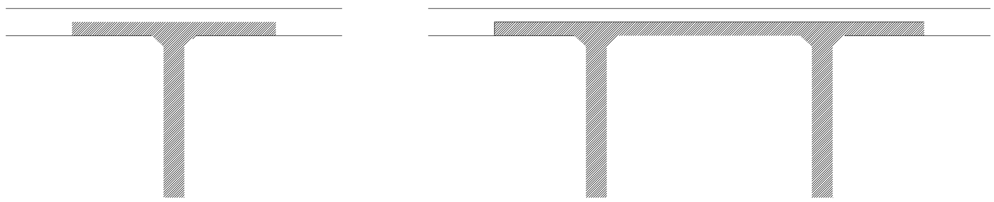


Fig. 4.6 Some of Standard Sections

4.1.2.2.2 Pre-cast Post-tensioned Bridge Superstructures

Some of the more recent bridge superstructures have been composed of precast members, which have been post-tensioned before or after erection.

One widely employed system embodies the sequential erection of precast segments by the method of progressive cantilevering. After each segment is erected and the joint has been made, it is post-tensioned back to the pier head.

In a second scheme, the precast segments may be erected in their final position on false work. After adjusting for dead weight deflection to final point, joints are made and the span is post-tensioned, lifting the positive moment region off of the false work.

5.1.2.3 Cast-in–place Post-tensioned Concrete Superstructures

Cast-in–place post-tensioned bridge construction methods are also extensively employed and are especially adapted to long spans and complex and curved crossings typical of highway interchanges. Cast-in-place methods have been extensively used with cantilevers segmental construction, when successive segments are cast against the preceding member and prestressed to it before proceeding.

4.1.2.4 Major Construction Methods

4.1.2.4.1 Launching Gantry Method of Erection

One of these methods involved the use of a special erection or launching gantry, which may include means for moving itself forward as portions of the bridge are completed.

The launching gantry is typically a steel framed bridge of cantilever-truss or cable-stayed truss type, which spans two bays. While a segment is being erected it is supported on a central pier, a forward pier, and a rear pier. After a span has been erected, the launching gantry slides itself forward, cantilevering over one pier, until it reaches the next one. It transfers legs on to this new pier, establishes a support by jacks, and the cycle recommences.

Under this system, a pre-cast segment is moved forward on rails or trucks, riding on the completed portion of the deck. The segment is then picked up by the launching gantry and carried forward and set in its position. The individual precast segments are then jointed and stressed before the next segment is launched.

In the other system of this method, it has been employed to move precast girders length wise from a completed portion of the superstructure to their span location. A light steel or aluminum launching nose is over balanced by a counter weight on the rear end of the girder. Forward movement could be accomplished by jacking, rolling, tracked carriages, or cranes. The girder must be analyzed for temporary stress conditions as it is cantilevered forward and, if necessary, strengthened by external trussing or temporary post- tensioning. This method is particularly suitable for a single span in remote locations.

4.1.2.4.2 Cantilever Segmental Construction

This is a widely used and useful method for the construction of concrete bridges with precast or cast-in -place segments. As each segment is placed, it is jointed and stressed back to the completed portion of the superstructure. The sequence of the erection is chosen to keep the partially completed superstructure balanced about a pier, in double-cantilever. Since it is not feasible to lift or concrete the two segments exactly simultaneously, a step-by-step sequence is adopted, in which one segment is erected on one side, then one on the other. This puts bending moments in the pier, for which it must be designed. The unbalanced load is the load of the segment plus any construction equipment. (Refer to Fig. 4.7)

This system may be employed with cast -in place segments or precast segments. With cast-in -place segments each segment is directly cast against the preceding one. Pre-cast segments are also employed in cantilever segmental construction. These are match- cast segments, jointed with epoxy glue. They enable very rapid completion of spans.

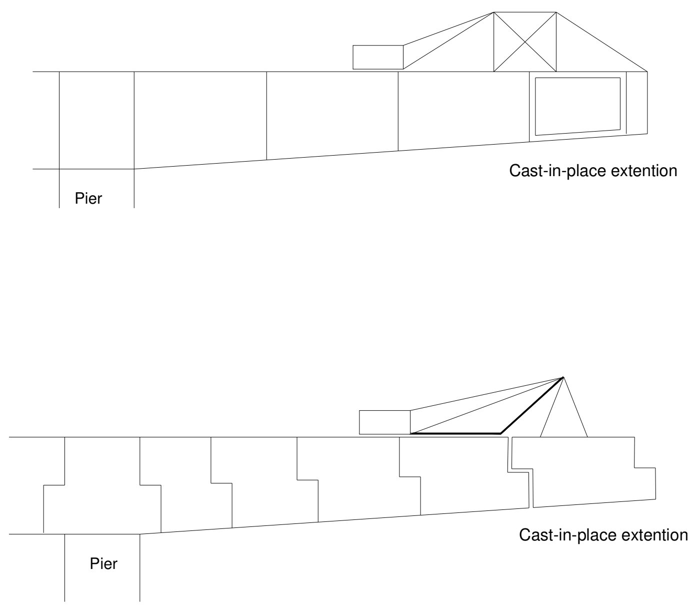


Fig. 4.7 Method of Progressive Cantilevering

4.2. Possibilities of Application in the Country

4.2.1 General

As it is known the country has big rivers and deep gorges in its topographic formation. In the major part of the effort to build the transportation infrastructure of the country, roads play the major part. And bridges are the major cost sharing components of road construction.

