Computer Program for Deflection Chart of Simply Supported Prestressed Railway Girder

Addis Ababa University
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Railway Civil Engineering

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A thesis submitted to school of graduate studies, Addis Ababa University, Addis Ababa Institute of Technology in partial fulfillment of the requirement for the degree of M.Sc. Railway Civil Engineering

Advisor: - Dr. Essayas G/Yohannes (Ass. Professor)

January, 2016
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Addis Ababa
Ethiopia

CERTIFICATION

I, the undersigned below, certify that I read and hereby recommend for the acceptance by the Addis Ababa University a dissertation entitled: **Chart Development for the Estimation of Deflection in Prestressed Railway Girder**: in partial fulfillment of a degree of Masters of Science in Railway Civil Engineering.

_____

Dr. Essayas G/Yohannes (Ass. Professor)

Thesis Advisor

Date: _____

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Date defended: _____

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   -- (External examiner) (Signature)

DECLARATION

Fiseha Nega declares that this thesis; titled as **Chart Development for the Estimation of Deflection in Prestressed Railway Girder** is my original work. This thesis is my individual work and it has not been presented before for any other institute and I will not present it to any other university for similar or any other degree award.

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Abstract

Light rail transit has been under construction in the city for the last two years and it crosses many intersection and roundabouts. Most of the time the rail crosses the road with elevated bridge using a prestressed girder but due to design limitation the span of the bridge is limited and this leads to a number of closely spaced piers in the city centers which interferes with the traffic flow and causes more traffic congestion to the ever growing city of Addis Ababa. Traffic congestion around intersection and round about is the major problem in big cities like Addis Ababa. Therefore when other infrastructure interferes with the existing flow the problem is worse. And one of the main factors that limit the span of the structure is deflection requirement. Therefore being able to assess and know the deflection of prestressed girder as precisely as possible will indeed help in designing an economical large span girders.

In designing prestressed girder, calculating deflection the necessary and rigorous step which one has to calculate at the end of the design process and tell whether the design girder is within the allowable limit or not. If it is not the designer has to change the cross sectional dimension and repeat the whole step once again. But if the designer can certainly know the deflection of that structure in advance of those steps he/she can eliminate such iteration.
Therefore this thesis intended to develop a program that plot a deflection chart for simply supported prestressed railway girder.

MATLAB programing language has been used to develop the program of deflection calculation and to plot the chart. Microsoft excel has also been used to develop a deflection calculation template for checking purpose.

Mainly AAHSTO LRFD Bridge specification, American Concrete Institute (ACI) publications and American Railway Engineering and Maintenance-of-way Association (AREMA) has been used in this thesis but other important material has also been referred when needed.

The findings were one can control the deflection or camber of the prestressed member with the magnitude of the applied load and varying the eccentricity of the tendon profile both at the support and at the mid-span. However the deflection vs. span length chart will show some unexpected behavior after a certain span length. It suddenly changes is dimension and direction in unanticipated manner. And this leads to the development of another chart “the design chart” which tells the minimum prestressing force needed and the working range of a certain geometric section for a given load. And from this we notice that for a given load and a given cross-sectional geometry a prestressed member has not only a maximum span length limit but also a minimum span length limit in order to serve the intended function without failure.

Acknowledgment

First and foremost, praises and thanks to God, the almighty, for providing me the capability to proceed successfully. The thesis appears in current form due to assistance and guidance of several people. I would like to thank offer my sincere thanks to all of them.

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I am greatly thankful to my cousin Getahun Melesse (B.Sc. Electrical Engineering), for his guidance and help in every step of the development of MATALB programming and for answering my silly and frequent questions without boredom and with great motivation.

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List of Abbreviation

ac :-area of concrete
acc :-area of concrete for composite section
Aps :- area of prestressing reinforcement
Apsmin :- minimum area of prestressing reinforcement
C :-cubic strength of prestressed concrete
Cs :-cubic strength of cast in-situ concrete
ec :- eccentricity at mid span
Ecp :- elastic modulus of prestressed concrete
Ecs :- elastic modulus of cast in-situ concrete
edg :- cantilever length for edge beam
ee :- eccentricity at support
emaxc :- maximum eccentricity at midspan corresponding to Pimin
emaxm :- maximum allowable eccentricity at midspan
emaxs :- maximum allowable eccentricity at support
eminm :- minimum allowable eccentricity at midspan
emins :- minimum allowable eccentricity at support
Ep :- elastic modulus of prestressing reinforcement
fbem :- bottom midspan stress at service
fbim :- bottom midspan stress at transfer
fbis :- bottom support stress at transfer
fbsm :- bottom midspan stress at erection
fbss :- bottom support stress at erection
fck :- characteristic compressive strength of concrete
fct :-compressive strength of concrete at transfer
fcw :-compressive strength of concrete at service
fpe :- effective tensile strength of tendon
fpew :- tensile strength of tendon at service
fpi :- applied tensile strength of tendon
fpit :- tensile strength of tendon at transfer
fpk :- tensile strength of tendon
fpy :- characteristic tensile strength of tendon
fr :- modulus of rupture of concrete
ftem :-top mid span stress at service
ftim :-top mid span stress at transfer
ftis :-top support stress at transfer
ftsm :-top mid span stress at erection

ftss :-top support stress at erection
ftt :- tensile strength of concrete at transfer
ftw :- tensile strength of concrete at service
H :- total Depth of a section
Ic :- cracked moment of inertia
Ie :- effective moment of inertia
If :- impact factor
Ig :-gross moment of inertia
Igc :-gross moment of inertia for composite section
L :- span length
le :- effective length
lef :- transformed effective length
Mdl :- moment due to dead load
Mll :- moment due to live load
Msl :- moment due to live load
Mtot :- total load due to all applied gravity load
np :- modular ratio between prestressing reinf and prestressed concrete
ns :- modular ratio between prestressed concrete and cast in-situ slab
Pe :- effective prestressing force
pff :- ratio of applied prestress force to the minimum prestressing force
Pi :-initial prestressing force
Pimin :- minimum prestressing force at transfer
Pmin :- minimum prestressing force
prl :-prestress loss
ts :- cast in-situ slab thickness
wdl :- self weight per meter length
wll :-live load per meter length
wsd :- super imposed load per meter length
yb :- distance from the neutral axis to bottom section of precast beam
ybg :- distance from the neutral axis to bottom section of composite beam
Yc :- height of neutral axis from bottom section
yt :- distance from the neutral axis to top section of precast beam
ytg :- distance from the neutral axis to top section of composite beam
yts :- distance from the neutral axis to top of cast in-situ slab
zb :- section modulus at bottom section of precast beam
zbp :- section modulus at bottom section of composite beam
zbs :- section modulus at bottom section of cast insitu slab
zt :- section modulus at top section of precast beam
ztp :- section modulus at top section of composite beam
γc :- unit weight of concrete

δdl :- deflection due to dead load
δi :- deflection due to prestressing force
δll :- deflection due to live load
δnes :- net deflection at erection just after super-imposed dead load
δnfl :- net deflection at service just after super-imposed dead load
δnt :- net deflection at transfer
δsl :- deflection due to superimposed live load
η :- the ratio of effective stress to initial stress

1 Introduction

1.1 Background

Each day, millions of commuters, tourists and students around the globe rely on regional rail transit as their primary source of transportation to and from city centers. This light rail is growing. Over the past several decades, the rapid expansion in city suburbs and a more environmentally conscious public have led city officials to believe that light rail can help alleviate traffic congestion and pollution. In addition, dozens of metropolitan areas across the country also see rail transit as a form of public transportation that can encourage economic development-if light-rail stations are strategically placed and city officials and private developers cooperate with one another.

Addis Ababa is also currently engaged in building This Light Rail Transit (LRT). The LRT, which has a total length of 34.35 km, has two routes – a 16.9-km section running from north to south, and a 17.35-km section stretching from east to west. The two sections run parallel for 2.7 km. The LRT will have a total of 41 cars, each with the capacity to carry 286 passengers at a time. This will enable the LRT to provide transportation to 15,000 people an hour, in one direction, and 60,000 in all four directions.

Addis Ababa's Light Rail Uses Semi-exclusive: A light rail transit alignment that is in a separate right-of-way or along a street or railroad right-of way where motor vehicles, pedestrians, and bicycles have limited access and cross at designated locations only." In this case we use Bridges to overpass intersection and roundabouts, to increase the span and to use the depth of the girder effectively, Addis Ababa LRT uses prestressed concrete girder.

Prestressed Concrete is a concrete in which internal stress of suitable magnitude and distribution are introduced so that the stresses resulting from external loads are counteracted to a desired degree. In reinforced concrete the prestress is commonly introduced by tensioning the steel reinforcement. Concrete is a building material that is strong but relatively weak in tension. So prestressing is introduced to overcome this problem. In prestressed concrete construction, steel tendons are stretched and anchored at each end so that compressive forces are applied to the concrete. The forces are transmitted from the tendons to the concrete either by the bond between the concrete and the tendon (pre-tensioning) or by embedded anchorage (post-tensioning), depending on the method of prestressing.

1.2 Statement of the Problem

Addis Ababa is a capital and the largest city of Ethiopia and it is the country's commercial, manufacturing, and cultural center. There are a large number of cars that uses the city's roads in a daily basis. Nowadays the number of car is increasing dramatically with the number of population that is why it became common to see traffic congestion everywhere in the city especially at cross section and roundabouts. The Addis Ababa LRT system has been under construction to minimize transport problem. The LRT passes all three modes of construction at different sites of the city, i.e. Underground, Aboveground and at grade. When the LRT pass through intersection and roundabouts it usually uses aboveground mode of construction (Bridge).

Prestressed girder is being used for construction of the bridge. However due to design limitation, difficulty of construction, and difficulty of hauling heavy pre-cast girder, and various factors the span of the prestressed girder is limited to a length of 40m, this leads to more closely spaced piers in the road. These piers become obstruction to the flow of traffic on the road, which causes more traffic congestion on the intersection and through roads where the rail track passes. And the main reason behind limited depth span is deflection requirement of prestressed girder. And the first step to achieve the possibility of designing large span prestressed girder is determining the deflection requirement of prestressed girder as precisely as possible. An accurate and precise determination of a beam like structure helps the designer to limit the span of the structure based on the allowable deflection for that specific intended purpose.

In designing prestressed girder, calculating deflection the necessary and rigorous step which one has to calculate at the end of the design process and tell whether the design girder is within the allowable limit or not. If it is not the designer has to change the cross sectional dimension and repeat the whole step once again. But if the designer can certainly know the deflection of that structure in advance of those steps he/she can eliminate such iteration.
Therefore this thesis intended to develop a program that plot a deflection chart for simply supported prestressed railway girder.

1.3 Objective

1.3.1 General Objective

The general objective of this thesis is to develop an empirical chart for the estimation of long term and short term deflection in prestressed railway girder based on the method recommended by ACI-435.

1.3.2 Specific Objective

The specific objectives of this thesis are

- To ease the calculation of deflection in prestressed girder
- To help the designer to certainly know the deflection of the selected beam prior to detail design and rigorous calculation.
- To question the ACI-435 method of deflection calculation (As the code itself questions it.) One can use another method or simulation technique to find the deflection of a specified beam and can compare it to the estimated deflection by ACI-435 method as it will be provided in the chart.
- For preliminary design of prestressed railway girder.
- To determine the working range of a given section.
- To increase the adaptation and use of prestressed girder design in the country

2 Literature Review

2.1 Prestressed Concrete

2.1.1 Introduction to Prestress Concrete

2.1.1.1 Prestressed concrete

Prestressing is more a philosophy than a specific technique. It means preparing a structure to receive a load by applying a pre-emptive countervailing load. For instance, if it is known that a column will be deflected 100 mm to the left by applied loads, the designer can arrange to bend it 50 mm to the right; the column then only has to be designed to resist a deflection of +/- 50 mm, rather than the full 100 mm. (1)

Prestressed concrete is basically concrete in which internal stresses of a member a suitable magnitude and distribution are introduced so that the stresses resulting from external loads are counteracted to a desired degree. In reinforced concrete members, the prestress is commonly introduced by tensioning the steel reinforcement. (2)

The tensile strength of plain concrete is only a fraction of its compressive strength and the problem of it being deficient in tensile strength appears to have been the driving factor in the development of the composite material known as "reinforced concrete". (2)

The development of early cracks in reinforced concrete due to incompatibility in the strains of steel and concrete was perhaps the starting point in the development of a new material like "prestressed concrete", The application of permanent compressive stress to a material like concrete, which is strong in compression but weak in tension, increases the apparent tensile strength of that material, because the subsequent application of tensile stress must first nullify the compressive prestress. (2)

- Prestressing can be either Pre-tensioned or Post-Tensioned based on method(system) of prestressing
  - *Pre-tensioning:-* A method of prestressing concrete in which the tendons are tensioned before the concrete is placed. In this method, the prestress is imparted to concrete by bond between steel and concrete. (2)
  - *Post-tensioning:-* A method of prestressing concrete by tensioning the tendons against hardened concrete. In this method, the prestress is imparted to concrete by bearing. (2)

Note: - Tendon is stretched element used in a concrete member of structure to impart pre- stress to the concrete. Generally, high-tensile steel wires, bars cables or strands are used as tendons. (2)

- Based on the interaction of tendon and concrete they may be classified as Bonded and Non-bonded

- *Bonded Prestressed concrete:* - Concrete in which prestress is imparted to concrete through bond between the tendons and surrounding concrete. Pre-tensioned members belong to this group (2).
- *Non-bonded Prestressed concrete:* - *A* method of construction in which the tendons are not bonded to the surrounding concrete. The tendons may be placed in ducts formed in the concrete members or they may be placed outside the concrete section. (2)

➢ Based the allowable tensile strength of concrete in the design they may be classified as
- *Full prestressing:* - Prestressed concrete in which tensile stresses in the concrete are entirely obviated at working loads by having sufficiently high prestress in the members. (2)
- *Limited or partial prestressing:* - The degree of prestress applied to concrete in which tensile stresses to a limited degree are permitted in concrete under working loads. In this case, in addition to tensioned steel, a considerable proportion of un-tensioned reinforcement is generally used to limit the width of crocks developed under service loads. (2)

➢ Based on the tendon profile they can be classifies as
- *Axial prestressing:* - Members in which the entire cross-section of concrete has a uniform compressive prestress. In this type of prestressing, the centroid, of the tendons coincides with that of the concrete section. (2)
- *Eccentric prestressing:* - A section at which the tendons are eccentric to the centroid, resulting in a triangular or trapezoidal compressive stress distribution. (2)
- *Concordant prestressing:* - Prestressing of members in which the cables follow a concordant profile, In the case of statically indeterminate structures, concordant prestressing does not cause any change in the support reactions. (2)
- *Non-distortional prestressing:* - In this type, the combined effect of the degree of pre- stress and the dead-weight stresses is such that the deflection of the axis of the member is prevented. In such cases the moments due to prestress and dead-weight exactly balance resulting only in an axial force in the member. (2)
- *Uniaxial, biaxial and triaxial prestressing*: - The terms refer to the cases where concrete is prestressed (i) in only one direction, (ii) in two mutually perpendicular directions and (iii) in three mutually perpendicular directions. (2)
- *Circular prestressing:* - The term refers to prestressing in round members, such as tanks and pipes... (2)

2.1.1.2 Advantage of Prestressing

➢ Tension and cracking under service loads may be avoided or reduced to a low level,

depending on the magnitude of the prestressing force. Avoiding of tensile cracks increase the efficiency of utilization the section compared to reinforced concrete. (3)

- Downward deflections of beams and slabs under service loads may be avoided or greatly reduced. (3)
- Fatigue resistance (i.e. the ability to resist the effect of repeated live loading due to, for instance, road and rail traffic) is considerably enhanced. (3)
- Enable Segmental forms of construction in which different members of a structure are constructed separate and joined to become a monolithic structure. (3)
- Very high strength steel may be used to form the tendons this results in lighter and slender members than is possible with reinforced concrete. The two structural features of prestressed concrete, namely high-strength concrete and freedom from cracks, contribute to the improved durability of the structure under aggressive environmental conditions. Prestressing of concrete improves the ability of the material for energy absorption under impact loads. (2)
- Beam and slab sections may be smaller than in reinforced concrete, due mainly to the capacity to reduce deflection. In the long-span range, prestressed concrete is generally more economical than reinforced concrete and steel. (3)
- Prestressed concrete possess improved resistance to shearing forces, due to the effect of compressive prestress, which reduces the principal tensile stress. The use of curved cables, particularly in long-span members, helps to reduce the shear forces developed at the support sections. (2)
- A prestressed concrete flexural member is stiffer under working loads than a reinforced concrete member of the same depth. However after the onset of cracking, the flexural behavior of a prestressed member is similar to that of reinforce concrete member. (2)

2.1.2 Materials for Prestressed Concrete

2.1.2.1 Concrete

Prestressed concrete requires concrete which has a high compressive strength at a reasonably early age, with comparatively higher tensile strength than ordinary concrete. Low shrinkage, minimum creep characteristics and a high value of Young's modulus are generally deemed necessary for concrete used for prestressed members. Many desirable properties, such as durability, impermeability and abrasion resistance, are highly influenced by the strength of concrete. The minimum 28-day cube compressive strength prescribed in the Indian standard code IS: 1343-1980 is 40 N/mm2 for pre-tensioned members and 30 N/mm2 for post- tensioned members. (2)

2.1.2.2 High Tensile Steel

For prestressed concrete members, the high-tensile steel used generally consists of wires, bars, or strands. The higher tensile strength is generally achieved by marginally increasing the carbon content in steel in comparison with mild steel. High-tensile steel usually contains 0.6 to 0.85 per cent carbon, 0.7 to 1 percent manganese, 0.05 percent of sculpture and phosphorus with traces of silicon. (2)

The following are standard tensile strength of tendon as N Krishna Raju put it in prestressed concrete fourth edition book by referring the Indian standard

Nominal Diameter (mm)	Tensile strength (minimum) (N/mm2)	Elongation (%)
2.50	2010	2.50
3.00	1865	2.50
4.00	1715	3.00
5.00	1570	4.00
7.00	1470	4.00
8.00	1375	4.00

Table 2-1 Standard Tensile Strength of prestressing bar

2.1.3 Prestress Loss

The initial prestress in concrete undergoes a gradual reduction with time from the stage of transfer due to various causes. This is generally referred to as 'loss of pre- stress'. A reasonably good estimate of the magnitude of loss of prestress is necessary from the point of view of design. Types of prestress loss in pre-tensioned and post-tensioned members are summarized as follows. (2)

Types of Loss No	**Pre-Tensioned Member**	**Post-tensioned Member**
	Elastic deformation of concrete	No loss due to elastic deformation if all wires are simultaneously tensioned, However if the wires are successively there will be loss
1	Relaxation of stress in steel	Relaxation of stress in steel
2	Shrinkage of concrete	Shrinkage of concrete
3	Creep of concrete	Creep of concrete
4		Friction
5		Anchorage Slip

Table 2-2 Types of loss in pre-tensioned and post-tensioned members

In addition to the above, there may be losses of prestress due to sudden changes in temperature, especially in steam curing of pre-tensioned units. The rise in temperature causes a partial transfer of prestress (due to the elongation of the tendons between adjacent units in the long-line process) which may cause a large amount of creep if the concrete is not properly cured. (2)
The details of the prestress loss will be presented in *section 2.3.2*

2.1.4 Tendon Profile (Eccentricity/e)

Eccentricity is the distance of the tendon from the neutral axis. The combination of prestress force and eccentricity greatly affect the deflection of a member. In fact designing prestressed member is all about finding the combination of prestressing force and the corresponding eccentricity at a section for a given loading.