From literal understanding one could easily estimate that road construction require a huge amount of investment. This huge amount of investment usually comes from the little income the country has and the majority contribution from long-term loan or donation.

One thing that should not have to be forgotten is that, the country is highly underdeveloped. Hence it is definitely needed to use very effectively the little resource the country has to improve the over all standard of living.

This could be possible one way by the application of new systems that are very economical and efficient. From this point of view prestressed concrete bridge construction system could be implemented in the country.

4.2.2 Possible Methods

It may not be necessary to apply all the latest technology at once. It could be possible to adapt those that are suitable to our local conditions and capacity. The following methods could be applied.

4.2.2.1 Pre-cast Post-tensioned Girders

A number of small bridges one after the other in a short distance could easily be constructed using pre-cast post-tensioned superstructure system. A pre-casting yard can be located at an average distance to minimize transportation cost. In order to reduce weight even it is possible to produce girders in segments and post tension them on site. The pre cast post-tensioned girders can be put in place using mobile cranes on the river bed if the river is dry and the bridge is accessible from bottom. After the girders are in place the deck slab can be cast -in-place using shuttering supported on the beams. Other wise if the riverbed is not accessible from the bottom it may be possible to use launching gantry, which can easily be manufactured locally and to be assembled on site.

If the production of all girders at one site and transporting is either impossible or costly, it is possible and still a better solution to cast on site and post-tensioning.

Even the continuity of a bridge at its intermediate support is required, it could be done after erecting the pre-cast post-tensioned girders in place.

In this respect one can learn a lot from the experience gained from the construction of pre-cast normally reinforced concrete bridges in the country by a local contracting firms.

5.2.2.2 Segment Method

As one can learn from the construction of Abay Bridge in Bure- Nekmete route, this system is also applicable here.

Whenever the span of the bridge is long and it is not possible to handle one big pre-cast element to erect or under the condition that when the terrain or the river flow do not allow the use of scaffolding to cast concrete, this method of construction may be used.

It is also possible to manufacture shuttering locally that can be used to produce segments in cantilever method of constructing bridges.

4.2.2.3 Cast-in-place Post-tensioned

Under some circumstances like viaduct construction, it may be cost effective and efficient to apply cast-in-situ post-tensioned construction method.

The availability of modern scaffolding and very short height to bridges or viaducts concrete may be cast-in-place using shuttering supported on the scaffolding and the post-tensioning could be done.

With the development of towns, high traffic jams could be reduced using traffic interchanges. This could be possible by constructing light and aesthetically attractive viaducts using prestressed concrete structures.

5.1 Design Example on Prestressed Concrete Bridge

5.1.1 General Condition

An example bridge is designed to the following conditions.

- Simply supported 25 meters span girder bridge
- Live load ASSHTO loading
- 7.2 meters road way width with two lane

AASHTO SI Unit 2nd Edition 1998

- Pre-cast pretensioned I- section girders supporting a cast- in- situ slab.
- Analysis of the reinforcing steel requirement for the cast in- situ slab is not included.

5.1.2 Design calculations

Step1. Geometric and material properties

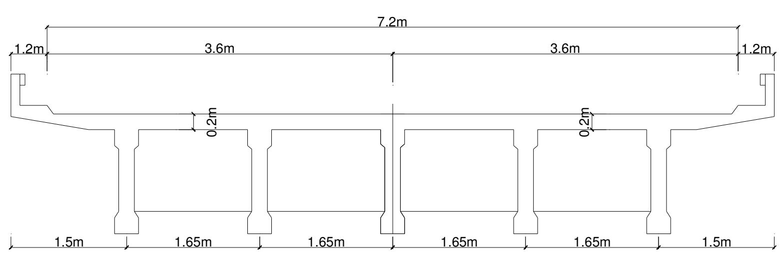


Fig. 5.1 Bridge Cross-section

- Minimum depth of $d = 0.045\,L$

Table 2.5.2.6.3.1

$$d = 0.045 \times 25 = 1.13 \text{ m}$$

take $d = 1.13$ m

- Considering the size of standard PCI type **III** I –girder section

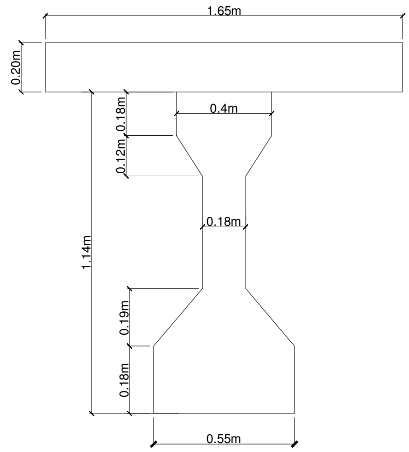


Fig.4.2Beam Cross-section

- Property of composite section

$$A_c = 0.6205 \times 10^6 \quad \text{mm}^2$$

$$y_b = 0.82 \text{ m}$$

$$I_c = 0.1304 \text{ mm}^2$$

$$Z_t = \frac{I_c}{(1.34 - 0.82)} = \frac{0.1304}{0.52} = 0.2508 \text{ m}^3$$

$$Z_b = \frac{I_c}{y_b} = \frac{0.1304}{0.52} = 0.159 \text{ m}^3$$

- The above are the section properties of the composite section but we need also the property of the I-beam alone.

$A_c = 0.3598 \text{ m}^2$

$Z_t = 0.0832 \text{ m}^3$

$y_b = 0.515$ m

$I_c = 0.05198$ m^4

$Z_b = 0.101$ m^3

The following design conditions are considered in the design calculation.

- Prestressing stand is Grade 270 with tensile strength (f_{pu}) of 1860 MPa. Table 5.4.4.1-1

- Concrete strength ($f_c^{'}$) of 45 MPa for the pre-cast I-girder and 30 MPa for cast-in- situ slab.

- Strength of concrete at transfer ($f_c^{'}$) is 45 MPa (attained full strength)

- The modules of elasticity of concrete. 5.4.2.4-1

$$E_c = 0.043 y_c^{1.5} \sqrt{f_c^{'}}$$

$$= 0.043(2400)^{1.5}\sqrt{45}$$

= 33900 MPa for pre- cast beam and

$E_c = 0.043(2400)^{1.5}\sqrt{30} =$ 27700 MPa for the slab

- The ratio in $\dfrac{E_{cslab}}{E_{cbeam}} = \dfrac{27700}{33900} = 0.8$

Step2. Compute stresses

- Stress in beam due to self-weight at center of span.

$$M_G = \frac{wl^2}{8} = \frac{24 \times 0.3598 \times 25^2}{8} = 674.63 \text{ kNm}$$

$$f_G^t = \frac{M_G}{Z_t} = \frac{674.68}{0.0832} = 8.11 \text{ MPa}$$

$$f_G^b = \frac{M_G}{Z_b} = \frac{-674.63}{0.101} = -6.68 \text{ MPa}$$

- Here the beam alone will support the dead load of slab and diaphragms.

- Stress in beam center of span due to cast-in-situ slab and diaphragm.

  - Moment due to slab

$$M_{ss} = \frac{0.2 \times 1.65 \times 24 \times 25^2}{8} = 618.75 \text{ kNm}$$

  - Moment due to middle diaphragm

$$M_{SD} = (0.2 \times 0.77 \times 1.65 \times 24) \times \tfrac{1}{2} \times \tfrac{25}{2} = 38.12 \text{ kNm}$$

Total moment $M_s = M_{ss} + M_{SD} = 618.75 + 38.12 = 656.87$ kNm

$$f_s^t = \frac{656.87}{0.0832} = 8.0 \text{ MPa}$$

$$f_s^b = \frac{-656.87}{0.101} = -6.5 \text{ MPa}$$

- Live load distribution

Table 4.6.2.2.2b-1

$$= 0.075 + \left(\frac{S}{2900}\right)^{0.6} \left(\frac{S}{l}\right)^{0.2}$$

where S = center to center spacing of girders

l = clear distance between girders

$$= 0.075 + \left(\frac{1650}{2900}\right)^{0.6} \left(\frac{1650}{1250}\right)^{0.2}$$

$= 0.829$

- Dynamic load allowance 3.6.2

= (1+ *IM*/100) = (1+33/100) =1.33

- Live load moment due to design truck and lane load 3.6.1.3

$M_L =$ (1.33(1651.13) +724.08) 0.829

= 2420.75 kNm

- The stresses are

$$f_L^{ts} = \frac{M_L}{Z_t} = \frac{2420.75}{0.2508} = 9.7 \text{ MPa}$$ at slab top fiber

$$f_L^{t} = 9652 \times \frac{0.32}{0.52} = 5.9 \text{ MPa}$$ at beam top fiber

$$f_L^{b} = \frac{M_L}{Z_b} = \frac{-2420.75}{0.159} = -15.2 \text{ MPa}$$

Step 3. Determining prestressing force and number of tendons

- Here computation is based on conditions after all stress losses.
- Maximum allowable compression

Table 5.9.4.2.1-1

$= 0.45\, f_c^{'} = 0.45(45) = 20.25$ MPa

- Allowable tensile stress (in the precompressed tensile zone) is

Table 5.9.4.2.2-1

$$= 0.5\sqrt{f_c^{i}} = 0.5\sqrt{45} = 3.35 \text{ MPa}$$

- From the above stress calculations in Step2, the total tensile stresses in the bottom fiber under design load conditions.

$f_G^b + f_s^b + f_L^b$ = -(6679.5 + 6503.7 + 15224.8)

= -28.41 MPa

- Since a tensile stress of 3.35 MPa under design load, the compressive stress, which must be developed by the tendon is

= 28.41 – 3.35 = 25.0 MPa.

but this is in excess of the 20.25 MPa permitted in compression and means that some of the stands must be deflected so that the bottom fiber stress due to prestress at the ends of the beam will be less than 20.25 MPa.

- At the mid- span the stress can be reduced due to girder self weight: -

$= 25.00 - 6.68 = 18.32$ MPa < 20.25 MPa   Ok!