Once the section modulus of a beam is defined the next step is determine what combination of prestress force, P and eccentricity, e, to use at that section. Taking each stress limit in turn: (4)

a. Tensile Stress at transfer (top section)

$$-ftt \leq \frac{Pi}{Ac} - \frac{Pi * e}{Zt} + \frac{Mg}{Zt} \qquad \text{Equation 2-1}$$

Solving for e

$$e \leq \frac{Zt * ftt}{Pi} + \frac{Zt}{Ac} + \frac{Mg}{Pi} \qquad \text{Equation 2-2}$$

Solving for p

$$pi \geq \frac{-ftt + Mg/Zt}{1/Ac - e/Zt}$$

$$\frac{1}{Pi} \geq \frac{1/Ac - e/Zt}{-ftt - Mg/Zt} \qquad \text{Equation 2-3}$$

b. Compressive Stress at Transfer (Bottom Section)

$$fct \geq \frac{Pi}{Ac} + \frac{Pi * e}{Zb} - \frac{Mg}{Zb} \qquad \text{Equation 2-4}$$

Solving for e

$$e \leq \frac{Zb * fct}{Pi} - \frac{Zb}{Ac} + \frac{Mg}{Pi} \qquad \text{Equation 2-5}$$

Solving for p

$$pi \leq \frac{fct + Mg/Zb}{1/Ac + e/Zb}$$

$$\frac{1}{Pi} \geq \frac{1/Ac + e/Zb}{fct + Mg/Zb}$$ Equation 2-6

c. *Compressive Stress in Service (Top section)*

$$fcw \geq \frac{Pe}{Ac} - \frac{Pe * e}{Zt} + \frac{Mtot}{Zt}$$ Equation 2-7

Solving for e

$$e \geq -\frac{Zt * fcw}{\eta Pi} + \frac{Zt}{Ac} + \frac{Mtot}{\eta Pi}$$ Equation 2-8

Solving for p

$$pi \leq \frac{fcw - Mtot/Zt}{\eta * (1/Ac - e/Zt)}$$

$$\frac{1}{Pi} \geq \frac{\eta * (1/Ac - e/Zt)}{fcw - Mtot/Zt}$$ Equation 2-9

d. *Tensile Stress in Service (Top section)*

$$ftw \leq \frac{Pe}{Ac} + \frac{Pe * e}{Zb} - \frac{Mtot}{Zb}$$ Equation 2-10

Solving for e

$$e \geq -\frac{Zb * ftw}{\eta Pi} - \frac{Zb}{Ac} + \frac{Mtot}{\eta Pi}$$ Equation 2-11

Solving for p

$$pi \geq \frac{-ftw + Mtot/Zb}{\eta * (1/Ac + e/Zb)}$$

$$\frac{1}{Pi} \leq \frac{\eta(1/Ac + e/Zb)}{-ftw + Mtot/Zb}$$ Equation 2-12

Those equations solved for e gives the limiting zone of the eccentricity.

I.e. Equation 2.2 and 2.5 gives the maximum allowable limit, whereas the Equation 2.8 and 2.11 gives the minimum range of the limiting zone.

Note: - The effect of tendon profile in deflection will be discussed in section 2.3.6

Using the equations solved for p and 1/pi vs. e we get the famous Magnel Diagram (5)

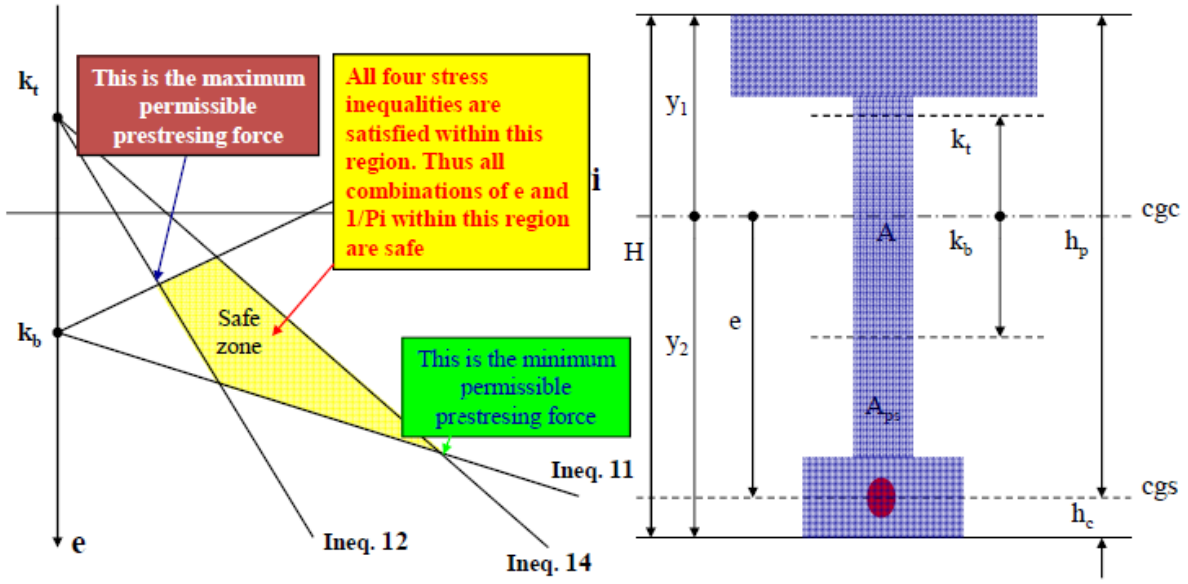


Figure 2-1 Magnel Diagram (5)

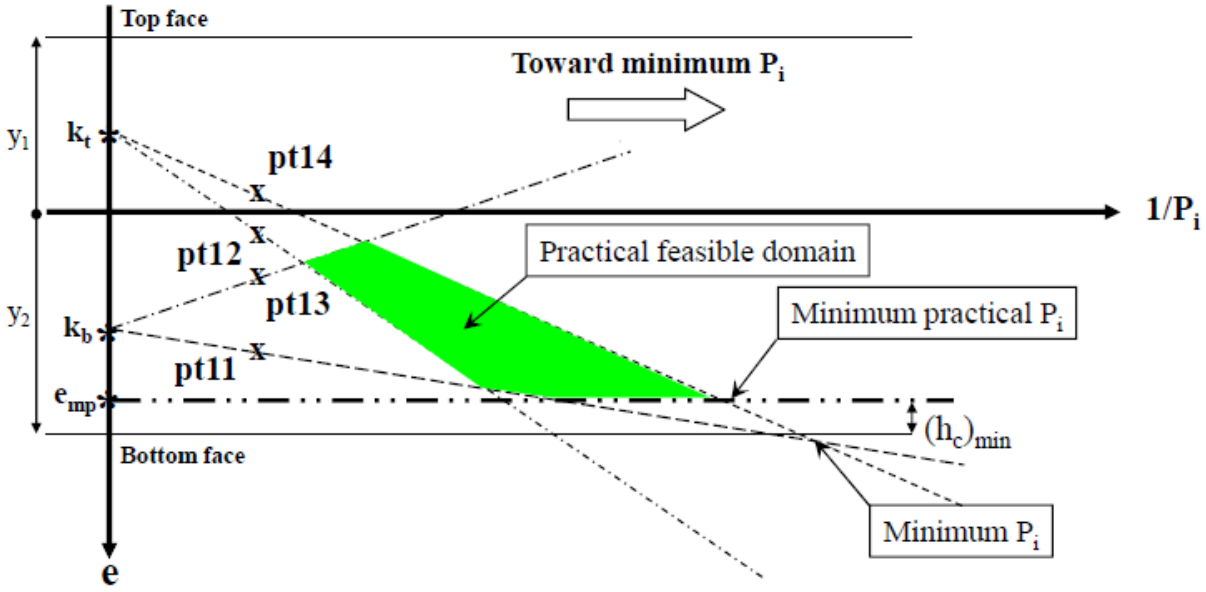


Figure 2-2 Critical Points on Magnel Diagram (5)

2.1.5 Minimum Prestressing Force and Maximum Eccentricity

As we can see from the above diagram the minimum possible prestressing (pi_{min}) force is when 1/pi is maximum in the allowable range. And at that point there will be the corresponding Maximum eccentricity (e_{max}). This condition met when the line from case A and the line from case D cross each other.

Case A Tensile stress at transfer (Top Section)

Case D Tensile stress at Service (Bottom Section)

Equating *Eqn 2-3* and *Eqn 2-12*

$$\frac{-Zb * ftw}{\eta Pi} - \frac{Zb}{Ac} + \frac{Mtot}{\eta Pi} = \frac{Zt * ftt}{Pi} + \frac{Zt}{Ac} + \frac{Mg}{Pi}$$ **Equation 2-13**

Solving for pi

$$Pimin = Ac * \frac{1/\eta * (-Zb * ftw + Mtot) - (Zt * ftt + Mg)}{(Zt + Zb)}$$ **Equation 2-14**

Solving for e

$$emax = \frac{Zt * Zb * (-ftw/\eta + ftt + Mtot/(\eta * Zb) + Mg/Zt)}{Ac * (-Zb * ftw + Zt * ftt + Mtot/\eta + Mg)}$$ **Equation 2-15**

Similarly Maximum Possible prestressing force and minimum eccentricity can be found using case B and C.

Case B Compressive stress at transfer (Bottom Section)

Case C Compressive Stress at Service (Top Section)

Solving for pi

$$Pimax = Ac * \frac{1/\eta * (Zt * fcw - Mtot) + (Zb * fct + Mg)}{(Zt + Zb)}$$ **Equation 2-16**

2.2 Deflection in Reinforced Concrete Beam

With the advancement of concrete technology, plus the use of higher strength concrete with steel reinforcement has permitted more material efficient reinforced concrete designs as prestressed members enables to produce shallower and smaller sections. Smaller sections have less stiffness that can result in larger deflections. (6)

Consequently, precise determination of short-term and long-term deflection of these structures has become more critical. In many structures, deflection rather than stress limitation is the controlling factor. Deflection computations determine the proportioning of many of the structural system elements. As prestressed concrete is hardly different from reinforced ones, understanding and analyzing of the deflection of reinforced concrete members should be the first step towards that of the analysis of deflection of prestressed concrete member. (6)

2.2.1 Material properties

The principal material parameters that influence concrete deflection are (6)

- modulus of rupture
- modulus of elasticity
- Creep, and shrinkage

2.2.1.1 Concrete modulus of rupture-

It is the flexural tensile strength of concrete. .AC1 318 (1989) recommends the following equation for computing the modulus of rupture of concrete with different density

$fr = 0.623 * \lambda\sqrt{fc'}$ MPa	Equation 2-17

Where λ = 1.0 for normal density concrete 2325 to 2400 kg/m3
= 0.85 for semi low-density 1765 to 2325 kg/m3
= 0.75 for low-density concrete 1445 to 1765 kg/m3

The above equation is to be used for low-density concrete when the tensile splitting strength, *fct*, is not specified. Otherwise, it should be modified by substituting *fct*/6.7 for $\sqrt{fc'}$, but the value of *fct/6.7* should not exceed $\sqrt{fc'}$ (6)

ACI Committee 435 (1978) recommended the following equation for computing the modulus of rupture of concrete with densities (*wc*) in the range of 1445 kg/m3 to 2325 kg/m3. This equation yields higher values of *fr* (6)

$fr = 0.01 * \sqrt{wc * fc'}$, MPa	Equation 2-18

According to ACI 363 (1984), the modulus of rupture of both low-density and normal density high-strength concretes (more than 42 MPa) ranges between $0.623\sqrt{fc'}$ and $0.997\sqrt{fc'}$. ACI 363

(1992) recommended the following equation for the prediction of the modulus of rupture of normal density concretes having compressive strengths of 21 MPa to 83 MPa. (6)

$fr = 0.971\sqrt{fc'}$	Equation 2-19

The degree of scatter in results in the above equations is indicative of the uncertainties in predicting computed deflections of concrete members. (6)

2.2.1.2 Concrete modulus of elasticity –

It is the slope of stress and strain diagram or simply the ratio of stress to strain. The modulus of elasticity of concrete is mainly influenced by the concrete materials and proportions used. As the compressive strength of concrete increases the modulus of Elasticity also increases, since the slope of the ascending branch of the stress-strain diagram becomes steeper for higher-strength concretes, but at a lower rate than the compressive strength. The value of the secant modulus of elasticity for normal-strength concretes at 28 days is usually around 28,000 MPa, whereas for higher-strength concretes, the modulus of elasticity can range between 49,000 to 56,000 MPa. These higher values of the modulus can be used to reduce short-term and long term deflection of flexural members since the compressive strength is higher, resulting in lower creep levels. (6)

Normal strength concretes are those with compressive strengths up to 42 MPa while higher strength concretes achieve strength values beyond 42 MPA. ACI 435 (1963) recommended the following expression for computing the modulus of elasticity of concretes with densities in the range of 1445 kg/m3 to 2325 kg/m3 based on the secant modulus at 0.0.0031fc' intercept (6)

$E = 0.043 * wc^{1.5} * \sqrt{fc'}$ MPa	Equation 2-20
For normal weight concrete around (150pcf)/2400Kg/m3- $Ec = 57000 * \sqrt{fc'}$ psi $Or\ Ec = 4734 * \sqrt{fc'}$ MPa	Equation 2-21

However, as the strength of concrete increases, the ***Ec*** value determined by this formula will be lower than the actual value. Therefore ACI 363 (1984) recommended the following modified expression for normal weight concrete: (6)

$EC = 3321 * \sqrt{fc'} + 6895, \text{Mpa}$	Equation 2-22

2.2.1.3 Steel reinforcement modulus of elasticity-

AC1 318 specifies using the value Es = 200 x 10^6 MPa (29 x 10^6 psi) for the modulus of elasticity of non prestressed reinforcing steel. (6)

2.2.1.4 Concrete creep and shrinkage-

Deflections are also a function of the age of concrete at the time of loading due to the long-term effects of shrinkage and creep which significantly increase with time. A CI 435

(1978) suggested that the average values for C_u and $(\epsilon_{sh})_u$ can be estimated as 1.60 and 400 x 10^{-6} respectively. (6)

Where C_u = ultimate creep coefficient of concrete

$(\epsilon_{sh})_u$ = ultimate shrinkage strain of concrete

These values correspond to the following conditions:

- 70 percent average relative humidity
- age of loading, 20 days for both moist and steam cured concrete
- Minimum thickness of component, 6 in. (152 mm)

Table 2.1 of ACI 435 gives creep and shrinkage ratios at different times after loading

The coefficient for creep at time t (days) after load application, is given by the following expression: (6)

$$Ct = \left(\frac{t^{0.6}}{10 + t^{0.6}}\right) * Cu \qquad \text{Equation 2-23}$$

Where $Cu = 2.35 * \gamma CR$

$\gamma CR = Khc * Kdc * Ksc * Kfc * Kacc * Ktoc$ = 1 for standard condition

Graphic representations and general equations for the modification factors (*K*-values) for non-standard conditions are given in Fig. 2.1 of ACI 435.

For moist-cured concrete, the free shrinkage strain which occurs at any time t in days, after 7 days from placing the concrete (6)

$$(\epsilon sh)t = \left(\frac{t}{35 + t}\right) * (\epsilon sh)u \qquad \text{Equation 2-24}$$

And for steam cured concrete, the shrinkage strain at any time in days, after l-3 days from placing the concrete

$$(\epsilon sh)t = \left(\frac{t}{55+t}\right) * (\epsilon sh)u \qquad \text{Equation 2-25}$$

Where $(\epsilon sh)u\ max = 780E - 6 * \gamma sh$

$\gamma sh = Khs * Kds * Kfs * Kacs * Kbs = 1$ For standard condition.

Graphic representations and general equations for the modification factors (*K*-values) for non-standard conditions are given in Fig. 2.1 of ACI 435

Except for age of concrete at load application, the standard conditions for both creep and shrinkage are (6)

a) Age of concrete at load applications = 3 days (steam), 7 days (moist)
b) Ambient relative humidity = 40 percent
c) Minimum member thickness = 6 in. (150 mm)
d) Concrete consistency = 3 in. (75 mm)
e) Fine aggregate content = 50 percent f) Air content = 6 percent

2.2.2 Short-term Deflection

2.2.2.1 Uncracked members - Gross moment of Inertia, Ig

When the maximum flexural moment at service load in a beam or a slab causes a tensile stress less than the modulus of rupture, fr no flexural tension cracks develop at the tension side of the concrete element if the member is not restrained or the shrinkage and temperature tensile stresses are negligible. In such a case, the effective moment of inertia of the uncracked transformed section, (It) is applicable for deflection computations. However, for design purposes, the gross moment of inertia (Ig) neglecting the reinforcement contribution, can be used with negligible loss of accuracy. (6)

The elastic deflection for non-cracked members can thus be expressed in the following general form

$$\delta = K * \frac{M * l^2}{Ec * Ig}$$ **Equation 2-26**

Where: - K is a factor that depends on support fixity and loading conditions. M is the maximum flexural moment along the span. Ec is the modulus of elasticity. (6)

2.2.2.2 Cracked members-*Effective moment of inertia I_e*

Tension cracks occur when the imposed loads cause bending moments in excess of the cracking moment, thus resulting in tensile stresses in the concrete that are higher than its modulus of rupture. The cracking moment, Mcr computed as follows.

$$MCr = \frac{fr * Ig}{Yt}$$ **Equation 2-27**

Where yt is the distance from the neutral axis to the tension face of the beam, and fr is the modulus of rupture of the concrete. (6)

Cracks develop at several sections along the member length. While the cracked moment of inertia, Icr applies to the cracked sections, the gross moment of Ig inertia, applies to the uncracked concrete between these sections. (6)

the initial deflections δi occurring in a beam or a slab after the maximum moment Ma has exceed the cracking moment can be evaluated using the effective moment of inertia Ie.

To find Ie according to ACI 318 – 89

a*. *Simply Supported beams - the effective moment of inertia Ie proposed by Branson

$$Ie = (\frac{Mcr}{Ma})^3 * Ig + [1 - (\frac{Mcr}{Ma})^3] * Icr \leq Ig$$

$$\text{Ie} = \text{Icr} + (\frac{\text{Mcr}}{\text{Ma}})^3 * (\text{Ig} - \text{Icr}) \leq \text{Ig}$$ **Equation 2-28**

The two moments of inertia Ig and Icr are based on the assumption of bilinear load-deflection

behavior. Ie provides the transition between the upper bounds of Ig and lower bounds of Ie. The cracking moment of inertia, Icr can be obtained from Fig. 2.3 of ACI 435R – 95 (6)

b. Continuous beams – Ie, effective moment of inertia taken as the average values obtained from the above equation for critical positive and negative moment section i.e. at mid span and the support section. Accordingly ACI 318 – 89 given the following weighted average formula. (6)

For beams continuous on one end only

$Ie = 0.85 * (Iem) + 0.15 * (Ie\ 1)$	Equation 2-29

For beams continuous on both ends

$Ie = 0.70 * (Iem) + 0.15\ (Ie1 + Ie2)$	Equation 2-30

Where m refers to mid span and 1 and 2 refers to the two end span.

2.2.3 Long – Term Deflection

2.1.3.1 ACI method. Time dependent deflection of one way flexural member is due to creep and shrinkage and can be calculated by applying a multiplier, λ, to the elastic deflection ACI–318. (7)

$\lambda = \frac{\xi}{1 + 50\rho'}$	Equation 2-31

ρ' = reinforcement ratio for non-prestressed compression steel reinforcement

ξ = time dependent factor, from (ACI 318, 1989)

Therefore the total long term deflection is

$\delta lt = \delta l + \lambda t * \delta sus$	Equation 2-32

δl = initial live load deflection

δsus = initial deflection due to sustained load

λt = time dependent multiplier for a defined duration time.

Due to lower creep strain characteristics, high strength concrete members exhibit less sustained load deflection than those of low strength concrete members.