- The section selected is adequate.
- The prestressing force is applied only to the beam rather than to the composite section.
- Assuming an approximate value for *e* say 15% height above bottom of beam to be center of gravity of strand (c.g.s).

$$e = y_b - 0.15 \times 1.14$$

$$= 0.515 - 0.17 = 0.345 \text{ m}$$

- From $$\frac{F}{A_c} + \frac{Fe}{Z_b} = f_G^b + f_s^b + f_L^b - f_{tp}$$

$$\frac{F}{0.3598} + \frac{F(0.345)}{0.101} = 6679.5 + 6503.7 + 15224.8 - 335.0$$

$$F = 4535 \text{ kN}$$

- 15.24 mm diameter strand size is selected.
- The maximum allowable stress in tendon $= 0.7 f_{pu}$ Table 5.9.3-1

$$0.7(1860) = 1302 \text{ MPa}$$

- Assuming prestress lump sum loss of Table 5.9.3.3-1

$$= 230\left(1 - 0.15 \times \frac{(f_i - 41)}{41}\right) + 41(PPR)$$

$$= 230\left(1 - 0.15 \times \frac{(45 - 41)}{41}\right) + 41 \times 0.6$$

$$= 251.2 \text{MPa}$$

- Capacity of single strand after all losses

$$= 1302 - 251 = 1051 MPa$$

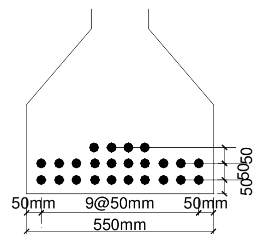
$$\text{number of strand} = \frac{F}{\text{capacity of single strand} \times \text{area of single strand}}$$

$$= \frac{4535 \times 10^3}{1051 \times 182.41}$$

$$= 24 \text{ strands}$$

Step 4. Locate the strand position and compute the actual eccentricity.

- Minimum clear distance between strands is 44mm in horizontal and 25mm in vertical arrangement. 5.10.3.3.1



- C.G.S = 87.5 m
- e = 515-87 = 427.5 mm
- The compressive stress in the bottom fiber due to new strand pattern.
- The new $F = 24 \times 1051 \times 182.4 = 4600.9$ kN

$$f_F^b = \frac{F}{A_c} + \frac{Fe}{Z_b}$$

$$= \frac{4600 \times 10^3}{0.3598} + \frac{4600 \times 10^3 \times 0.428}{0.101}$$

$$= 32.3 \text{ MPa}$$

- This stress is larger than required hence we need to adjust

$$\frac{28.1 \times 24}{32.3} = 20.9 \qquad \text{provide 22 strands}$$

- The new eccentricity e = 0.433 m and the force

$$F = 22 \times 1051 \times 182.4 = 4217.5 \text{ kN}$$

- The stresses due to prestressing are

$$f_F^b = \frac{4217.5}{0.3598} + \frac{4217.5 \times 0.433 \times 10^3}{0.101}$$

$$= (11721.8 + 1808) \times 103 = 29.8 \text{ MPa}$$

$$f_F^t = (11721.8 - 18081) \times 10^3 = -6.36 \text{ MPa}$$

Step 5. Check stress at top and bottom at mid span at transfer.

- Assuming 10% initial loss of prestressing forces

$$F_o = 0.9(22 \times 182.4 \times 1302) = 4702.2\ kN$$

- The stresses are therefore

$$f_{Fo}^b = \frac{F_o}{F} f_F^b = \frac{4702.2}{4217.5}(29.8) = 32.2 \text{ MPa}$$

$$f_{Fo}^t = \frac{F_o}{F} f_F^t = 1.11(-6.36) = -7.06 \text{ MPa}$$

- Allowable compression stress in concert at transfer 5.9.4.1.1

$$= 0.6 f_{ci}^{'} = 0.6 \times 45 = 27 \text{ MPa}$$

- The net stress at transfer

$$f_{Fo+G}^b = 33.2 - 6.7$$

$$= 26.5 \text{ MPa} \quad < \quad 27 \text{ MPa} \qquad \text{OK!}$$

$$f_{Fo+G}^t = -7.06 + 8.12 = 1.06 \text{ MPa} > 0 \qquad OK!$$

- But allowable in tension at transfer 5.9.4.1.2

$$= 0.58\sqrt{f_{ci}^{'}} = 0.58\sqrt{45} = 3.89 \text{ MPa}$$

Step 6. Check stress at service condition.

- The net top stress

$$= - 6.36 + 8.11 + 7.9 + 3.29 = 12.94 \text{ MPa} < 20.25 \text{ MPa} \qquad \text{OK!}$$

- The net bottom stress

$$= 29.8 - 6.68 - 6.5 - 15.2 = +1.42 \text{ MPa} > 3.35 \text{ MPa} \qquad \text{OK!}$$

Step 7. **Check for ultimate flexural strength.**

- According to AASHTO strength – I loading the load combination

$$= 1.25(M_{DL}) + 1.75\ (M_L + IM)$$

$$= 1.25(674.63 + 618.75 + 38.12) + 1.75(2420.75)$$

$$= 1,664.38 + 4.236.32$$

$$= 5900.70 \text{ kNm}$$ 5.7.3

- The ultimate flexural resistance for bonded tendons

$$/_{ps} = /_{pu}\ (1- Kc/d)$$

for which K= 2(1.04 - $/_{py}$/$/_{pu}$) and C for rectangular section

$$c = \frac{A_{ps} f_{pu} + A_s f_y}{0.85 f_c^{'} \beta_1 b + K A_p \, {}^{f_{pu}}\!/_{d}}$$

$$K = 2(1.04\text{-}0.9) = 0.28$$ 5.7.2.2

$$c = \frac{4013 \times 1860}{0.85(45)0.75(1650) + 0.28(4013)\,{}^{1860}\!/_{1250}} = 158 \text{ mm} < 200 \text{ mm}$$

hence rectangular

$$/_{ps} = 1860(1- 0.28 \times 158/1258)$$

$$/_{ps} = 1794.6 \text{ MPa}$$

- The flexural resistance 5.7.3.2

$$M_r = \omega M_n$$

$$\omega = 1.0$$

$$M_r = 1.0\ (M_n)$$

$$M_n = A_p /_{ps} (d\text{-}a/2)$$

$$= 4013(1794.0)\left(1258 - \frac{0.75(158)}{2}\right)$$

$$= 8630 > 5900 KNm$$ OK!

- Maximum reinforcement 5.7.3.3

$$c/d < 0.42$$

$$158/1258 = 0.126 < 0.42$$ OK!

Step 8. Establish the path of the tendons.

- As mentioned in Step 2 it is necessary to slope some of the strands up wards near the ends of the beam.

- Stresses in concerts at mid-span due to prestress only are

f^t_{Fo} = -7.06 MPa $\qquad$ f^t_F = -6.36 MPa

f^b_{Fo} = 33.2 MPa $\qquad$ f^b_F = 29.8 MPa

- The maximum permissible value of *e* is that which bring each of the foregoing stresses within the allowable. The maximum *e* for each condition can be computed by setting the allowable stress equal to the stress caused by the prestressing force.

1) Top of beam at transfer:

$$f_{Fo}^{t} = \frac{F_o}{A_c} - \frac{F_o e}{Z_t}$$

$$-3.67\times10^{3} = \frac{4218}{0.3598} - \frac{4218e}{0.0832}$$

$$e = 0.304 \text{ m}$$

2) Bottom of beam at transfer

$$f_{Fo}^{b} = \frac{F_o}{A_c} + \frac{F_o e}{Z_b}$$

$$27\times10^{3} = \frac{4702}{0.3598} + \frac{4702\ e}{0.101}$$

$$e = 0.30\ m$$

3) Top of beam under final conditions

$$f_{Fo}^{t} = \frac{F}{A_c} + \frac{Fe}{Z_t}$$

$$-3.35\times10^{3} = \frac{4218}{0.3598} - \frac{4218e}{0.0832}$$

$$e = 0.297 \text{ m}$$

4) Bottom of beam under final conditions

$$f_{F}^{b} = \frac{F}{A_c} + \frac{Fe}{Z_b}$$

$$20.25\times10^3 = \frac{4218}{0.3598} + \frac{4218e}{0.101}$$

$$e = 0.204 \text{ m}$$

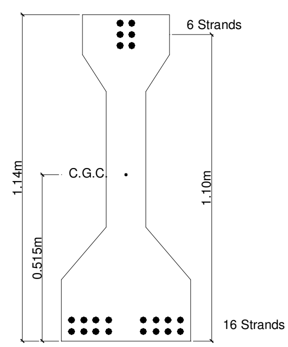
- The maximum allowable e at the ends of the beam is 0.204 m
- The moment of 22 strands about bottom of beam = $22 \times 0.311 = 6.842$
- If 6 strands are to deflect upwards, the moment of those remaining straight

$$= 16 \times 0.075 = 1.2$$

- The moment of 6 strands deflected up ward must be at least

$$= 6.84 - 1.2 = 5.64$$

- The distance from bottom of beam to the c.g.s of the six strands must be at least 5.64/6 = 0.94 m
- Arrange the c.g.s of 6 strands at 1.10 m from bottom of beam.



- The c.g.s of the entire group = 0.355 m
- The eccentricity $e = 0.515 - 0.355 = 0.16$ m
- Stresses at end of beam under final conditions

$$f_F^t = \frac{F}{A_c} - \frac{Fe}{Z_t}$$

$$= \frac{4218}{0.3598} - \frac{4218(0.16)}{0.0832}$$

$$= 3.61 \text{ MPa}$$

$$f_F^b = \frac{F}{A_c} - \frac{Fe}{Z_b}$$

$$= \frac{4218}{0.3598} - \frac{4218(0.16)}{0.101}$$

$$= 18.4 \text{ MPa} \qquad OK!$$

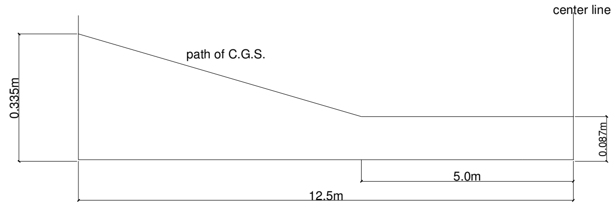
- Stresses at transfer

$$f_{Fo}^t = 1.11 f_F^t = 1.11 \times 3.61 = 4.0 \text{ MPa}$$

$$f_{Fo}^b = 1.11 f_F^b = 1.11 \times 18.4 = 20.4 \text{ MPa} \qquad OK!$$

- The location of the hold down points can be taken $0.4 \times (25/2)$ =5.0 m from center of span both directions.

- These points can be located after calculating the critical combinations of loading conditions at critical point along the beam, but after working a few examples the engineer can recognize the critical conditions and locations.