ACI 365R – 95 Introduce a modifier μ to account for material and section property (6)

$0.7 \leq \mu = 1.3 - 0.00005fc' \leq 1.0$		Equation 2-33
Accordingly	$\lambda = \frac{\mu\xi}{1 + 50\mu\rho'}$	Equation 2-34

2.2.4 Temperature-induced deflections

In unrestrained flexural member deflection occur when there is a temperature gradient between the opposite faces of the member. If the member is restrained from deforming under the action of temperature changes, internal stresses are developed even if the temperature is uniform on the opposite faces. Cracking that occurs when tensile stresses exceed the concrete tensile strength reduces the flexural stiffness of the member and results in increased deflections under subsequent loading. (6)

2.3 Deflection of Prestressed Concrete Beam

In the design of prestressed concrete structures, the deflections (i.e. both short term and long term) are usually the governing criteria in the determination of the required member sizes. The use of high strength materials which result in slender members leads to higher deflection. And because of this deflection in prestressed member should be carefully examined. (6)

2.3.1 Prestressing Reinforcement

Because of the creep and shrinkage which occurs in concrete, effective prestressing can only be achieved by using high strength steel. (6)

2.3.1.1 Modulus of elasticity-

In considering time-dependent deflections resulting from shrinkage and creep at the level of the prestressing steel, it is important to have a reasonably good estimate of the modulus of elasticity of the prestressing reinforcement. However for short term deflection, since the cross sectional area of the reinforcing tendon in a beam is insignificant the deflection is calculated based on the gross moment of inertia of concrete. (6)

since the reinforcement is rarely stressed into the inelastic range in calculating deflections under working loads, it is sufficient to use the modulus of elasticity of the prestressing reinforcement rather than to be concerned with the characteristics of the entire stress-strain curve. According to PCI Design hand book the modulus of elasticity of prestressed reinforcement E = 28.5E6 psi (196.5Gpa). (6)

2.3.1.2 Steel relaxation-

Stress relaxation in prestressing steel is the loss of prestress that occurs when the wires or strands are subjected to essentially constant strain over a period of time. As time goes the loss of prestress force increases. This decreasing of the magnitude of prestress force depends not only on the duration of the sustained prestressing force, but also on the ratio fpi/fpy of the initial prestress to the yield strength of the reinforcement. Such a loss in stress is termed intrinsic stress relaxation. (6)

2.3.2 Loss of prestress

2.3.2.1 Elastic shortening loss –

A concrete element shortens when a prestressing force is applied to it due to the axial compression imposed. Simultaneously the tendons that are bonded to the adjacent concrete shorten because of this they lose part of the prestressing force that they carry. In pretensioned members, this, force results in uniform longitudinal shortening, which in turn results in a prestress loss. In posttensioned beams, elastic shortening is zero if all tendons are simultaneously

jacked and becomes half the value of that of the pretensioned case if several sequential jacking steps are applied. (6)

2.3.2.2 Loss of prestress due to creep of concrete-

The deformation or strain resulting from creep losses is a function of the

- Magnitude of the applied load and its duration
- Properties of the concrete including its mix proportions
- Curing conditions
- The size and shape of the element
- Its age at first loading and the environmental conditions. (6)

The creep stress/strain relationship is linear and one can relate the elastic strain (Єel) to the creep strain (Єcr).

According to ACI 435 – 95 the ultimate creep coefficient Cu = Єcr/Єel

As we discussed in the reinforced concrete section (6)

$$Ct = \left(\frac{t^{0.6}}{10 + t^{0.6}}\right) * Cu$$ **Equation 2-35**

The value of C, usually ranges between 2 and 4, with an average of 2.35 for ultimate creep.

The prestress loss due to creep is defined as (6)

$$\Delta fpcr = Cu \frac{EPS}{EC} fcs$$ **Equation 2-36**

Where fcs is the stress in the concrete at the level of the prestressing tendon centroid. In post tension tendon members, the loss can be considered essentially uniform along the whole span. Therefore an average value of the concrete stress between the anchorage points can be used for calculating the creep in post-tensioned members.

Accordingly A modified ACI-ASCE expression for creep loss is

$$\Delta fpcr = Kcr \frac{EPS}{EC}(fcs' - fcsd)$$ **Equation 2-37/**

Where Kcr = 2.0 for pretensioned member

= 1.60 for post tensioned

fcs' = stress in concrete at the *cgs* level of the reinforcement immediately after transfer

fcsd = stress in concrete at the *cgs* level of the reinforcement due to all superimposed dead loads applied after prestressing is accomplished. (6)

2.3.2.3 Loss of prestress due to shrinkage of concrete –

Factors that affect the magnitude of shrinkage of concrete is pretty much the same as that of creep. In addition type of aggregate and cement also affect magnitude of shrinkage of concrete plus magnitude of the applied load has no effect on it.

According to ACI 365 Approximately 80 percent of shrinkage takes place in the first year of life of the structure. The average value of ultimate shrinkage strain in both moist-cured and stream-cured concrete is given as 780E-6 mm/mm. This average value is affected by the duration of initial moist curing, ambient relative humidity, volume-surface ratio, temperature; and concrete composition. To take such effects into account, the average value of shrinkage strain should be multiplied by adjusting for relative humidity at volume-to-surface ratio a correction factor γsh (6)

$$Єsh = (780E - 6) * \gamma sh$$ **Equation 2-38**

PCI handbook gives nominal ultimate shrinkage strain Єshu = 820E-6 mm/mm (8)

The loss in prestressing in pretensioned members is

$$\Delta fpsh = Єsh * Eps$$ **Equation 2-39**

For post tensioned members

$$\Delta fpsh = (8.2E - 6) * Ksh * Eps \left(1 - 0.06\frac{v}{S}\right)(100 - RH)$$ **Equation 2-40**

Where Ksh = 1.0 for pretensioned members

Ksh value for post tensioned member is give in table 3.1 of ACI 365 – 95

V/S is the volume surface ratio.

Єsh is the shrinkage strain after adjusting for relative humidity at volume-to-surface ratio V/S (6)

Adjustment of shrinkage losses for standard conditions as a function of time t in days (6)

- Moist curing, after seven days:

$$(Єsh)t = \frac{t}{35 + t}(Єsh)u$$ **Equation 2-41;**

- Steam curing, after one to three days

$$(Єsh)t = \frac{t}{55 + t}(Єsh)u$$ **Equation 2-42**

2.3.2.4 Friction losses in post-tensioned beams -

Loss of prestressing occurs in post-tensioned members due to friction between the tendons and the surrounding concrete ducts.

The magnitude of this loss is a function of

- The curvature effect (the tendon alignment and form)
- The wobble effect (local deviation in alignment)

Section 186.2 of ACI 318-89 give the friction coefficients that apply to the friction loss in the various types of prestressing wires and tendons. (6)

2.3.2.5 Total Loss allowed for design-

It is normal practice in the design of prestressed concrete members to assume total loss of stress as a percentage of the initial stress and provide for this in the design computations.

Assuming temporary overstressing is done to reduce relaxation, to compensate for friction and anchorage losses the following table is provided as a summary of average loses in *prestressed concrete, fourth edition by N Krishna Raju.* (2)

Types of losses	percentage of losses	
	Pre-tensioning	Post-tensioning
Elastic shortening and bending of concrete	4	1
Creep of concrete	6	5
Shrinkage of concrete	7	6
Creep in steel	8	8
Total	**25**	**20**

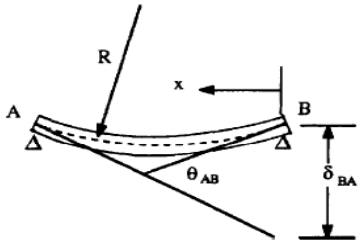
$$n(reduction\ factor) = \frac{fpi}{fpe}$$

The value of n is generally taken as 0.75 for pre-tensioning and 0.8 for post-tensioning. (2)

2.3.3 General approach to curvature and Deflection

The curvature at any section is defined as Φ = 1/R where R is the radius of the curve. And from the famous Mohr circle (moment area theorem) the two equations can be derived. (6)

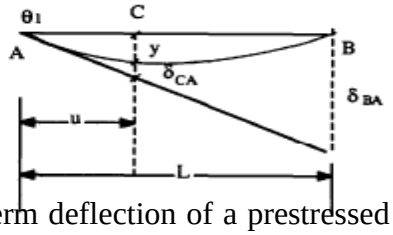
$(Rotation)\ \theta AB = \int_A^B \Phi dx$	Equation 2-43
$(Deflection)\delta BA = \int_B^A \Phi x dx$	Equation 2-44



When the material is linearly elastic Φ can be replaced by M/EI

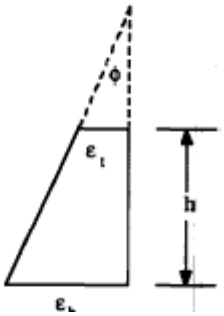
To calculate the deflection at any point

$\theta = \delta BA/L$	Equation 2-45
$\delta BA = \int_B^A \Phi x dx$	Equation 2-46
$y = \theta 1 u - \delta CA$	Equation 2-47



Critical variables that affect the magnitude of short term deflection of a prestressed beam are the magnitude of the strain or stress gradient or the curvature at a section and its variation along the span. Which are a function of

- the magnitude and distribution of the load
- the magnitude and eccentricity of the prestress,
- the length of the span,
- the size and configuration of the cross section,
- boundary conditions
- and the properties of the concrete. (6)

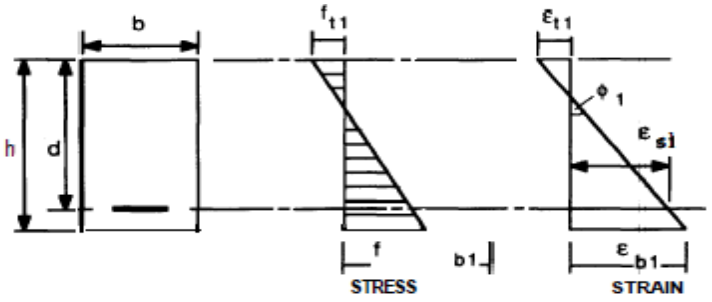


Curvature $\Phi = (\epsilon b - \epsilon t)/h = M/(EcI)$ (2.26)

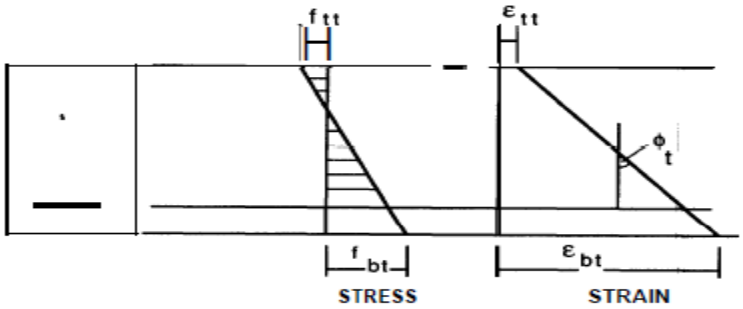
2.3.3.1 Beams subjected to prestressing only-

The stress at any level is given by (6)

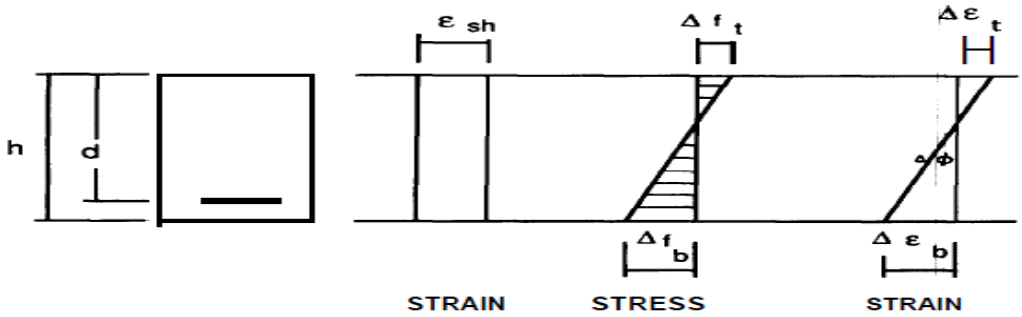
$f = \frac{P}{A} \pm \frac{My}{I}$	Equation 2-48
$M = P * e$	Equation 2-49



1. *Stress and strain distribution immediately after application of prestress*



2. *Stress and strain distribution at a time* t *after initial application of prestress*



3. *Stress and strain due to shrinkage*

The distribution of shrinkage strain causes a reduction in the reinforcement strain which corresponds to a reduction in the prestress. The loss in prestress causes a change in the stress

distribution over the depth of the section. (6)

$\Delta\Phi = \frac{\Delta\epsilon b - \Delta\epsilon t}{h}$	Equation 2-50

The effect of the relaxation losses in the steel reinforcement is quite similar to that of shrinkage. The effects of the creep of the concrete are not as simple, since the reduction in steel stress causes changes in the rate of creep strain. The change in strain caused by creep change the strain distribution. This change involves a contraction at the level of the steel, hence, a reduction in prestress. The reduction in prestress caused by creep, shrinkage, and relaxation decreases the normal stress, which in turn reduces the rate of creep. (6)

2.3.3.2 Beams subjected to prestressing and external loads

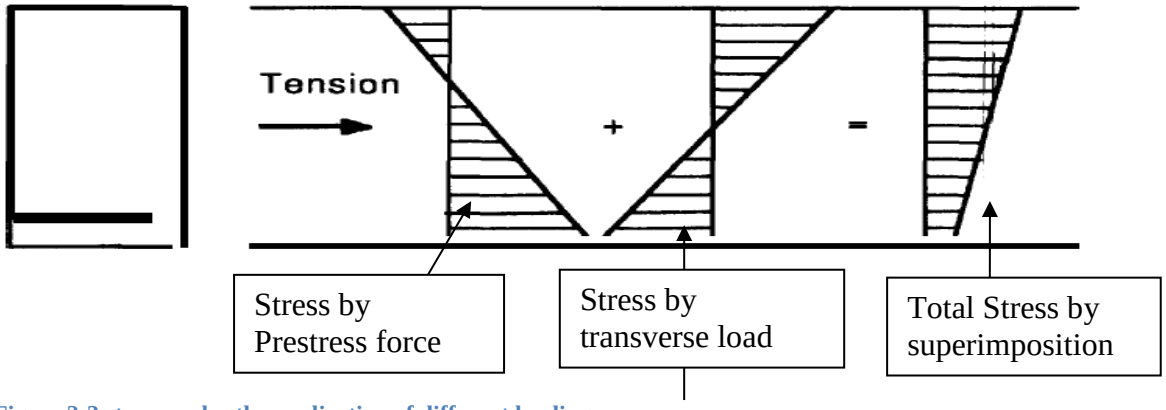


Figure 2-3 stress under the application of different loading

In the above diagram it is assumed that neither concrete nor reinforcement is strained into the inelastic range.

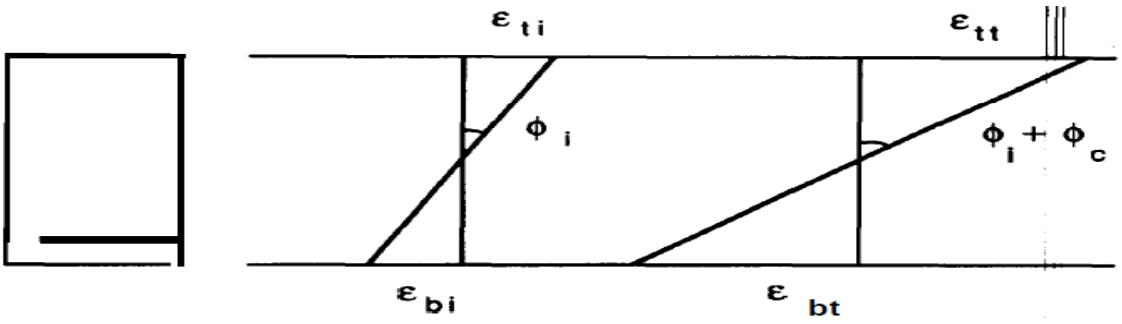


Figure 2-4 strain under the application of different loading

$\Phi i = \frac{\epsilon bi - \epsilon ti}{h}$	Equation 2-51	Strain distribution by transverse load only

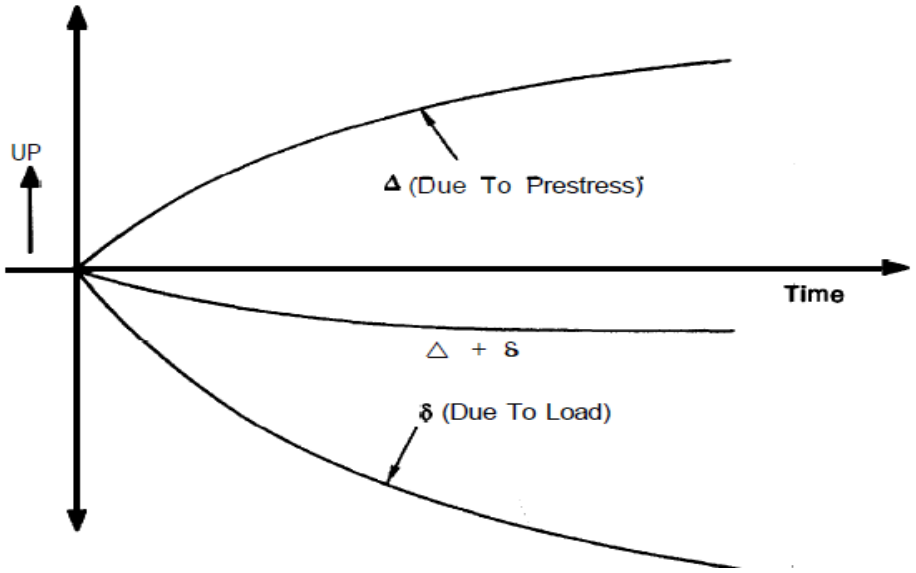


Figure 2-5 Deflection versus time due to prestress and transverse loads

The changes in the curvature or in the deflection of the beam caused by the combined prestress and the transverse load are determined by superposition.