- The critical conditions are : -

  1) Prestress at transfer plus dead load of beam only at top and bottom fiber.

  2) Prestress at service plus all applied loads at top and bottom fiber.

- The hold-down point must be located far enough from mid span to eliminate excessive tensile stresses in the bottom fiber under final prestress plus full live load and it must not be too far from mid-span or there will be excessive tensile stresses in the top fiber after transfer of prestress to the member when only its own dead weight is acting.
- It is necessary to make sure that stresses are within the allowable at all points along the beam under the critical conditions.
- The stresses can be plotted and the net stress can be checked to be within the allowable.
- Here only the stress at hold-down point is checked.
- For the first critical condition

$$M_G^{'} = 0.84(674.63)$$

$$f_G^t = \frac{M_G^{'}}{Z_t} = \frac{566.7}{0.0832} = 6.8 \text{ MPa}$$

$$f_G^b = \frac{M_G^{'}}{Z_b} = \frac{566.7}{0.101} = -5.61 \text{ MPa}$$

- Net stress $f^t = 6.8 - 7.06 = -0.96$ MPa

$f^b = -5.61 + 33.2 = 27.59$ MPa

which is a beat higher than the allowable 20.25 MPa

- For the 2nd critical condition

$M_{DL} = 0.84(674.63 + 656.87)$

$= 1118.5$ kNm

$M_{LL} = 1933$ kNm

$M_T = 1118.5 + 1933$

$= 3051.5$ kNm

$$f_{TL}^t = \frac{3501.5}{0.2508} = 12.2 \text{ MPa}$$

$$f_{TL}^b = \frac{3021.5}{0.159} = -19.2 \text{ MPa}$$

- Net stress $f^t = 12.2 - 6.36 = 5.84$ MPa

$f^b = -19.2 + 29.8 = 10.6$ MPa OK!

Step 9. Design for shear steel.

- In prestressed concrete, shear under design load condition, is never critical in members which meet ultimate load requirements for shear. Therefore the ultimate condition is checked for shear.
- The ultimate shear at *d* distance from face of support, which is 1.3 m from center of support.

Dead load shear = 185.4 kN

Live load shear = 271.12 kN from truck load

= 104.16 kN from lane load

- The ultimate shear force

V_{UT} = 1.25 (185.4) + 1.75 (1.33(271.12) + 104.16)

= 1045.1 kN

- Transverse reinforcement shall be provided for 5.8.2.4

$V_u > 0.5\omega(V_c + V_p)$

$V_c = 0.083\beta\sqrt{f_i}bd$

- Where the concrete shear $= 0.083(2.0)\sqrt{45}(0.22)1.0\times10^6$

= 245 kN

- V_P is the vertical component of prestressing strand sloping up ward

$$V_p = \frac{0.355 - 0.0875}{12.5 - 5.0}\times 4218$$

$= 150.4KN$

- V_{UT} = 1045.1 > V_u = 0.5 (0.9) (245 + 150)^4 = 178KN

- Hence transverse reinforcement is required.

- The nominal shear resistance is the lesser of

$V_n = V_c + V_s + V_p$

$V_n = 0.25\ f_c^{'}$ b d +V_p

where V_c = 245 kN

V_p = 150 kN

- Starting from the first equation

$$V_s = V_n - (V_c + V_p)$$

$$= 1045 - (245 + 150)$$

$$= 650 \text{ kN}$$

- Using 2ϕ12 stirrups $A_s = 2(113) = 226 \text{ mm}^2$ and the spacing required

$$S = \frac{A_v f_y d \ (\cot\theta + \cot\alpha)}{V_s} \sin\alpha$$

$$= \frac{226(410)1000(\cot 45 + \cot 45)}{650 \times 10^3} \sin 45$$

$$S = 285 \text{ mm}$$

- Checking the second equation

$$V_n = 0.25\ (48)\ 220 \times 1000 \times 150 \times 10^3$$

$$= 2625 \text{ kN} > 1045 \text{ kN} \qquad \text{OK!}$$

- Provide 2ϕ12 stirrups @285mm

5.2 The Summery of Ordinary Reinforced Concrete Bridge Design

5.2.1 General Design conditions

The same bridge 25meter span is redesigned using the normal reinforced concrete principles to the following conditions for the purpose of cost comparison.

- The design is according to AASHTO SI Unit 1998 2nd edition.
- Concrete quality of f_c' = 35 MPa and G-60 reinforcement steel.
- 7.2 m wide roadway width with two lanes.
- The design of deck slab is not included.

5.2.2 Design Calculations Summary

The same general arrangement of bridge cross section is selected.