2.3.3.3 Moment-curvature relationship

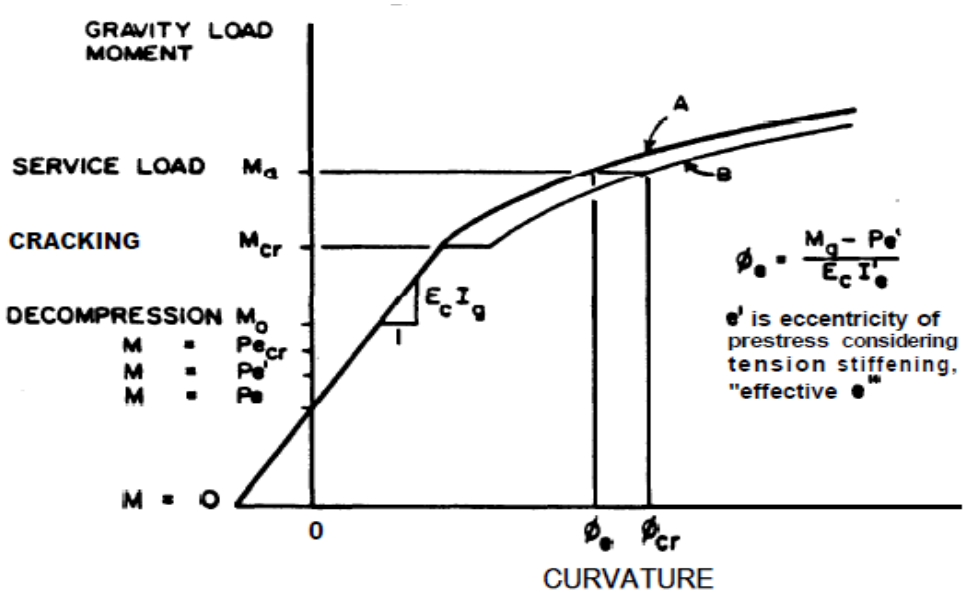


Figure 2-6 Moment curvature diagram according to ACI 435

Concrete can sustain tensile stresses and contribute to the carrying capacity of a member until cracking occurs at a moment Mcr. A moment Ma, larger than the cracking moment **Mcr** produces curvature that can be defined as follows (6)

$$\Phi cr = \frac{Ma - P * ecr}{h}$$	Equation 2-52

ecr is the eccentricity of prestress force (p) relative to the centroid of the cracked section. The drop in rigidity due to cracking is represented by the horizontal line at the Mcr level. For the prestressed section, both Icr and ycr (and in turn ecr) are dependent on the loading level, with the M -Φ becoming nonlinear after cracking. It is important to note that the shift in the centroid of the cross section upon cracking results in larger prestressing force eccentricity, ecr than the uncracked member eccentricity. This fact is particularly significant in flanged members, such as double tees which are characterized by the relatively low steel area ratio (pf) and because concrete tensile strength is not zero, cracking does not extend to the neutral axis. In addition, uncracked concrete which exists between cracks in the tension zone, contributes to the stiffness of the member (tension stiffening). Taking this into account, the M-Φ diagram becomes continuous, as indicated by line A. (6)

2.3.4 Short-term deflection and camber evaluation in prestressed beams

2.3.4.1 Uncracked Members –

When a concrete section is subjected to a flexural stress which is lower than the modulus of rupture of concrete fr, the section is assumed to be uncracked and thus its behavior is linear. Under this condition, the deflection is calculated by the basic principles of mechanics of elastic structures. In prestressed concrete, the immediate deflection, or camber, due to the effects of initial prestressing Pi and member self-weight is generally in the elastic uncracked range. Therefore the elastic formulas presented in the previous discussion could be used to calculate the instantaneous deflection of the member. Pi is the prestress force after the initial loss due to anchorage set, elastic shortening and relation between jacking and release time. (6)

For uncracked section we use the gross moment of inertia Ig for pretensioned member and the net moment of inertia In for post tensioned member with unbonded tendons.

2.3.4.2 Cracked Members –

Effective Ie method--In prestressed concrete members, cracks can develop at several sections along the span under maximum load. The cracked moment of inertia Icr applies at cracked sections while the gross moment of inertia applies in between cracks. Therefore we use an effective moment of inertia (Ie), which has equivalent effect for the whole section. (6)

$Ie = Icr + (\frac{Mcr}{Ma})^3 * (Ig - Icr) \leq Ig$	Equation 2-53

Where

$\left(\frac{Mcr}{Ma}\right) = \left(1 - \frac{ftl - fr}{fl}\right)$	Equation 2-54

λ = reduction factor for sand light weight concrete 0.85 and 0.7 for all light weight concrete.

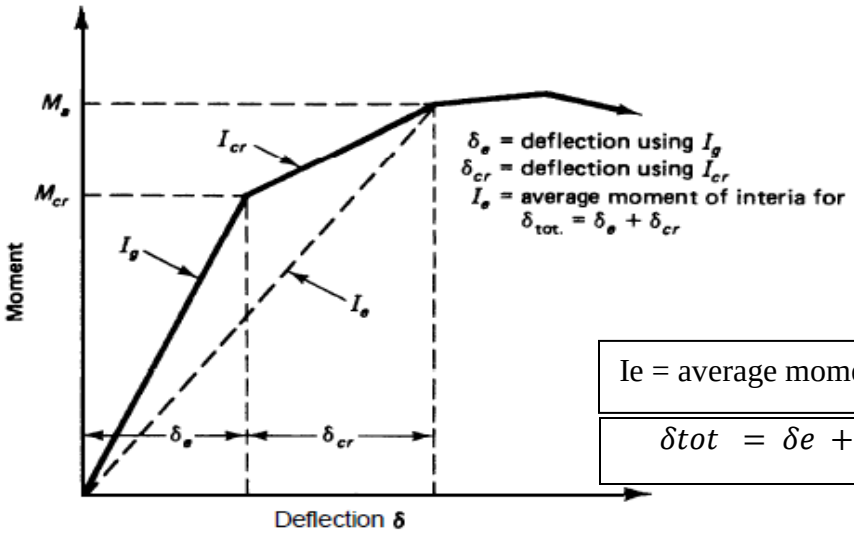
fr= modulus of rupture = $7.5\lambda\sqrt{fc}$

ftl = total calculated stress in member.

fl = calculated stress due to live load.

yt = distance from the neutral axis to tensile face

2.3.4.3 Bilinear computation method –



Ie = average moment of inertia

$\delta tot = \delta e + \delta cr$	Equation 2-55 .

Bilinear moment - deflection relationship

Computation of deflection in the cracked zone in the bonded tendon beams is based on the transformed section whenever the tensile stress ft in the concrete exceeds the modulus of rupture fr, hence, δcr is evaluated using the transformed Icr utilizing the contribution of the reinforcement in the bilinear method of deflection computation. The cracked moment of inertia can be calculated by the PCI approach for fully prestressed members. (8)

$Icr = np * Aps * dp^2 * (1 - 1.6\sqrt{np * \rho p}) = C * b * dp^3$	Equation 2-56

Where $np = Eps/Ec$ and $\rho p = Aps/(b * d)$

C is acofficient can be obtained from table 3.3 of ACI 435

In partial presterssing (where non – pestressed reinforcement is used to carry the tensile stress)

$$Icr = (np * Aps * dp^2 + ns * As * d^2) * \left(1 - 1.6\sqrt{np * \rho p + ns * \rho}\right)$$ Equation 2-57.

Where $ns = Es/Ec$ for non prestressed steel

d = effective depth to the center of mild steel.

2.3.4.4 Incremental moment-curvature method –

The analysis is performed assuming two stages of behavior, namely, elastic uncracked stage and cracked stage. The cracked moment of inertia can be calculated more accurately from the moment-curvature relationship along the beam span and from the stress and, consequently, strain distribution across the depth of the critical sections. (6)

For strain Єci at first cracking

$$\Phi cr = \frac{Ecr}{c} = \frac{M}{Ec * Icr}$$ Equation 2-58

Єcr is the strain at extreme compression fiber at first cracking

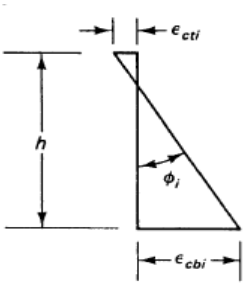
M is the net moment

f is the stress at extreme compression fiber.

Then rearranging the above equation

$$Icr = \frac{M}{Ec * Icr} = \frac{Mc}{f}$$ Equation 2-59

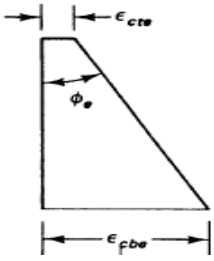
i. Initial prestress



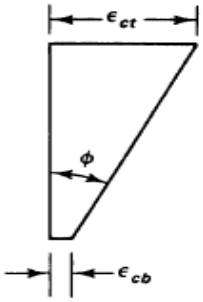
$$\Phi i = \frac{Ecbi - Ecti}{h}$$ Equation 2-60

ii. effective prestress after loss

$$\Phi e = \frac{Ecbe - Ecti}{h}$$ Equation 2-61



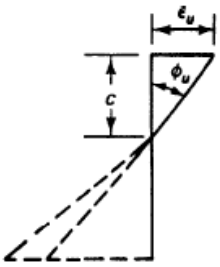
iii. service load



$$\Phi = \frac{Ect - Ecb}{h}$$ Equation 2-62

iv. failure

$\Phi u = \frac{\epsilon u}{c}$	Equation 2-63



Note: - a plus sign for tensile strain and a minus sign for compressive strain

It is important to recognize that curvatures Φ at various load levels and at different locations along the span can be specifically evaluated if the beam is divided into enough small segments. (6)

2.3.5 Long-term deflection and camber evaluation in prestressed Beams.

As time goes the deflection in prestressed member increase mainly because of the effects of creep and shrinkage of concrete and relaxation of the prestressing reinforcement. The determination of long-term camber and deflection in prestressed members is more complex than for non prestressed members due to the following factors:

1. The long-term effect of the variation in prestressing force resulting from the prestress losses.
2. The increase in strength of the concrete after release of prestress and because the camber and deflection are required to be evaluated at time of erection. (6)

2.3.5.1 PCI Multipliers Method

ACI -435 recommended PCI multiplier method for calculating long term deflection. That is the most popular method for predicting time-dependent camber and deflection of precast, prestressed member is the set of multipliers given in table 8.7.1-1 of PCI Bridge manual 1997. The use of this method is fairly straight forward. First, short term deflection are calculated using conventional elastic analysis techniques. And these values are multiplied by the appropriate factor selected from table 8.7.1-1 (of PCI Bridge manual 1997) to determine the deflection that will occur as a result of time-dependent behavior. (8)

-PCI multipliers C, for long-term camber and deflection		
	Without composite topping	With composite topping
At Erection		
Deflection (downward) component-apply to the elastic deflection due to the member weight at release of prestress	1.85	1.85
Camber (upward) component-apply to the elastic camber due to pre- stress at the time of release of prestress	1.8	1.8
Final		
Deflection (downward) component-apply to the elastic deflection due to the member weight at release of prestress	2.7	2.4
Camber (upward) component-apply to the elastic camber due to pre- stress at the time of release of prestress	2.45	2.2
Deflection (downward)--apply to the elastic deflection due to the superimposed dead load only	3	3
Deflection (downward)-apply to the elastic deflection caused by the composite topping		2.3

Table 2-3 PCI suggested multipliers to be used for calculating long term camber and deflection.

2.3.5.2 Incremental time-steps method –

The incremental time-steps method is based on combining the computations of deflections with those of prestress losses due to time-dependent creep, shrinkage, and relaxation. The design life of the structure is divided into several increasingly larger time intervals. The strain distributions, curvatures, and prestressing forces are calculated for each interval together with the incremental shrinkage, creep, and relaxation losses during the particular time interval. The procedure is repeated for all subsequent incremental intervals, and an integration or summation of the incremental curvatures is made to give the total time dependent curvature at the particular section along the span. These calculations should be made for a sufficient number of points along the span to be able to determine with reasonable accuracy the form of the moment-curvature diagram. (6)

The general expression for the total curvature at the end of a time interval (6)

$$\Phi t = \frac{Pi * ex}{Eci * Ic} + \sum_{0}^{t}(P(n-1) - Pn) * \frac{ex}{Ec * Ic} - \sum_{0}^{t}\big(Cn - C(n-1)\big) * P(n-1) * \frac{ex}{Ec * Ic}$$

Equation 2-64

Where Pi = initial prestress (at transfer) before losses

ex = eccentricity of tendon at any section along the span

C = creep coefficient

The total deflection at a particular section and at a particular time t is

$\delta x = \Phi t * k * l^2$	Equation 2-65

Where *k* is a function of the span and geometry of the section, and the location of prestressing tendon. The total deflection (upward or downward) due to prestressing force can be obtained from

$\Phi T = \Phi n - 1 + \Phi n$	Equation 2-66
$\delta T = \Phi T\, kl^2$	Equation 2-67

Where k is the aging coefficient varies from 0 to 1.0 being 0.7 -0.8 for most application (according to ACI 365). Several investigators have proposed different formats for estimating the additional time-dependent deflection. (6)

1. integrating the modified curvature along the beam span (Tadros and Dilger)
2. expressed the long-term deflection in terms of midspan and support curvatures at a time interval t (Naaman)

Naaman's expression gives, for a parabolic tendon,

$\Delta\delta(t) = \Phi 1(t) * \frac{l^2}{8} = [\Phi 2(t) - \Phi 1(t)] * \frac{l^2}{48}$	Equation 2-68
$\Phi(t) = \frac{M}{Ece(t)Ic}$	Equation 2-69
$Ece(t) = \frac{Ec(t1)}{1 + x * Cc(t)}$	Equation 2-70

Where Φ1(t) = midspan curvature at time t
Φ2(t) = support curvature at time t
Ece(t) = time adjusted modulus
Ec(tl) = modulus of elasticity of concrete at start of interval
x = aging coefficient usually assumed equal to 0.8
Cc(t) = creep coefficient at end of time interval

2.3.5.3 Approximate time-steps method-

The approximate time-steps method is based on a simplified form of summation of constituent deflections due to the various time dependent factors. (6)

$\Phi e = \frac{P1 * ex}{Ec * Ic} + (Pi - Pe) * \frac{ex}{Ec * Ic} - \left(\frac{Pi + Pe}{2}\right) * \frac{ex}{Ec * Ic} * Cu$	Equation 2-71
$\delta et = -\delta e - \left(\frac{\delta i + \delta e}{2}\right) * Cu \qquad (2.49)$	Equation 2-72

Where Cu = long-term creep coefficient
Φe = curvature at effective prestress Pe
δet = final deflection under Pe

The final time-dependent increase in deflection due to prestressing and sustained loads is given by

$$\Delta\delta = -\delta e - \left(\frac{\delta i + \delta e}{2}\right) * Cu + (\delta d + \delta sd) * (1 + Cu)$$ Equation 2-73

Then the final net deflection becomes.

$$\delta T = -\delta e - \left(\frac{\delta i + \delta e}{2}\right) * Cu + (\delta d + \delta sd) * (1 + Cu) + \delta L$$ Equation 2-74

Where δd and δsd are deflection due to self weight and superimposed dead load

δL = deflection due to live load. (6)

2.3.5.4 Axial strain and curvature method (Ghali-Favre)-

This approach gives a procedure for the analysis of instantaneous and long-term stresses and strains in reinforced concrete cross-sections, with or without prestressing. The method does not require determination of prestress losses. It introduces the deflection by such effect as in the Naaman approach. In this method after cracking, the concrete in tension is ignored and only the concrete in the compression zone of depth c is included in calculating the properties of the transformed section. The transformed section is composed of the area of concrete and the area (prestressing reinforcement) Aps and (non prestressed reinforcement) As of the reinforcement multiplied by the respective modular ratios, nps or n. (6)

2.3.5.5 Prestress loss method-

This method assumes that sustained dead load due to self weight does not produce cracking such that the effects of creep, shrinkage, and relaxation are considered only for uncracked cross sections. Additional stress in the concrete caused by live load may result in cracking when the tensile strength of concrete is exceeded. Whether cracking occurs, and the extent to which it occurs when the live load is applied depend upon the magnitude of the prestress losses. (6)

The method recommends stress loss coefficients due to creep, shrinkage and relaxation such that the change in the prestressing force ΔPc, is given by. (6)

$$\Delta Pc = -Ats * (\Delta fpsh + \Delta fpcr - Aps * \Delta fpr)$$ Equation 2-75

$$\delta td = (1 + Cu) * (\delta d + \delta i) + (1 + Cu') * \delta sd + (1 + x * Cu) * \delta pl$$ Equation 2-76

δtd = total deflection before live load application

δd = self-weight deflection

δi = initial prestress camber

δsd = superimposed dead load deflection, and

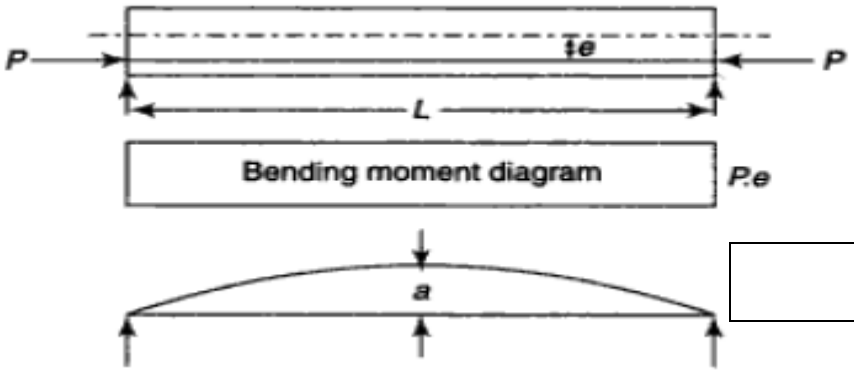
δpl = prestress loss deflection

Cu, Cu', x are given in table 3.5 of ACI 365

2.3.6 Effect of Tendon Profile on Deflection

In most of the cases of prestressed beam, tendons are located with eccentricities towards the soffit of a beam to counteract the sagging of bending moment due to transverse loads. Consequently, the beam deflect upwards (camber) on the application of prestress forces. Since the prestressing force and eccentricity, the tendon profile itself will represent the shape of the bending moment diagram. The method of computing deflection of beam with different cable profile is outlined as follows. (2) Note deflection (δ) = is symbolized as **a** in the figure.

2.3.6.1 Straight Tendon



(2)

$$\delta = -\frac{P * e * L^2}{8 * EI} \qquad \text{Equation 2-77}$$

Figure 2-7 Camber of beam with straight tendon (2)

2.3.6.2 Trapezoidal Tendon

The deflection at the center of a beam is obtained by taking the moment area of the bending moment diagram over one-half of a beam.

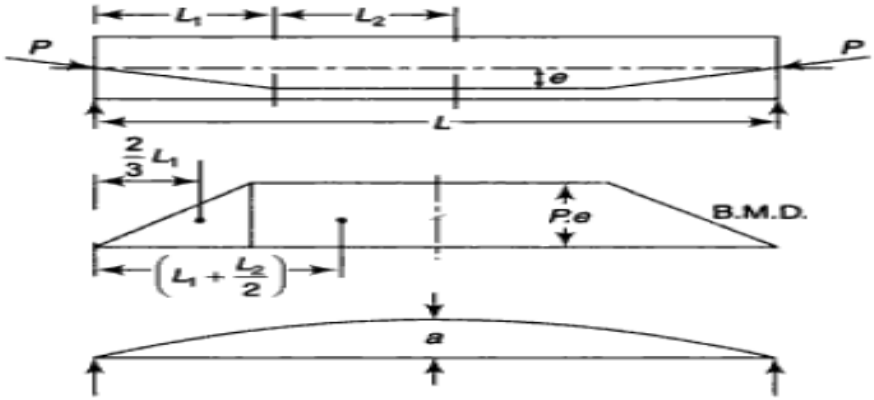
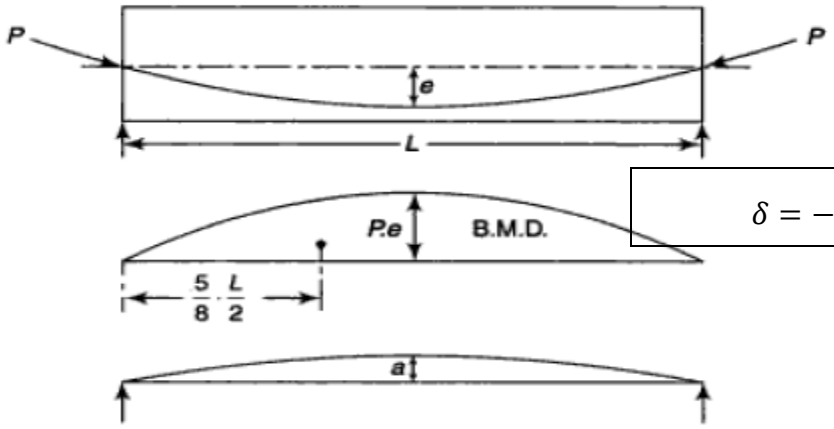


Figure 2-8 Camber of a beam Trapezoidal or Draped Tendon

$$\delta = -\frac{P * e}{6 * EI} * (2 * L1^2 + 6 * L1 * L2 + 3 * L2^2) \qquad \text{Equation 2-78}$$

2.3.6.3 Parabolic tendon central anchors

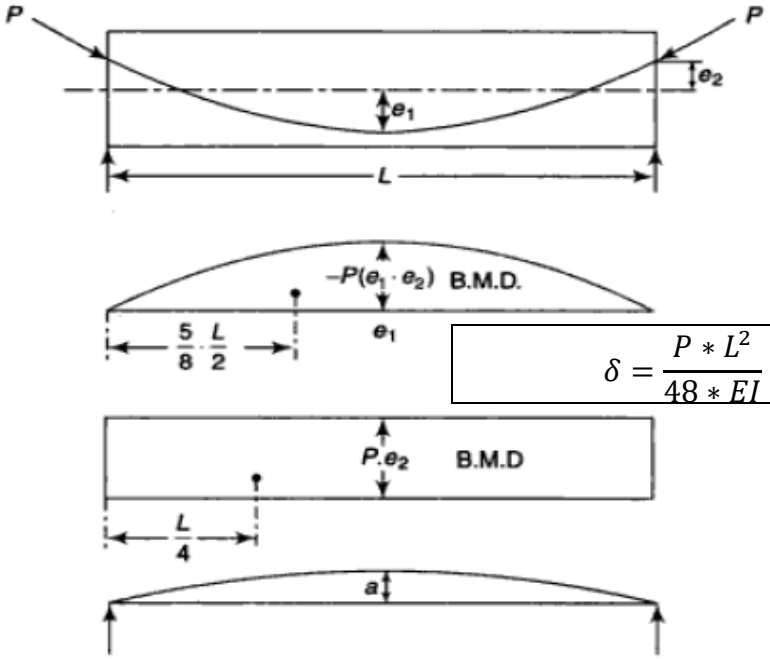


$$\delta = -\frac{5 * P * e * L^2}{48 * EI}$$ Equation 2-79

Figure 2-9 camber of parabolic tendon central anchors (2)

2.3.6.4 Parabolic tendon eccentric anchors

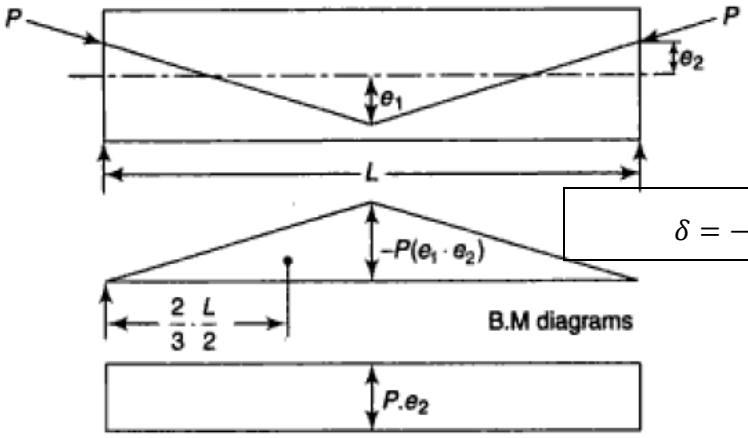
The resultant deflection at the center is obtained as the sum of the upward deflection of a beam with a parabolic tendon eccentricity (e1+e2) at the center and zero at the supports and the downward deflection of abeam subjected to uniform sagging bending moment of Pe*e2 throughout the length.



$$\delta = \frac{P * L^2}{48 * EI} * (-5 * e1 + e2)$$ Equation 2-80

Figure 2-10 Camber of parabolic tendon eccentric anchors (2)

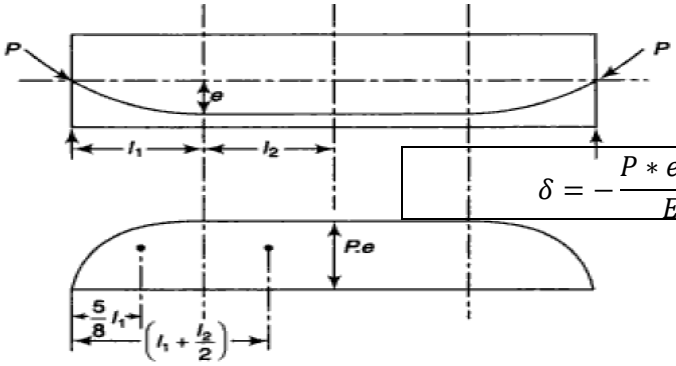
2.3.6.5 Slopping tendon eccentric anchor



$$\delta = -\frac{P * L^2}{24 * EI} * (-2 * e1 + e2) \quad \text{Equation 2-81}$$

Figure 2-11 camber of slopping tendon eccentric anchor (2)

2.3.6.6 Parabolic and straight tendon central anchor



$$\delta = -\frac{P * e * L^2}{EI} * (5L1^2 + 12L1L2 + 6L2^2) \quad \text{Equation 2-82}$$

Figure 2-12 camber of parabolic and straight tendon central anchor (2)

2.3.6.7 Parabolic and straight tendon eccentric anchor

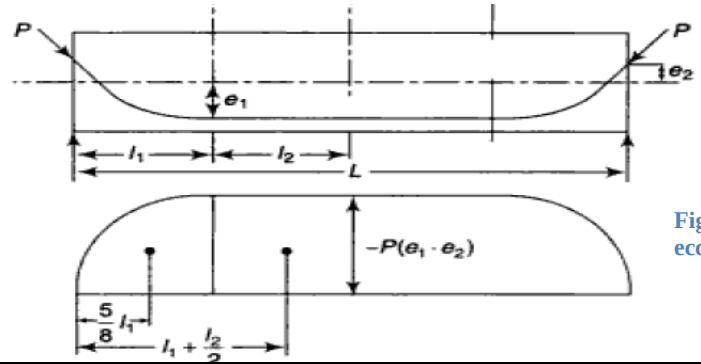


Figure 2-13 Camber of parabolic and straight tendon eccentric anchor (2)

$$\delta = -\frac{P * (e1 + e2) * L^2}{12 * EI} * (5L1^2 + 12L1L2 + 6L2^2) + (\frac{P * e2 * L^2}{8 * EI}) \quad \text{Equation 2-83}$$

2.4 Allowable Limit According to AREMA MRE volume 2

2.4.1 Deflection and Camber

According to section 17.13 of AREMA MRE volume 2

Flexural members of bridge structures shall be designed to have adequate stiffness to limit deflections or any deformations that may adversely affect strength and serviceability of the structure at service load. Members having simple or continuous spans shall be designed so that the deflection due to service live load plus impact does not exceed *l*/640 of the span. (9)

2.4.2 Minimum Concrete Cover

a. For Precast Concrete the following minimum concrete cover shall be provided for prestressing tendons and non-prestressed reinforcement, and ducts:

	Minimum Cover
Pre-tensioning tendons	1½ in. (40 mm)
Post-tensioning ducts	1½ in. (40 mm), but not less than dd/2
Non-prestressed reinforcement.....	1½ in. (40 mm)
Stirrups, ties and spirals.....	1 in. (25 mm)

b. For Cast-in-Place Concrete, the following minimum concrete cover shall be provided for prestressing tendons and non-prestressed reinforcement, and ducts:

	Minimum Cover
Post-tensioning ducts	3 in. (75 mm), but not less than dd/2
Non-prestressed reinforcement.....	2 in. (50 mm)
Stirrups, ties and spirals	2 in. (50 mm)
Concrete cast against earth	3 in. (75 mm)

c. In corrosive or marine environments or other severe exposure conditions, the amount of tendon and reinforcement protection shall be increased by use of more dense and impervious concrete, by increasing the minimum cover or other suitable means. (9)

Where dd is outside diameter of post-tensioning duct, in. (mm)

3 Methodology

3.1 Camber due to prestressing Tendon

In this paper the short term and long term deflection of a simply supported prestressed railway girder with parabolic tendon eccentric anchor is computed according to the recommendation of ACI – 435 R 95.

Camber due to prestressing tendon is calculated using Equation 2-78 in which

$\delta = -\frac{P * e * L^2}{48 * EI} * (-5 * e1 + e2)$	

3.2 Short Term Deflection

From equation 2-25 the general deflection formula for simply supported beam is.

$\delta s = \frac{5wL^4}{384EcI}$	Equation 3-1

Before crack occurs in a member the whole section is effective and deflection in this stage is computed by using the second moment of area of the gross concrete section. Therefore short term deflection is calculated using elastic method (i.e. using gross moment of inertia Ig)

After crack a prestressed concrete behaves in a manner similar to that of a reinforced concrete beam and the computation of deflection in this stage is made by considering a moment curvature relationship, which the section properties of cracked beam. Therefore if the section is cracked, deflection is computed using effective moment of inertia.

Uncracked Section: **-** When a concrete section is subjected to a flexural stress which is lower than the modulus of rupture of concrete fr, the section is assumed to be uncracked and thus its behavior is linear. Under this condition, the deflection is calculated by the basic principles of mechanics of elastic structures. In prestressed concrete, the immediate deflection, or camber, due to the effects of initial prestressing Pi and member self-weight is generally in the elastic uncracked range. Therefore the elastic formulas presented in the previous discussion could be used to calculate the instantaneous deflection of the member. Pi is the prestress force after the initial loss due to anchorage set, elastic shortening and relation between jacking and release time. For uncracked section we use the gross moment of inertia Ig for pretensioned member and the net moment of inertia In for post tensioned member with unbonded tendons.

Cracked Section: **-** Effective Ie method-In prestressed concrete members, cracks can develop at several sections along the span under maximum load. The cracked moment of inertia Icr applies at cracked sections while the gross moment of inertia applies in between cracks. Therefore we use an effective moment of inertia (Ie), which has equivalent effect for the whole section.

$Ie = Icr + (\frac{Mcr}{Ma})^3 * (Ig - Icr) \leq Ig$	$\left(\frac{Mcr}{Ma}\right) = \left(1 - \frac{ftl - fr}{fl}\right)$

Where	
fr= modulus of rupture = $7.5\lambda\sqrt{fc}$ ftl = total calculated stress in member.	fl = calculated stress due to live load yt = distance from the neutral axis to tensile face.

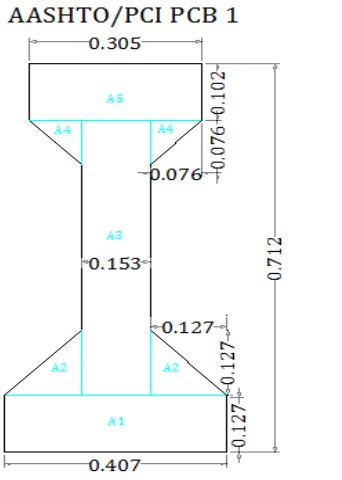
3.3 Long Term Deflection

For long term deflection PCI multipliers method is used as recommended by ACI -435.
In this method the long term deflection is obtained by multiplying the initial deflection by the constant values as provided in table 2-3

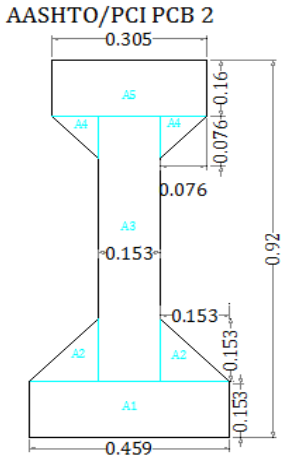
3.4 Geometry

Initially it was assumed it would be enough to do the analysis for all AASHTO I beams as presented in Appendix B of PCI Bridge Design Manual, i.e. from type I to type VI. In which the units are converted to SI unit and presented as follows. But during the process I found out that largest AASHTO I beam (type 6) works without failure about up to 22 m for this loading. As a result I decided to include another custom made I beam in which it is important dimension are inputted by the user. In this case the number of dimension that the users are going to feed is kept to the minimum to simplify the usage of the program.

Note: - I beams are chosen since they have a high second moment of area (Moment of inertia). That is because since the two flanges (or the mass of the area) is on the two other end of the centroidal axis. This intern increases the moment arm of those flange area. As moment of inertia is the square product of this moment arm, I beam has largest moment of inertia with equivalent area of other section. As result they are exceptionally strong in bending and shear and this makes them a good choice for the construction of I beam.



a, AASHTO Type I



b, AASHTO Type II

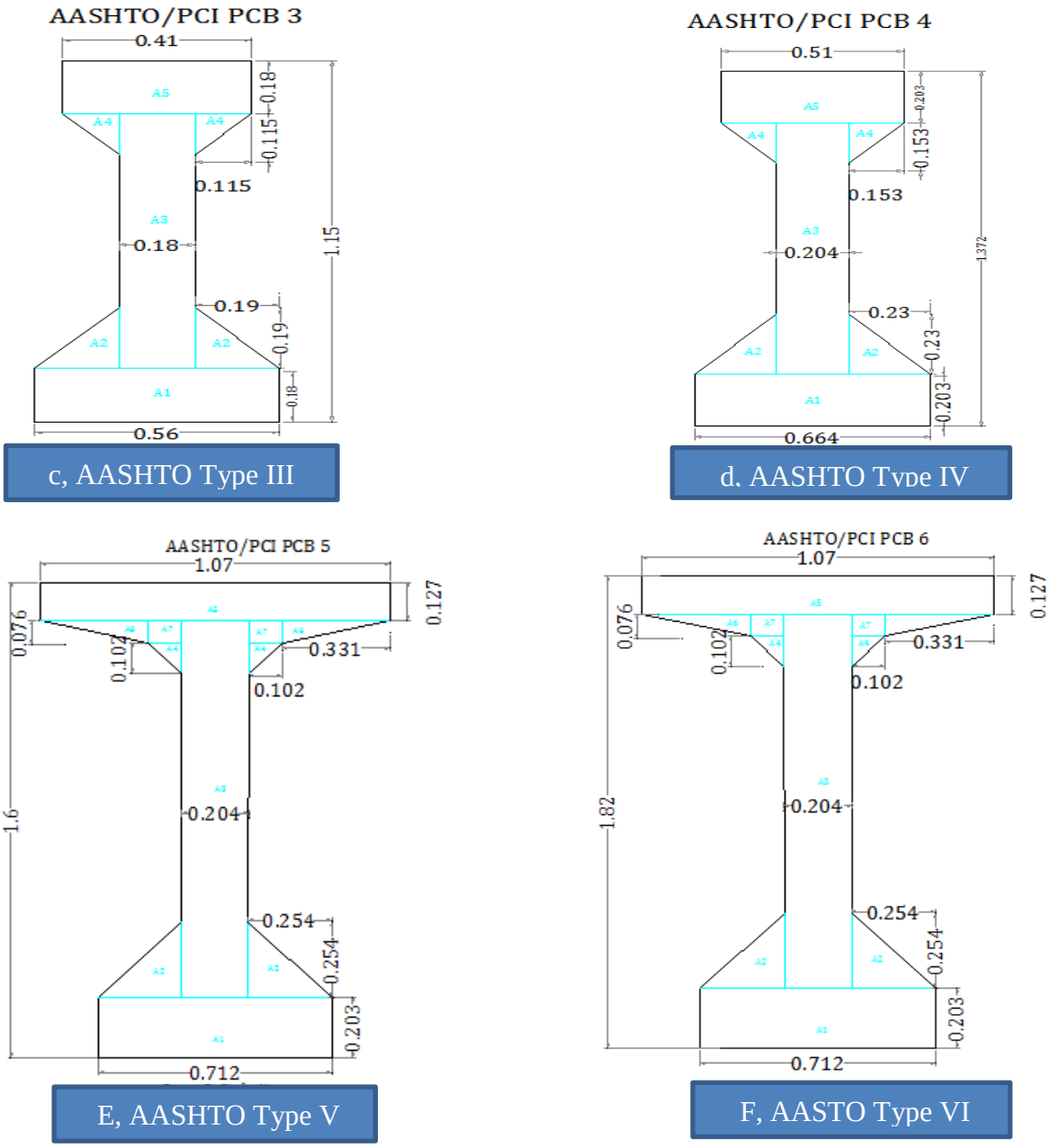


Figure 3-1 AASHTO/PCI I Beams

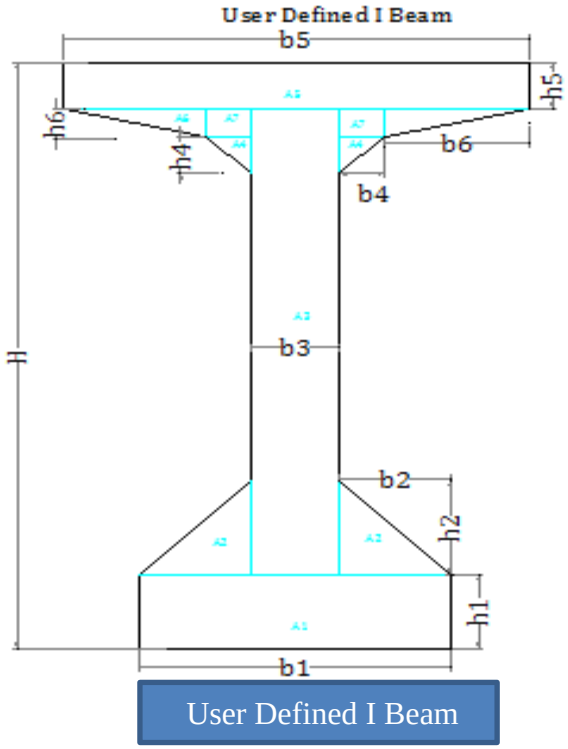


Figure 3-2 User Defined I beam based AASHTO I beam.

3.5 Loading

Super imposed dead load- the values given in AREMA MRE volume 2 will be used

Live load- Cooper loading E80 and E60 will be used for live rail loading as provided in AREMA manual for railway engineering volume 2.

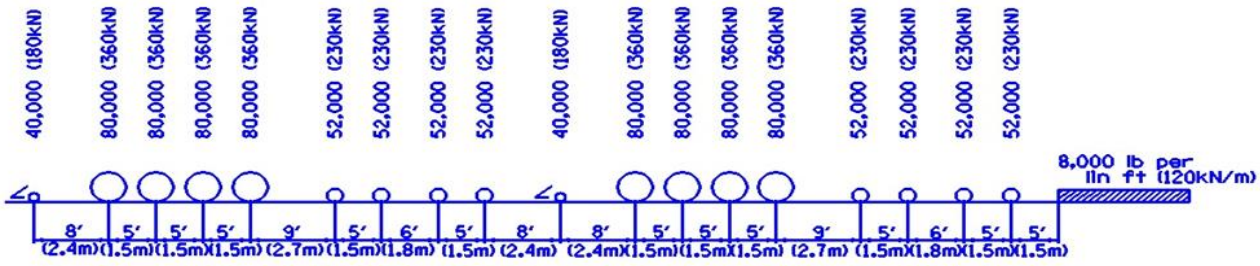


Figure 3-3 Copper E-80 (EM 360) Axle load Diagram (9)

Finally is the calculation will be coded in MATLAB programing language and presented

Sample charts will be generated and provided in this paper.

4 Analysis

4.1 Factors affecting Deflection

The deflections of prestressed concrete members are influenced by the following factors.

1. Self-weight and Imposed load
2. Span Length of a member
3. Second moment of area of cross section
4. Modulus of elasticity of concrete
5. Shrinkage, creep and relaxation of steel stress.
6. Magnitude of the prestressing force
7. Cable (Tendon) Profile
8. Fixity condition.

4.2 Loading

According to AREMA Manual for Railway Engineering the following loads and forces shall be considered in the design of railway concrete structures supporting tracks:

- Dead Load
- Live Load
- Impact
- Centrifugal Force
- Longitudinal Force from Live Load
- Longitudinal Force due to Friction or Shear Resistance at Expansion Bearings
- Earth pressure
- Buoyancy
- Wind Load on Structure
- Wind Load on Live Load
- Earthquake (Seismic)
- Stream Flow Pressure
- Ice Pressure

Of all these loads we will only choose the dead load (self-weight plus superimposed dead load), the live load and the impact load. Which directly affect the deflection of the beam.