- Minimum depth according to AASHTO recommendation

$$0.07L = 0.07(25) = 1.75 \text{ m}$$

Table 2.5.2.6.3-1

- Selecting the standard PCI **IV**- section with modified height

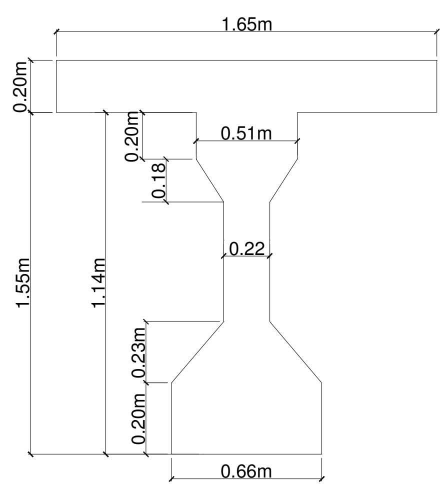


Fig. 5.3 Beam Cross-Section

- Live load distribution to each girder for moment

$= 0.075 + (S/2900)^{0.6} (S/L)^{0.2}$

$= 0.075 +(1650/2900)^{0.6} (1650/1250)^{0.2}$

$= 0.83$

- Dynamic load allowance 3.6.2

$(1 + IM/100) = (1 + 33/100) = 1.33$

- Dead load moment

$$M_{DL} = \frac{wL^2}{8} = \frac{22.33 \times 25^2}{8} = 1745 \text{ kNm}$$

and due to diaphragm = 50.1 kNm

- Live load moment 3.6.1.3

due to truck load =1651.13 kNm

due to lane load =724.08 kNm

- Total live load moment

$=(1.33 \times 0.83 \times 1651.13) + 0.83(724.08)$

$= 2423.7$ kNm

- Dead load shear

$$V_{DL} = \frac{22.33 \times 25}{2} + 4.0 = 283.13 \text{ kN}$$

- Live load shear = 271.66 kN due to truck load

and = 117 kN due to lane load

- Live load distribution for shear

$$= 0.2 + S/3600 - (S/10700)^{2.0}$$

$$= 0.2 + 1650/3600 - (1650/10700)^{2} = 0.635$$

- Total live load

$$V_u = [1.33\ (271.66) + 117]\ 0.635 = 304 \text{ kN}$$

Table 4.6.2.2.3a-1

Design for Flexure

- The design is according to strength –I combinations.
- After the design calculations 16 ϕ30 bars are required.

Table 3.4.1-1

Design for Shear

- After shear design calculations ϕ12-stirrup bars at spacing 210 mm are required.

5.3 Comparison of Cost

5.3.1 In Terms of Material Usage

The material usage in the two types of bridge construction systems is compared based on the unit price derived from local market and data obtained from different bodies involved in this area and it include only the materials cost. The unit prices are set for one girder without its slab portion at Addis Ababa site.

1. **Ordinary Reinforced Concrete**

Material unit prices are: -

1. Concrete C-35 =600 Birr/m^3
2. AASHTO G-60 reinforcement steel bars = 6.00 Birr/kg

Material quantities per meter length: -

1. Concrete volume = 11.84 m^3
2. Total steel provided = 2500 kg

Total cost: -

1. For concrete = 11.84×600=7104.00
2. For reinforcement steel = 2500×6=15,000.00

Total cost in **Birr =2,2104.00**

2. **Prestressed Concrete**

Material unit prices: -

1. Concrete C-45 =650.00 Birr/m^3
2. Prestressing steel standard 15.24 mm strand =12.00 Birr/kg
3. Reinforcing steel =6.00Birr/kg

Material quantities are: -

1. Volume of concrete = 8.995 m^3
2. Weight of prestressing steel = 786.5 kg
3. Weight of reinforcing steel=317 kg

Total cost: -

1. For concrete = 8.995×650=5846.75
2. For prestressing steel = 786.5×12.00=9438.00
3. For reinforcement steel = 317×6.00=1268.00

Total cost in **Birr =1,7186.75**

3. Cost Difference

The total cost difference, only considering the girder cross-section between the two systems in percentage is: -

$$\frac{(22,104.00-17,186.75)}{22,104.00}\times 100 = 22\ \%$$

5.3.2 In terms of Construction Cost and Time

1. **In terms of cost**

The advantages of prestressed concrete concept in the construction or bridges are already discussed. In this section the construction cost of a cast-in-situ normally reinforced concrete bridge constructed using the common Eucalyptus false work and a pre- cast post-tensioned girder at site with cast-in-situ slab are compared.

The comparison is carried out to form one-meter length of the girder including its slab portion.

The unit prices are calculated for location in Addis Ababa and includes all labour and equipment cost involved.

Normally Reinforced Concrete

Activities and their volume of work: -

1. River bed preparation for false work = 0.99 m^3 (earth work)
2. False work erection = 0.77 m^3 (Eucalyptus)
3. Shuttering = 5.18 m^2
4. Bar laying = 119.44 kg
5. Concreting = 0.8 m^3

Unit costs are: -

1. For river bed preparation =54 Birr/m^3
2. For false work =1000 Birr/m^3
3. For shuttering = 100 Birr/m^2
4. For bar laying = 1.25 Birr/kg
5. For concreting = 120 Birr/m^3

Total cost: -

1. River bed preparation = 0.99×54 = 53.46
2. False work = 0.77×1000 = 770.00
3. Shuttering = 5.18×100 = 518.00
4. Bar laying = 119.44×1.20 = 149.30
5. Concreting =0.474×120 = 56.80

Total cost in **Birr** per meter = **1,547.56**

B. Prestressed Concrete

Activities and their volume of work: -

1. Per-cast bed by the bridge site = 0.163 m^2 (hard core with cement screed)
2. Shuttering = 4.316 m^2
3. Post-tensioning = 22strands per girder
4. Concreting = 0.69 m^3
5. Pre-cast girder moving & erection = in terms of the various equipment time

Unit costs are: -

1. For pre-cast bed = 35 Birr/m^2
2. For shuttering = 60 Birr/m^2
3. For post tensioning = 460 Birr/m
4. For concreting = 80 Birr/m^3
5. For moving and erection work = 40 Birr

Total cost: -

1. Pre-casting bed = 0.163×35 = 5.80
2. Shuttering = 4.316×60 = 258.96
3. Post-tensioning = 1×460 = 460.00

4. Concreting $= 0.69 \times 80 = 55.20$

5. Moving and erection $= 40.00$

Total cost in **Birr** per meter = **820.00**

C. Cost Differences

The total cost difference between the two systems based on construction cost in percentage: -

$$\frac{1547.26 - 820.00}{1547.26} \times 100 = 47\ \%$$

This 47% is very appreciable number at least at preliminary study level. One can easily understand the advantages of the prestressed concrete construction system.