4.2.1 Self-weight (Wdl)

Self weight per unit meter length is the cross sectional area of a beam times unit weight of concrete which is equal to 25KN/m^3

$$Wdl = Ac * \gamma c \qquad \text{Equation 4-1}$$

4.2.2 Super imposed Dead Load

The unit weight of materials comprising the dead load, except in special cases involving unusual conditions or materials, shall be assumed as follows

- Track rails, inside guardrails and fastenings – 200 lb. per linear foot of track. (3kN/m)
- Ballast, including track ties – 120 lb. per cubic foot. (1900 kg/m^3)
- Reinforced concrete – 150 lb. per cubic foot. (2400 kg/m^3)
- Earth filling materials – 120 lb. per cubic foot. (1900 kg/m^3)
- Waterproofing and protective covering – estimated weight

Table 4-1 Summary of Super Imposed Dead Load

Assuming Double Track and Two Girder as The LRT case

Super Imposed Dead Load KN/m		
Item	Total	For one Girder
Track rail, inside guardrails and fastenings	3	1.5
Ballast	19*0.3*ccb*2	Wb
Sleeper: The numbers of sleepers per rail length can be known by taking the design standard from LRT. Sleeper spacing = 0.6m	3.0307	1.51536
Deck slab	25*ts*le	Ws
Concrete curb	15	7.5
Waterproofing and protective covering	0.15	0.075
Catenary system	0.02	0.01
Future utilities	0.05	0.025
Girder cross tie (diaphragm) is a transverse girder that is used to stiffen the main load bearing girders. According to AREMA, Art. 8.17.12 c, diaphragm for I-beam section is not required.	0	0
Total	57.16+Ws+Wb	**10.625+Ws Wb**

Where Ws is weight of the Deck slab per unit meter length of beam

4.2.3 Live Load

The recommended live load for each track of main line structure is Cooper E 80 (EM 360) loading with axle loads and axle spacing as shown in Fig 3.2 On branch lines and in other locations where the loading is limited to the use of light equipment, or cars only, the live load may be reduced, as directed by the engineer. For structures wherein the material in the primary load-carrying members is not concrete, the E loading used for the concrete design shall be that used for the primary members. (9)

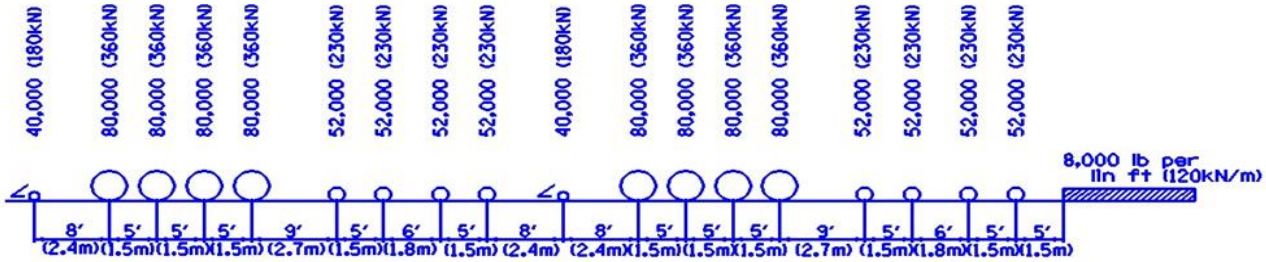


Figure 4-1 Copper E-80 (EM 360) Axle load Diagram (9)

Fortunately for simple span this load is tabulated in table 15-1-15 of AREMA manual for railway engineering volume 2 as follows.

Table 4-2 Calculation of moments on short, simple span from Table 15-1-6 of AREMA MRE

Span, L	Location of Mmax	Maximum Moment (Cooper E-80)
0.00 ft. < L £ 8.54 ft.	L/2	Mmax = 10L
8.54 ft. < L £ 11.12 ft.	L/2 +/-1.25 ft	Mmax = 20L - 100 + 125/L
11.12 ft. < L £ 18.66 ft.	L/2	Mmax = 30L - 200
18.66 ft. < L £ 27.61 ft.	L/2 +/-1.25 ft L/2 +/-	Mmax = 40L - 400 + 250/L
27.61 ft. < L £ 34.97 ft.	0.389 ft L/2 +/-0.961 ft	Mmax = 45L - 530 + 27.2/L
34.97 ft. < L £ 38.72 ft.	L/2 +/-0.211 ft L/2 +/-	Mmax = 51.5L - 762 + 190/L
38.72 ft. < L £ 49.56 ft.	1.45 ft L/2 +/-0.127 ft	Mmax = 58L - 1009 + 10.35/L
48.31 ft. < L £ 53.54 ft.	L/2 +/-1.374 ft L/2 +/-	Mmax = 64.5L - 1334 + 542.2/L
53.54 ft. < L £ 58.47 ft.	0.068 ft L/2 +/-0.088 ft	Mmax = 71L - 1672 + 4.6/L
58.47 ft. < L £ 63.42 ft.		Mmax = 77.5L - 2062 + 585.4/L
63.42 ft. < L £ 75.15 ft.		Mmax = 84L - 2465 + 1.6/L
75.15 ft. < L £ 79.83ft		Mmax = 97L - 3442 + 3/L
1At L = 80 ft., the last formula will give a value which is 99.98% of the value given in Table 15-1-15.		
Span, L	**Location of Mmax**	**Max Moment (Alt LL: 4 - 100k Axles)**
0.00 ft. < L £ 8.54 ft.	L/2	Mmax = 12.5L
8.54 ft. < L £ 12.94 ft.	L/2 +/-1.25 ft L/2 +/-	Mmax = 25L - 125 + 156.25/L
12.94 ft. < L £ 20.24 ft.	0.167 ft L/2 +/-1.5 ft	Mmax = 37.5L - 275 + 4.17/L
L > 20.24 ft.		Mmax = 50L - 550 + 450/L

Table 4-3 Maximum moments, shears and pier Reaction for Cooper E 80 Live load or alternate load (9)

Note all values are for one rail (one half-track load)

Span Length Ft	Maximum Moment Ft–Kips		Maximum Moment Quarter Point Ft–Kips		Maximum Shears Kips						Maximum Pier Reaction Kips (2)	
					At End		At Quarter Point		At Center			
	E-80	Alt.	E-80	Alt.	E-80	Alt.	E-80	Alt.	E-80	Alt.	E-80	Alt.
5	50	62.5	37.5	46.88	40	50			20	25	40	50
6	60	75	45	56.25	46.67	58.33	30	37.5	20	25	53.33	58.33
7	70	87.5	55	68.75	51.43	64.29	31.43	39.29	20	25	62.86	71.43
8	80	100	70	87.5	55	68.75	35	43.75	20	25	70	81.25
9	93.89	117.36	85	106.25	57.58	72.22	37.78	47.23	20	25	75.76	88.89
10	112.5	140.63	100	125	60	75	40	50	20	25	80	95
11	131.36	164.2	115	143.75	65.45	77.27	41.82	52.28	21.82	27.28	87.28	100
12	160	188.02	130	162.5	70	83.33	43.33	54.17	23.33	29.17	93.33	108.33
13	190	212.83	145	181.25	73.84	88.46	44.61	55.76	24.61	30.76	98.46	115.39
14	220	250.3	165	200	77.14	92.86	47.14	57.14	25.71	32.14	104.29	121.43
16	280	325.27	210	250	85	100	52.5	62.5	27.5	34.38	113.74	131.25
18	340	400.24	255	318.79	93.33	111.11	56.67	68.05	28.89	36.11	121.33	138.89
20	412.5	475	300	362.5	100	120	60	72.5	28.7	37.5	131.1	145
24	570.42	668.75	420	500	110.83	133.33	70	83.33	31.75	41.67	147.92	154.17
28	730.98	866.07	555	650	120.86	142.86	77.14	92.86	34.29	46.43	164.58	
32	910.85	1064.06	692.5	800	131.44	150	83.12	100	37.5	50	181.94	
36	1097.3	1262.5	851.5	950	141.12	155.56	88.9	105.56	41.1	55.56	199.06	
40	1311.3	1461.25	1010.5	1100	150.8	160	93.55	110	44	60	215.9	
45	1601.2	1710	1233.6	1287.48	163.38	164.44	100.27	114.45	45.9	64.45	237.25	
50	1901.8	1959	1473	1481.05	174.4		106.94	118.42	49.73	68	257.52	
55	2233.1		1732.3		185.31		113.58	120.91	52.74	70.91	280.67	
60	2597.8		2010		196		120.21	123.33	55.69	73.33	306.42	
70	3415		2608.2		221.04		131.89		61.45	77.14	354.08	
80	4318.9		3298		248.4		143.41		67.41	80	397.7	
90	5339.1		4158		274.46		157.47		73.48	82.22	437.15	
100	6446.3		5060.5		300		173.12		78.72	84	474.24	
120	9225.4		7098		347.35		202.19		88.92		544.14	
140	12406		9400		392.59		230.23		101.64		614.91	

Span Length Ft	Maximum Moment Ft–Kips		Maximum Moment Quarter Point Ft–Kips		Maximum Shears Kips						Maximum Pier Reaction Kips (2)	
					At End		At Quarter Point		At Center			
	E-80	Alt.	E-80	Alt.	E-80	Alt.	E-80	Alt.	E-80	Alt.	E-80	Alt.
160	15908.00 (1)		11932		436.51		265.51		115.2		687.5	
180	19672.00 (1)		14820		479.57		281.96		128.12		762.22	
200	23712.00 (1)		17990		522.01		306.81		140.8		838	
250	35118.00 (1)		27154		626.41		367.3		170.05		1030.4	
300	48800.00 (1)		38246		729.34		426.37		197.93		1225.3	
350	65050.00 (1)		51114		831.43		484.64		225.51		1421.7	
400	83800.00 (1)		65588		933		542.4		252.44		1619	
Note (1) - Values for Cooper E-80 Live Load. Moment values taken at center span. Note (2) - Maximum pier reactions are for equal span lengths.												

The values are converted to SI unit and also adjusted to copper E60 live load as follows

Table 4-4Maximum Live load moments for cooper E80 and cooper E 60 live load

	Maximum Moments at Mid span (KNm)	
Span Length (m)	E - 80 Loading	E - 60 Loading
1.52	67.79	50.84
1.83	81.35	61.01
2.13	94.91	71.18
2.44	108.46	81.35
2.74	127.30	95.47
3.05	152.53	114.40
3.35	178.10	133.57
3.66	216.93	162.70
3.96	257.60	193.20
4.27	298.28	223.71
4.88	379.62	284.72
5.49	460.97	345.73
6.10	559.27	419.45
7.32	773.38	580.03
8.53	991.06	743.30
9.75	1234.93	926.20
10.97	1487.72	1115.79

	Maximum Moments at Mid span (KNm)	
Span Length (m)	E - 80 Loading	E - 60 Loading
12.19	1777.86	1333.40
13.72	2170.91	1628.18
15.24	2578.46	1933.85
16.76	3027.64	2270.73
18.29	3522.10	2641.57
21.34	4630.06	3472.54
24.38	5855.56	4391.67
27.43	7238.75	5429.06
30.48	8739.89	6554.92
36.58	12507.80	9380.85
42.67	16820.05	12615.04
48.77	21568.07	16176.05
54.86	26671.30	20003.47
60.96	32148.73	24111.55
76.20	47612.98	35709.74
91.44	66163.04	49622.28
106.68	88194.79	66146.09
121.92	113616.04	85212.03

4.2.4 Impact Load

Impact forces, applied at the top of rail, shall be added to the axle loads specified. For rolling equipment without hammer blow (diesels, electric locomotives, tenders alone, etc.), the impact shall be equal to the following percentages of the live load: (9)

For L < 4m $I = 60$

For 4 m < L < 39 m $I = 125/\sqrt{L}$

For L > 39 m $I = 20$

Where L is the span length in feet (meters).

This formula is intended for ballasted-deck spans and substructure elements as required.

4.3 Geometry

4.3.1 Precast Section

As described on the above section, I have selected AASHTO I beams for the calculation of deflection. The analysis will be done for all AASHTO I beams as presented in Appendix B of

PCI Bridge Design Manual, i.e. from type I to type VI. Section property for each of AASHTO I beam is calculated and tabulated as follows.

Precast Girder only						PCB 1			
No	b	h	A	y	Ay_i	I = (b*h3)/12	d_i	Ad_i^2	I = Io + Adi^2
A1	0.407	0.127	0.0517	0.0635	0.00328	6.95E-05	0.2567	0.00341	0.00348
A2	0.127	0.127	0.0161	0.1693	0.00273	1.45E-05	0.1509	0.00037	0.00038
A3	0.153	0.483	0.0739	0.3685	0.02723	1.44E-03	0.0483	0.00017	0.00161
A4	0.076	0.076	0.0058	0.5847	0.00338	1.85E-06	0.2645	0.00040	0.00041
A5	0.305	0.102	0.0311	0.6610	0.02056	2.70E-05	0.3408	0.00361	0.00364
TOTAL			0.1786		0.05719				0.00951

Precast Girder Only				
Area	Ac =	∑A	0.17860	m^2
Centroid	Y =	∑AY/∑A	0.32018	m
Second Moment of Inertia	I =	∑I	0.00951	m^4
Centroid	yb =	Y	0.32018	m
	Yt =	H-Yb	0.39182	m
Section Modulus	Zt =	I/Yt	0.02428	m^3
	Zb =	I/yb	0.02971	m^3

Table 4-5 section properties of AASHTO I beam type I

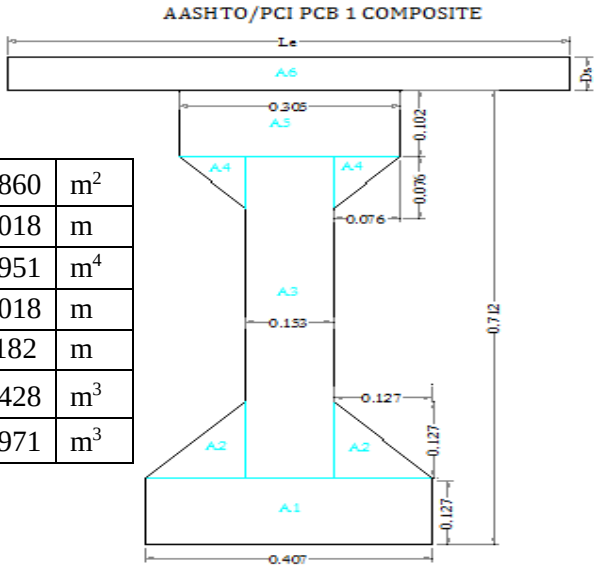


Figure 4-2 AAHSTO I beam Type II composite section

Precast Girder only									
No	b	h	A	y	Ay_i	I = (b*h3)/x	d_i	Ad_i^2	I = Io + Adi^2
A1	0.46	0.153	0.0702	0.0765	0.0054	1.37E-04	0.3291	0.00761	0.00774
A2	0.15	0.153	0.0234	0.2040	0.0048	3.04E-05	0.2016	0.00095	0.00098
A3	0.15	0.607	0.0929	0.4565	0.0424	2.85E-03	0.0509	0.00024	0.00309
A4	0.08	0.076	0.0058	0.7347	0.0042	1.85E-06	0.3291	0.00063	0.00063
A5	0.31	0.16	0.0488	0.8400	0.0410	1.04E-04	0.4344	0.00921	0.00931
TOTAL			0.2411		0.0978				0.02176

Precast Girder Only				
Area	Ac	∑A	0.24108	m^2
Centroid	Y =	∑AY/∑A	0.40558	m
Second Moment of Inertia	I	∑A	0.02176	m^4
Centroid	yb =	Y	0.40558	m
	Yt =	H- Y	0.51442	m
Section Modulus	Zt =	I/Yt	0.04229	m^3
	Zb =	I/yb	0.05364	m^3

Table 4-6 section properties of AASHTO I beam type II

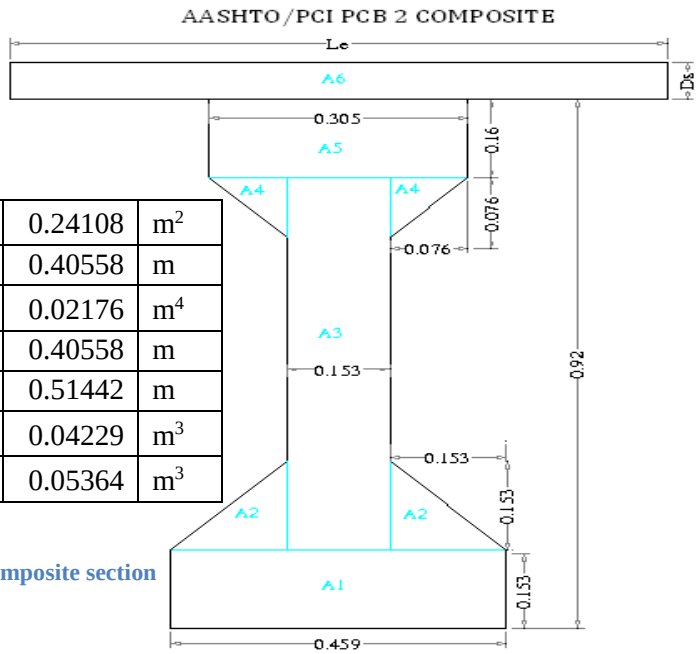


Figure 4-3 AAHSTO I beam Type II composite section

Precast Girder only				PCB 3					
No	b	h	A	y	Ay_i	I = (b*h3)/12	d_i	Ad_i^2	I = Io + Adi^2
A1	0.560	0.180	0.1008	0.0900	0.00907	2.72E-04	0.4294	0.01859	0.01886
A2	0.190	0.190	0.0361	0.2433	0.00878	7.24E-05	0.2761	0.00275	0.00282
A3	0.180	0.790	0.1422	0.5750	0.08177	7.40E-03	0.0556	0.00044	0.00783
A4	0.115	0.115	0.0132	0.9317	0.01232	9.72E-06	0.4123	0.00225	0.00226
A5	0.410	0.180	0.0738	1.0600	0.07823	1.99E-04	0.5406	0.02157	0.02177
TOTAL			0.3661		0.19017				0.05354

Precast Girder Only				
Area	Ac=	∑A	0.36613	m^2
Centroid	Y =	∑AY/∑A	0.51941	m
Second Moment of Inertia	I	∑I	0.05354	m^4
Centroid	yb =	Y	0.51941	m
	Yt =	H- Y	0.63059	m
Section Modulus	Zt =	I/Yt	0.08491	m^3
	Zb =	I/yb	0.10308	m^3

Table 4-7 section properties of AASHTO I beam type III

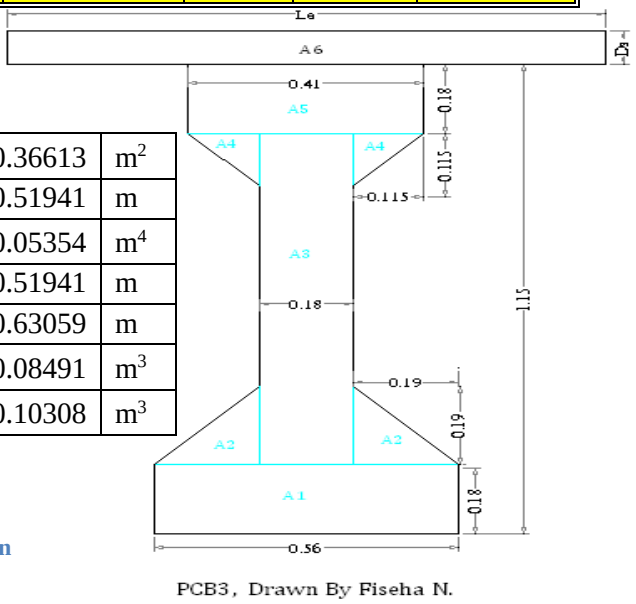


Figure 4-4 AAHSTO I beam Type III composite section

Precast Girder only				PCB 4					
No	b	h	A	y	Ay_i	I = (b*h3)/12	d_i	Ad_i^2	I = Io + Adi^2
A1	0.664	0.203	0.1348	0.1015	0.01368	4.63E-04	0.5265	0.03737	0.03783
A2	0.230	0.230	0.0529	0.2797	0.01479	1.55E-04	0.3484	0.00642	0.00658
A3	0.204	0.966	0.1971	0.6860	0.13519	1.53E-02	0.0580	0.00066	0.01599
A4	0.153	0.153	0.0234	1.1180	0.02617	3.04E-05	0.4900	0.00562	0.00565
A5	0.510	0.203	0.1035	1.2705	0.13153	3.56E-04	0.6425	0.04273	0.04309
TOTAL			0.5117		0.32137				0.10913

Precast Girder Only				
Area	Ac	∑A	0.51170	m^2
Centroid	Y =	∑AY/∑A	0.62805	m
Second Moment of Inertia	I	∑I	0.10913	m^4
Centroid	yb =	Y	0.62805	m
	Yt =	H- Y	0.74395	m
Section Modulus	Zt =	I/Yt	0.14669	m^3
	Zb =	I/yb	0.17377	m^3

Table 4-8 section properties of AASHTO I beam type IV

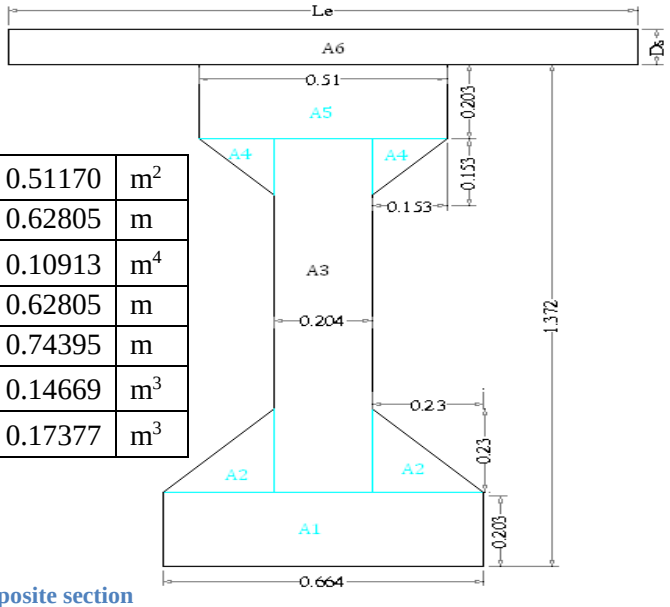
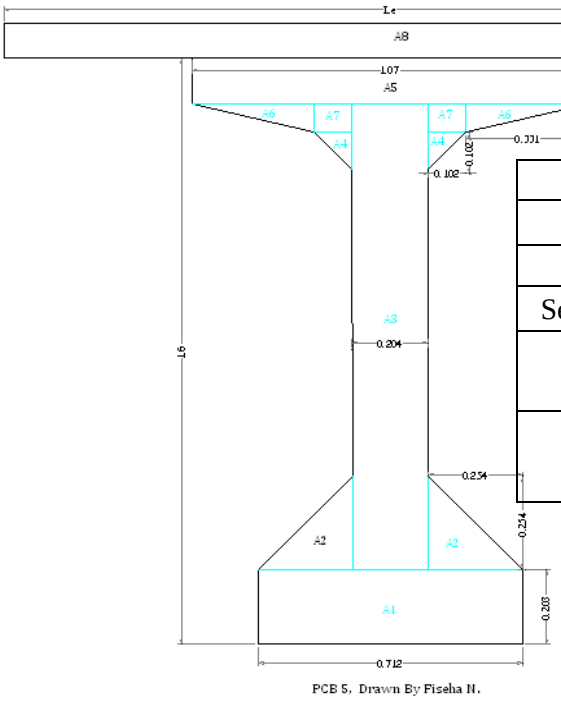


Figure 4-5 AAHSTO I beam Type IV composite section

PCB4, Drawn By Fiseha N.

Precast Girder only				PCB 5					
No	b	h	A	y	Ay_i	I = (b*h3)/12	d_i	Ad_i^2	I = Io + Adi^2
A1	0.712	0.203	0.1445	0.1015	0.0147	4.96E-04	0.7106	0.0730	0.0735
A2	0.254	0.254	0.0645	0.2877	0.0186	2.31E-04	0.5244	0.0177	0.0180
A3	0.204	1.27	0.2591	0.8380	0.2171	3.48E-02	0.0259	0.0002	0.0350
A4	0.102	0.102	0.0104	1.3630	0.0142	6.01E-06	0.5509	0.0032	0.0032
A6	0.331	0.076	0.0252	1.4477	0.0364	2.64E-05	-0.6356	0.0102	0.0102
A7	0.102	0.076	0.0155	1.4350	0.0222	2.49E-06	0.6229	0.0060	0.0060
A5	1.07	0.127	0.1359	1.5365	0.2088	1.83E-04	0.7244	0.0713	0.0715
TOTAL			0.6551		0.5320	3.58E-02		0.1815	0.2173



Precast Girder Only				
Area	Ac=	∑A	0.65509	m^2
Centroid	Y =	∑AY/∑A	0.81210	m
Second Moment of Inertia	I =	∑I	0.21731	m^4
Centroid	yb =	Y	0.81210	m
	Yt =	H- Y	0.78790	m
Section Modulus	Zt =	I/Yt	0.27580	m^3
	Zb =	I/yb	0.26760	m^3

Table 4-9 section properties of AASHTO I beam Type V

Figure 4-6 AAHSTO I beam Type V composite section

Girder only				PCB 6					
No	b	h	A	y	Ay_i	I = (b*h3)/12	d_i	Ad_i^2	I = Io + Adi^2
A1	0.712	0.203	0.1445	0.1015	0.01467	4.96E-04	0.8188	0.09689	0.09739
A2	0.254	0.254	0.0645	0.2877	0.01856	2.31E-04	0.6326	0.02582	0.02605
A3	0.204	1.49	0.3040	0.9480	0.28815	5.62E-02	0.0277	0.00023	0.05647
A4	0.102	0.102	0.0104	1.5830	0.01647	6.01E-06	0.6627	0.00457	0.00458
A6	0.331	0.076	0.0252	1.6677	0.04195	2.64E-05	-0.7474	0.01405	0.01408
A7	0.102	0.076	0.0155	1.6550	0.02566	2.49E-06	0.7347	0.00837	0.00837
A5	1.07	0.127	0.1359	1.7565	0.23869	1.83E-04	0.8362	0.09503	0.09521
TOTAL			0.7000		0.64415				0.30214

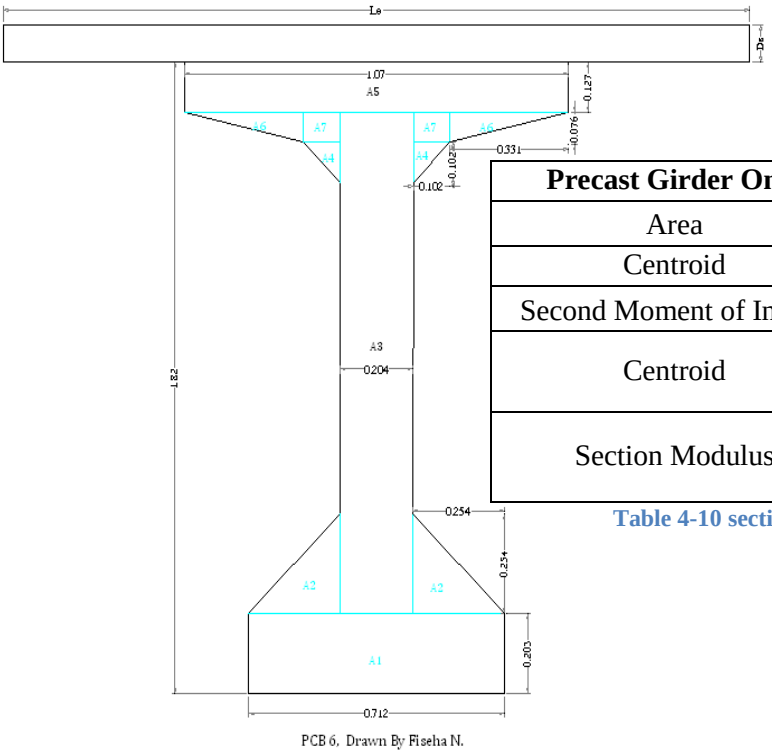


Figure 4-7 AAHSTO I beam Type VI composite section

Precast Girder Only				
Area	Ac =	∑A	0.69997	m^2
Centroid	Y =	∑AY/∑A	0.92027	m
Second Moment of Inertia	I =	∑I	0.30214	m^4
Centroid	yb =	Y	0.92027	m
	Yt =	H- Y	0.89973	m
Section Modulus	Zt =	I/Yt	0.33581	m^3
	Zb =	I/yb	0.32832	m^3

Table 4-10 section properties of AASHTO I beam type VI

4.3.1.1 Effective Length (Le)

According to AASHTO -2005 section 4.6.2.6

For edge beam

$Le \leq \frac{CCG}{2} + minmum\ of \begin{bmatrix} L/8 \\ 6 * ts + fwe \\ Wg \end{bmatrix}$	Equation 4-2
$fwe = maximum\ of \begin{pmatrix} fwi/4 \\ bwi/2 \end{pmatrix}$	Equation 4-3

- ***Effective Length (Le)***

For interior beam

$Le \leq minmum\ of \begin{bmatrix} L/4 \\ 12 * ts + fwm \\ CCG \end{bmatrix}$	Equation 4-4
$fwm = maximum\ of \begin{pmatrix} fwi/2 \\ bwi \end{pmatrix}$	Equation 4-5

Where

Le: - is effective length
CCG: - center to center distance between adjacent girders
L: - Span of a girder
ts: - thickness of cast insitu slab
fwi :- top flange width of precast section
bwi:- web thickness of precast section
Wg:- overhang length of edge girder

4.4 Material Property

4.4.1 Concrete

Unit weight of concrete (γc) taken as 25KN/m^3

In the program one Can choose the cubic strength (C) of concrete for which the deflection is about to be calculated and presented in the chart.

Once the cubic strength is known the rest of the important parameter of the concrete is calculated as follows. (10)

It is assumed that the quality of concrete is to be used for the cast insitu slab and the precast prestressed beam may differ. Therefore the modulus of elasticity of cast insitu concrete (Ecs) and precast concrete is equal to Ecp, gives modular ratio between the two concrete (ηm) is equal to Ecp/Ecs.

- The characteristic strength of concrete (Fck)

$Fck = 0.8 * C$	Equation 4-6

- Modulus of Elasticity of concrete (Ec)

$Ec = 4700 * \sqrt{C}$	Equation 4-7

- Modulus of Rupture of Concrete (fr)

$fr = 0.623 * \sqrt{C}$	Equation 4-8

- Permissible stress
  - At transfer

$Allowable\ compressive\ strenght\ Fct \leq 0.6 * Fck$	Equation 4-9
$Allowable\ tensile\ strenght\ Ftt = 0.21 * (Fck)^{2/3}$	Equation 4-10

  - Underworking

$Allowable\ compressive\ strength\ Fcw \leq 0.5 * Fck$	Equation 4-11
$Allowable\ tensile\ strength\ Ftw = 0.75 * Ftt$	Equation 4-12

  - Permissible stress limit under service

$Stress\ limit\ at\ the\ top\ Ftr = Fcw + \eta * Ftt$	Equation 4-13
$Stress\ limit\ at\ the\ botom\ Fbr = Fct + \eta * Ftw$	Equation 4-14

η is the ratio of effective stress to initial stress see the next section

4.4.2 Prestressing Tendon

Though the default value of 1860 Mpa is to be used in the computation tensile strength of the tendon (Fpk) is to be imputed by the user.

Stress loss (η) the expected stress loss is again to be provided by the user. The default value is 18%

Modulus of elasticity of tendon (Ep)

$Ep = 196.5E3\ Mpa$

- The characteristic tensile strength of prestressing tendon (Fpy)

$Fpy = 0.9 * Fpk$	Equation 4-15

- Stress limit
  - ***Initial Tensioning***

$Fpit = 0.8 * Fpk$	Equation 4-16

  - ***At transfer***

$Fpi = 0.75 * Fpk$	Equation 4-17

  - ***Under working***

$Fpw = 0.8 * Fpi$	Equation 4-18

- Effective stress (Fpe)

$\boldsymbol{Fpe \geq 0.6 * Fpk}$	Equation 4-19

- Modular ratio between tendon and concrete (ηp)

$\eta p = \frac{Ep}{Ec} = \frac{196.5 * 10^3}{EC}$	Equation 4-20

4.5 Eccentricity and Minimum prestressing reinforcement

4.5.1 Eccentricity

Eccentricity refers to the offset of prestressing tendon from the neutral axis of the precast girder section. The tendon eccentricity both at the support and at mid span will be imputed by the user. The possible range of those eccentricities for a given section will be calculated and provide to guide the user. The possible range of the eccentricities (maximum and minimum eccentricities at a section) will be calculated using Magnel diagram

Limiting Zone of prestressing

A. At support where moment is zero

1. $e \leq \frac{Zt * Ftt}{Pi} + \frac{Zt}{Ac}$ Equation 4-21

2. $e \leq \frac{Zb * Fct}{Pi} - \frac{Zb}{Ac}$ Equation 4-22

3. $e \geq -\frac{Zt * Fcw}{\eta * Pi} + \frac{Zt}{Ac}$ Equation 4-23

4. $e \geq -\frac{Zb * Ftw}{\eta * Pi} - \frac{Zb}{Ac}$ Equation 4-24

B. At Mid Span

1. $$e \leq \frac{Zt * Ftt}{Pi} + \frac{Zt}{Ac} + \frac{Mgl}{Pi}$$ **Equation 4-25**

2. $$e \leq \frac{Zb * Fct}{Pi} - \frac{Zb}{Ac} + \frac{Mgl}{Pi}$$ **Equation 4-26**

3. $$e \geq -\frac{Zt * Fcw}{\eta * Pi} + \frac{Zt}{Ac} + \frac{Mtot}{\eta * Pi}$$ **Equation 4-27**

4. $$e \geq -\frac{Zb * Ftw}{\eta * Pi} - \frac{Zb}{Ac} + \frac{Mtot}{\eta * Pi}$$ **Equation 4-28**

4.5.2 Minimum Prestressing reinforcement

If we introduce two parameters Finf and Fsup defined as

$$Finf = -\frac{ftw}{n} + \frac{(Mdl + Msl + Mll)}{n * Zb}$$ Equation 4-29

$$Fsup = -ftt - Mdl/Zt$$ Equation 4-30

Then 2-14 of minimum prestressing forces simplified as

$$Pimin = Ac * (\frac{Finf * Zb + Fsup * Zt}{Zt + Zb})$$ **Equation 4-31**

and 2-15 of the corresponding maximum eccentricity simplified as

$$emax = \frac{(Finf - Fsup) * Zt * Zb}{Ac * (Fsup * Zt + Finf * Zb)}$$ **Equation 4-32**

Accordingly the minimum prestressing reinforcement area is defined as

$$Asmin = \frac{Pimin}{fpi}$$ **Equation 4-33**

4.6 Deflection of Prestressed Beam

4.6.1 Section Stress

4.6.1.1 At transfer

a. Mid Span	
$Top: \quad ftim = \frac{Pi}{Ac} - \frac{Pi.ec}{Zt} + \frac{Mdl}{Zt}$	Equation 4-34
$bottm: \quad fbim = \frac{Pi}{Ac} + \frac{Pi.ec}{Zb} - \frac{Mdl}{Zb}$	Equation 4-35
b. Support	
$Top: \quad ftis = \frac{Pi}{Ac} - \frac{Pi.ee}{Zt}$	Equation 4-36
$bottm: \quad fbis = \frac{Pi}{Ac} + \frac{Pi.ee}{Zb}$	Equation 4-37

- If all stresses (*ftim, fbim,, ftis, fbis*) is in the range between (-fr ,fct) i.e. greater than *–fr* and less than *fct* the section is neither cracked nor crushed so elastic method of analysis will be used in the section.
- If one of them is less than *–fr* the section is cracked then effective Ie method of deflection calculation will be used in this section and in the following section.
- If one of them is greater than > *fct* the section is cracked and deflection calculation will be stopped and the program return a value of zero in this and the following section.

4.6.1.2 After un-shored slab is cast

a. Mid Span	
$Top: \quad ftsm = \frac{Pe}{Ac} - \frac{Pe.ec}{Zt} + \frac{(Mdl + Msd)}{Zt}$	Equation 4-38
$bottm: \quad fbsm = \frac{Pe}{Ac} + \frac{Pe.ec}{Zb} - \frac{(Mdl + MSd)}{Zb}$	Equation 4-39
b. Support	
$Top: \quad ftss = \frac{Pi}{Ac} - \frac{Pi.ec}{Zt}$	Equation 4-40
$bottm: \quad fbss = \frac{Pi}{Ac} + \frac{Pi.ec}{Zb}$	Equation 4-41

- If all stresses (*ftsm, fbsm,, ftss, fbss*) is in the range between *(-fr ,fct)* i.e. greater than *–fr* and less than *fct* the section is neither cracked nor crushed so elastic method of analysis will be used in the section.
- If one of them is less than *–fr* the section is cracked then effective Ie method of deflection calculation will be used in this section and in the following section.
- If one of them is greater than > *fct* the section is cracked and deflection calculation will be stopped and the program return a value of zero in this and the following section.

4.6.1.3 At service load for precast section

a. Mid Span

$Top: \quad ftem = \frac{Pe}{Ac} - \frac{Pe.ec}{Zt} + \frac{(Mdl + Msd)}{Zt} + \frac{Mld}{Ztp}$	Equation 4-42
$bottm: \quad fbem = \frac{Pe}{Ac} + \frac{Pe.ec}{Zb} - \frac{(Mdl + MSd)}{Zb} - \frac{Mld}{Zbp}$	Equation 4-43

a. At support.

Since the moment is zero at support the stresses are the same as the previous section.

- If all stresses (*ftem, fbem*) is in the range between *(-fr ,fct)* i.e. greater than *–fr* and less than *fct* the section is neither cracked nor crushed so elastic method of analysis will be used in the section.
- If one of them is less than *–fr* the section is cracked then effective Ie method of deflection calculation will be used in this section and in the following section.
- If one of them is greater than > *fct* the section is cracked and deflection calculation will be stopped and the program return a value of zero in this and the following section.