2. In terms of Time

One may consider equal construction time for substructure work for both cases. However the construction of the superstructure for cast-in -situ normally reinforced concrete bridge starts only after the substructure is completed. But in case of pre-cast post-tensioned system the girders could be completed before or equally with completion of the substructure work and the erection could be done immediately.

In addition to the time saving in construction of bridges, it has also a very important advantage in minimizing the risk of forest resources depletion.

6. Conclusion and Recommendation

6.1 Conclusion on the Design

As it is tried to show briefly the concept behind prestressed concrete behavior there is no as such complicated idea in it. It is simply the idea developed for better performance of concrete structures. It is also efficient way of using high strength concrete and steel.

If this is the secret behind prestressed concrete design it will not be difficult matter for those who are involved in the design of bridges. Hence they may easily learn the design. Once the design of bridges started using prestressing concept, the whole bridge design professionals could follow.

An effort is seen from the design professionals for the implementation of the prestressed concrete bridge design in the country.

6.2 Conclusion on the Construction

In general terms the bridge construction technology in Ethiopia is not well developed, as it should have been.

The assignment of one organizational body to take care of the design and construction of bridges in the whole country may be one of the main reasons why no substantial change is seen in the construction technology of bridges.

As explained in chapter two of this thesis, the prestressed concrete bridge construction technology is not a new technology to our country. Some how some people have already learned the technology while working with foreign contractors. But no effort is done to use this experience and apply it to new similar projects.

Even when the foreign consultants came up with design of prestressed concrete bridge, the main body, which bears the responsibility for the construction realization of major routes in the country, is not willing to accept it. The reason given is that the prestressed concrete construction technology requires machines and skilled labour. But this alone cannot be enough reason to reject the proposal.

In general it can be concluded that there is no reason for not applying prestressed concrete bridge construction in the country except the lack of coordination between the different bodies involved in this area.

6.3 Recommendation on the Application

It is tried here to give the recommendations separately in three general groups.

In the first group, training institutes are included. Those involved in this area in general must train engineers in the area that could bring a change in our very poor living condition by introducing new technologies in which the developed world already used, tested and proven it economical. In this respect the prestressed concrete concept has to be included as a major course in under graduate course at the university level.

The concept of prestressing with its method of application should be given at technical school level. The graduates from this technical school could easily be trained for prestressed concrete construction systems.

In the second category, the consulting firms involved in the design of bridges through out the country must come up with the design alterative in prestressed concrete bridge, clearly indicating the advantages in cost and time. They must also always investigate new systems of construction that could minimize cost, save time and boost quality of construction. These must be done not once but all the time.

In the third group, we found the main bodies, which plan, evaluate and allocate fund for the construction of bridges through out the country. These groups have the responsibility to encourage and support those who come up with new ideas that could minimize cost and save time with equal service. This will greatly helps in the realization of building bridges, giving access to the rural community, with the little fund available.

When a new system is adopted replacing the old one, there will definitely be a resistance by those involved in the old and this may incur additional expenses and challenges. Therefore it is necessary to work hard till the implementation, despite the challenges. Once these stages are passed then things will go by themselves.

It is highly believed that the main bodies, which are currently engaged in the planning and implementing the construction of bridges must have to take the first initiative and effort for the implement the prestressed concrete bridge construction. This could be carried out by producing the necessary skilled labor for work task through their training centers. The necessary machines and equipments could even be imported by them and could be leased for others.

At last but not least, all persons and organizations involved in the business of bridge design and construction and of course the professional associations and higher institutes must work hard hand -in-hand so that the whole nation could get the benefit out of the prestressed concrete bridge construction system in general.

.

References

1. AAASHTO LRFD Design Specification (1998)
2. AAHSTO Design Specification (1963)
3. R.I.Gilbert and N.C.MickLeborough, Design of Prestressed Concrete (1990)
4. Paul W. Ables, Introduction to Prestressed Concrete vol1&2(1964)
5. T.Y.Lin and NedH.Burns, Design of Prestressed Concrete Structures (1982) (3^{rd} Edition)
6. H.Kent Preston and Noman J.Sollenberger, Modern Prestressed Concrete (1967)
7. James R.Libby, Modern Prestressed Concrete (1971)
8. C.Menn, Prestressed Concrete Bridges (1990)
9. Firtz Leonhardt, Prestressed Concrete Design and Construction (1964)
10. Ben C. Gerwick, JR., Construction of Prestressed Concrete Structures (2^{nd} edition)
11. Design drawings of Abay River Bridge
12. Design of Awash River Bridge Firtz Leonhardt