4.6.2 Deflection Calculation

4.6.2.1 At Transfer

1. Due to initial prestressing

$\delta pi = -\frac{Pi * L^2}{48 * Ec * Ig} * (-5 * ec + ee)$	Equation 4-44

2. Due to Self weight

a. If the section is un-cracked

$\delta dl = \frac{5wdl * L^4}{384 * Ec * Ig}$	Equation 4-45

b. If the section is cracked

$\delta dl = \frac{5wdl * L^4}{384 * Ec * Ie}$	Equation 4-46

From Equation 2-52, 2-53 and 2-55	
$Ie = Icr + (\frac{Mcr}{Ma})^3 * (Ig - Icr) \leq Ig$	
$\left(\frac{Mcr}{Ma}\right) = \left(1 - \frac{ftl - fr}{fl}\right)$	
$ftl = total\ calculated\ stress\ in\ member = fbim$	
fl $= calculated\ stress\ due\ to\ self\ weight\ load\ only$ $= Mdl/Zb$	
Where $np = Eps/Ec$ and $\rho p = Aps/(b * dp)$	
$dp = ec + yt + ds$	
$Therefore\ net\ deflection\ at\ transefr\ is$	
$\boldsymbol{\delta nett = \delta i + \delta dl}$	Equation 4-47

4.6.2.2 After un-shored slab is cast

3. Due to Super imposed dead load.

a. If the section is un-cracked

$\delta sl = \frac{5wsl * L^4}{384 * Ec * Ig}$	Equation 4-48

b. If the section is cracked

$\delta sl = \frac{5wsl * L^4}{384 * Ec * Ie}$	Equation 4-49
From Equation 2-52, 2-53 and 2-55	
$Ie = Icr + (\frac{Mcr}{Ma})^3 * (Ig - Icr) \leq Ig$	
$\left(\frac{Mcr}{Ma}\right) = \left(1 - \frac{ftl - fr}{fl}\right)$	
$ftl = total\ calculated\ stress\ in\ member = fbsm$	
$fl = Msl/Zb\ if\ the\ section\ cracks\ in\ this\ stage$ $fl = (Mdl + MSd)\ if\ the\ section\ cracked\ in\ the\ transfer\ stage$	
Where $np = Eps/Ec$ and $\rho p = Aps/(b * dp)$	
$dp = ec + yt + ds$	
$Therefore\ net\ deflection\ after\ un-shored\ is\ cast\ is$	
$\boldsymbol{\delta nets = \delta nett + \delta sl}$	

4.6.2.3 Immediate service load deflection.

4. Due to live load
   a. If the section is un-cracked

$\delta ll = \frac{5wll * L^4}{384 * Ec * Igc}$	Equation 4-50

a. If the section is cracked

$\delta ll = \frac{5wll * L^4}{384 * Ec * Ie}$	Equation 4-51
From Equation 2-52, 2-53 and 2-55	
$Ie = Icr + (\frac{Mcr}{Ma})^3 * (Igc - Icrc) \leq Ig$	
$\left(\frac{Mcr}{Ma}\right) = \left(1 - \frac{ftl - fr}{fl}\right)$	
$ftl = total\ calculated\ stress\ in\ member = fbem$	
$fl = Mll/Zb\ if\ the\ section\ cracks\ in\ this\ stage$ $fl = (Msl + Mll)/Zb\ if\ the\ section\ cracks\ after\ the\ slab\ is\ cast$ $\boldsymbol{fl = (Mdl + Msl + Mll)\ if\ the\ section\ cracked\ in\ the\ transfer\ stage}$	
Where $np = Eps/Ec$ and $\rho p = Aps/(b * dp)$	
$dp = ec + yt + ds$	
$Therefore\ net\ immediate\ defection\ at\ service\ is$	
$\boldsymbol{\delta netl = \delta nets + \delta ll}$	

4.6.2.4 Long term deflection

Using table 2-3 of PCI multipliers

- Net deflection at transfer ***(δnet) = δnett***
- Net erection deflection before superimposed dead load ***(δne)***

$\delta ne = \delta pi * 1.8 + \delta dl * 1.85$	Equation 4-52

- Net erection deflection immediately after superimposed dead load (***δnes)***

$\delta nes = \delta ne + \delta sl$	Equation 4-53

- Net erection deflection immediately after service load (***δnel)***

$\delta nel = \delta nes + \delta ll$	Equation 4-54

- Net final deflection before superimposed dead load ***(δnf)***

$\delta nel = \delta pi * 2.2 + \delta dl * 2.4$	Equation 4-55

- Net final deflection immediately after superimposed dead load (***δnfs)***

$$\delta nef = \delta nf + \delta sl * 3 \quad \text{Equation 4-56}$$

- Net final deflection immediately after service load (***δnfl)***

$$\delta nfl = \delta nfs + \delta ll \quad \text{Equation 4-57}$$

Note: - The upward deflection (camber) is assigned as negative value and the downward deflection is assigned as positive value.

4.6.3 Example 4.1

I. DATA

	Input
The cubic strength of concrete for precast beam (fc)	50.00 Mpa
The tensile strength of tendon (Fpk)	1862.00 Mpa
Expected prestress loss	18.00%
The cubic strength of concrete for cast insitu slab (fcs)	30.00 Mpa
Span Length L	18.29 m
Beam type	6
Depth of cast insitu slab (ds)	250.00 mm
center to center distance between girder (ccb)	2200.00 mm
Cantilever distance if it is edge girder (W)	1500.00 mm
Area of prestressing tendon (Aps) = Aps Minimum	4698.85 mm
eccentricity at support	115.00 mm
eccentricity at midspan	100.0 m

II. Material Properties

Concrete			Prestressing Reinf	
fc' =	50	Mpa	fpk =	1862.00 Mpa
fck= 0.8*fc'	40	Mpa	fpy = 0.9*fpk =	1675.80 Mpa
fc' (toping) =	30	Mpa	fpi= 0.75*fpk	1396.50 Mpa
fr = 0.623*√fc' =	4.405275247	Mpa	Eps =	196500.00 Mpa
Ecp = 4730*√fc'	33234.01872	Mpa		
Ecc = 4730*√fc's	25742.9602	Mpa	Prestress loss	18%
fct= 0.6*fck	24	Mpa	n=	82%
$ftt = 0.21*(fck)^{(2/3)}$	2.4561749	Mpa	Aps min =	4698.85 mm2
fcw = 0.5*fct	20	Mpa	Aps =	4698.85 mm2
ftw = 0.75*ftt	1.842131175	Mpa	Pi =	6561938.50 N
ns = Ecp/Ecc =	0.774596669			

III. Geometric Properties

Type of Beam	PCB 6		
ds =	250.00 mm		
ccb=	2200.00 mm		
W =	1500.00 mm		
	Precast	Composite	
Ac =	699966	1.349966	mm^2
Ic=	3.02E+11	6.59E+11	mm^4
Cb =	920.2659077	1413.6688	mm
Ct=	899.7340923	406.33118	mm
Zb =	3.28E+08	4.66E+08	mm^3
Zt =	3.36E+08	1.62E+09	mm^3
Zts =		9.61E+08	mm^3
Zbs=		1.55E+09	mm^3
Span =	18.29 m		

Depth of Slab Ds = 0.25 m
Effective Length Le = 2.6m
Transformed effective length let= Le*Ecp/Ecs = 2.01395m

(Impact Factor) IF = 1+1.2/√L = 1.28

IV. Load

Ml = From Table 15.15 of AREMA MRE Manual 3522.82 KNm

Wdl = 17.50 KN/m
Wsl= 41.70 KN/m
Wll = 84.25 KN/m

Mdl = 731.74 KNm
Msl = 1743.51 KNm
Mll = IF*Ml = 4511.29852KNm
Mtot= 6986.55 KNm

V. Other

ec=	115.00 mm	Out of a range	
ee=	100.00 mm	Ok	
finf = Mtot/(n*Zb)-ftw/n =	23.70	Mpa	
fsup = -Mdl/zt-ftt =	-4.64	Mpa	

Minimum prestressing force

Pimin = Ac*(zb*finf+zt*fsup)/(zb+zt) 6561938.50 N

apsmin= pimin/fpi 4698.85 mm2

emaxp= 716.97 mm

Allowable eccentricity

	At support		At mid span	
emax =	605.45	mm	716.96	mm
emin =	-581.45	mm	716.96	mm

VI. Deflection Calculation

Effective Ie method for short term and PCI Method for long term deflection

Allowable Stress

fc =	24.00 Mpa	
fcw =	20.00 Mpa	
fr =	4.41 Mpa	
ηp =	Ep/Ecp =	5.84

1.00 Mid Span Section Stress

fpi =	1583.63 Mpa	
ec=	115.00 mm	
Pi =	As*fpi =	6561938.50 N
Md=	Wd*L*L/8	731.74 KNm

a At transfer

ft = pi/Ac - Pi*ec/Zt+Md/Zt

9.36 Mpa Ok

fb = pi/Ac + Pi*ec/Zb-Md/Zb

9.44 Mpa OK

b After unshored slab is cast

At this load level

fpe = 1145.13 Mpa

Pe = As*fpe = 5380789.57 N

Wsd= 41.70 Kn/m

Msd = Wsd*L*L/8 = 1743.51 KNm

ft= pe/Ac - Pe*ec/Zt+Md/Zt+Msd/zt

13.22 Mpa Ok

fb = pe/Ac +Pe*ec/-Md/Zb-Msd/Zb

2.03 Mpa OK

c At service load for precast section

Ml = Wl*L*L/8

4511.30 KNm

ft= pe/Ac - Pe*ec/Zt+Md/Zt+Msd/zt +Ml/Ztp

16.00 Mpa OK

fb = pe/Ac + Pe*ec/Zb-Md/Zb-Msd/Zbp

-7.64 Mpa Cracked

c Composite Slab stress

Precast **=**

Ecp = $4734*(fc')^{1/2}$

33474.44 Mpa

Insitu **=** Eci = 25929.19 Mpa

η = Eci/Ecp 0.77

fcst= η*Ml/Zct= 3.64 Mpa
fcsb = η*Ml/Zcb= 2.26 Mpa

2.00 support Section Stress

ee = 100.00 mm

a At transfer

ft = pi/Ac - Pi*ee/Zt
7.42 Mpa >ftt Ok

fb = pi/Ac + Pi*ee/Zb
11.37 Mpa <fct OK

b After unshored slab is cast

At this load level

Pe = As*fpe = 5,380,789.57

ft= pe/Ac - Pe*ee
7.36 Mpa Ok
fb = pe/Ac + Pe*ee/Zb
9.33 Mpa OK

c At service load for precast section

ft= pe/Ac - Pe*ee/Zt
7.36 Mpa OK
fb = pe/Ac - Pe*ee/Zb
9.33 Mpa Ok

`

summary of stress (Mpa)		Mid Span		Support	
At Transfer	Top	9.36	OK	7.42	OK
	Bottom	9.44	OK	11.37	OK
After unshored slab	Top	13.22	OK	7.36	OK
	Bottom	2.03	OK	9.33	OK
At Service	Top	16.00	OK	7.36	OK
	Bottom	-7.64	Cracked	9.33	OK

3.00 Camber and Deflection calculation at transfer

Eci = 29,937.92

Ec = 33,474.44

due to initial prestress only

$\delta i = pi * L^2/(48*Eci*Ig)*(-5*e_c+e_e)$

-2.40 mm Upward

due to self-weight

$\delta d = 5*Wd*l\text{^}4/(384*Eci*Ig)$

2.82 mm Downward

$\delta net = \delta i - \delta d =$

0.42 mm Downward

3.00 Immediate Service Load Deflection

Effective Ie Method

fr = 4.41 Mpa

dp = ec + yt+ds

1149.73 mm

$\rho p =$ Aps/(bdp) 1.86E-03

$Icr = \eta p * Aps*(dp\text{^}2)*(1-1.6*(\eta p*\rho p)\text{^}(1/2)$

3.02E+10 mm^4

Mcr/Ma 1- (ftl - fr)/fl 0.67

$(Mcr/Ma)^3$ 0.30

$Ie = (Mcr/Ma)^3(Ig - Icr)+Icr \leq Ig$ 1.10E+11 mm4

Ie = 1.10E+11 mm4

Before Service load and crack δsd due to the cast insitu slab

$$\delta sd = (5*wsd*l^4)/(384*Ecp*Ig)$$

6.01 mm

After Service load and crack δl due to the service load

$$\delta l = (5*wld*l^4)/(384*Ecp*Ie)$$

33.20 mm

5.00 Long Term Deflection (Camber) By PCI multiplier

-PCI multipliers C, for long-term camber and deflection		
	Without composite topping	With composite topping
At Erection		
Deflection (downward) component-apply to the elastic deflection due to the member weight at release of prestress	1.85	1.85
Camber (upward) component-apply to the elastic camber due to pre-stress at the time of release of prestress	1.8	1.8
Final		
Deflection (downward) component-apply to the elastic deflection due to the member weight at release of prestress	2.7	2.4
Camber (upward) component-apply to the elastic camber due to pre-stress at the time of release of prestress	2.45	2.2
Deflection (downward)--apply to the elastic deflection due to the superimposed dead load only	3	3
Deflection (downward)-apply to the elastic deflection caused by the composite topping		2.3

	Deflection/Camber (mm)					
Load	At Transfer	PCI Multiplier	Erection	PCI Multiplier	δ Final	
Prestressing Load Only (P)	**δi = -2.40**	1.80	-4.32	2.20	-9.51	
Self Weight (Wd)	2.82	1.85	5.21	2.40	12.52	
δnet	**δnt = 0.42**	**Downward**	0.89	Downward	3.01	Downward
Super Imposed Dead Load (Wsd)			6.01	3.00	18.02	
δnet =			**δnes = 6.90**	**Downward**	21.03	Downward
Service Load (Wl)			33.20		33.20	
δnet	0.42	Downward	40.10	Downward	**δnfl = 54.23**	**Downward**

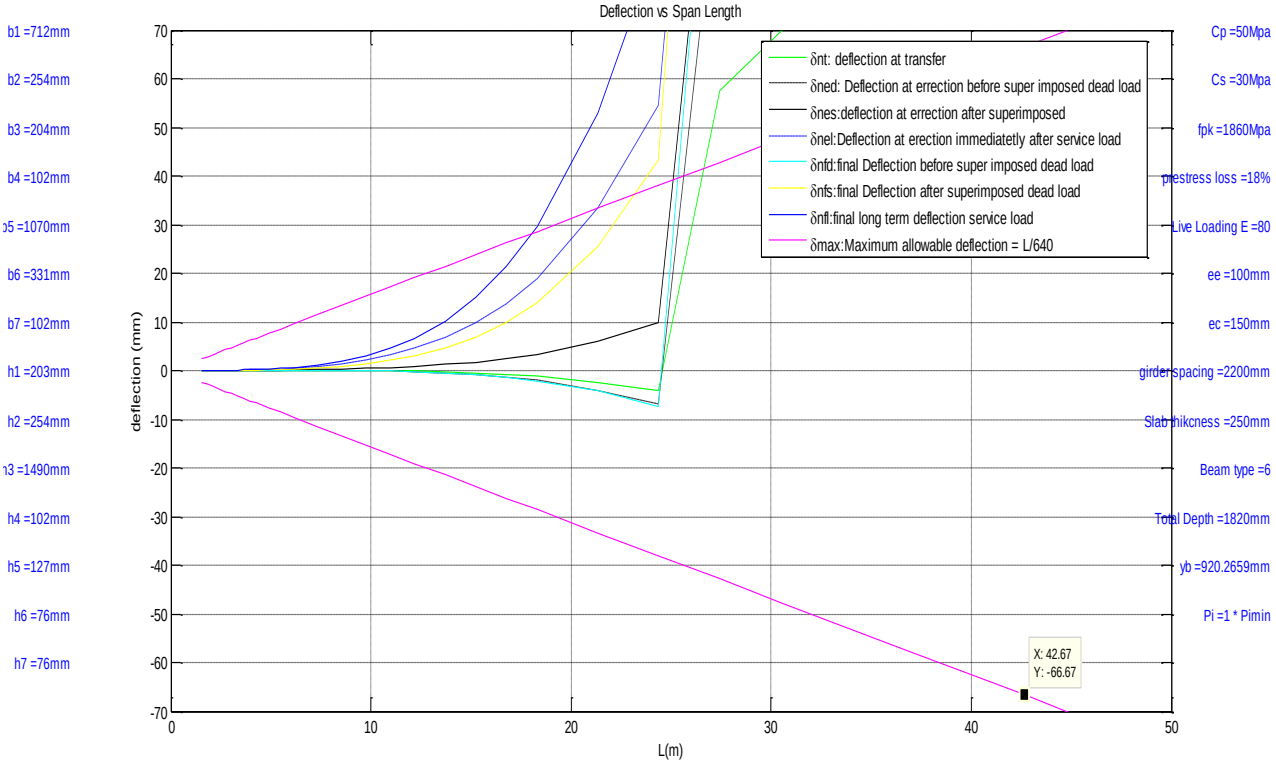


Figure 4-8 Deflection Chart for AASHTO Type 6 I beam.

5 Result and Discussion

As I was developing the chart for different beams something strange happened. That is the deflection vs. span length chart will show some unexpected behavior after a certain span length.it suddenly changes is dimension and direction in unanticipated manner. I was curious what makes this this sudden shift, so I try to change the eccentricity of the tendon profile. And for that I have to know the maximum and the minimum allowable range both at the support and mid span. I write the code for those parameters and draw a chart. Amazingly the maximum and minimum limits of the eccentricity will cross each other at some point both for mid span and Support section that means there is a span length for which the maximum limit become less than the minimum, which is practically impossible.

In this stage I doubt my code and start to calculate manually and using Microsoft excel template which I developed it for checking purpose. But the result was still the same. That means the section has a span length limit for a given load beyond which it will no longer work. Or it will fail if we chose to build it with some eccentricity.

The reason behind the maximum span length limit is to avoid tensile failure of concrete at mid span. In which for a given prestressing force and given self-weight the maximum is eccentricity is limited to avoid the tensile failure at the top and the compressive failure at the bottom during transfer. But in this case the tensile strength of concrete always dictate the maximum possible eccentricity. In other words the maximum eccentricity is set to avoid tensile failure at the top during transfer. On the other hand for a given prestressing force and a given service load the minimum eccentricity is set to avoid the tensile failure at the bottom and compressive failure at the top during service. Yet again the tensile strength of concrete comes to dictate the minimum possible eccentricity, which means the minimum possible eccentricity is set to avoid the tensile failure at the bottom during service. However after a certain span length it become impossible to avoid tensile failure at the one surface while maintaining the safety of the other surface. It means after a certain span length the eccentricity needed to avoid the tensile failure at the top during transfer will be less than the minimum allowable eccentricity required to avoid the tensile failure at the bottom during service and vice versa. In addition to that the maximum allowable prestressing force becomes less than the minimum allowable span length after

Note the whole same thing happen for the support section too. Since it happens after the mid-span section it does not govern the maximum span length.

What I found interesting in this chart was this shift of the maximum and minimum eccentricity will happen also prior to some span length that is the maximum allowable eccentricity will be less than the minimum up to a certain span length. This means unlike in reinforced concrete

which has no minimum span length limit structurally speaking prestressed member has a minimum limit of span length due to the applied axial compression force.

The reason for the minimum span length limit is to avoid the crushing of concrete. As discussed in the above paragraph, for a given prestressing force and given self-weight the maximum is eccentricity is limited to avoid the tensile failure at the top and the compressive failure at the bottom during transfer. And in this case the maximum eccentricity is set to avoid compressive failure at the bottom during transfer (The tensile failure at the will also occur juts right before the minimum span length). And for a given prestressing force and a given service load the minimum eccentricity is set to avoid the tensile failure at the bottom and compressive failure at the top during service. In this case the minimum possible eccentricity is set to avoid the compressive failure at the top during service. However before a certain span length the eccentricity needed to avoid the compressive failure at the bottom during transfer will be less than the minimum allowable eccentricity required to avoid the compressive failure at the top during service and vice versa.

If one use the minimum prestressing force as an applied force the maximum allowable mid span eccentricity (***emaxm***),the minimum allowable mid-span eccentricity (***eminm***) and the maximum mid-span eccentricity corresponding to the minimum prestressing force (***emaxc***) will be the same for a working range. Before or beyond that range the maximum allowable mi-span eccentricity (***emaxm***) will be less than the maximum mid-span eccentricity corresponding to the minimum prestressing force (***emaxc***) which in turn less than the minimum allowable mid-span eccentricity (***eminm***).

That is ***emaxm*** = ***eminm*** =***emaxc*** for the possible working range and

emaxm < ***emaxc*** < ***eminm*** before and beyond the possible working range.

Which means if one has to choose the minimum prestressing force as an applied prestressing force in prestressing design there is only one specific point that can use as the eccentricity of tendon at the mid span section.

By range I mean span length.

However If one chooses to use the applied prestressing force higher than the minimum value the maximum allowable mid span eccentricity (***emaxm***) will be higher than the minimum allowable mid-span eccentricity (***eminm***) for the working range and the maximum allowable mid span eccentricity (***emaxm***) will be lower than the minimum allowable mid-span eccentricity (***eminm***) outside of the working range.

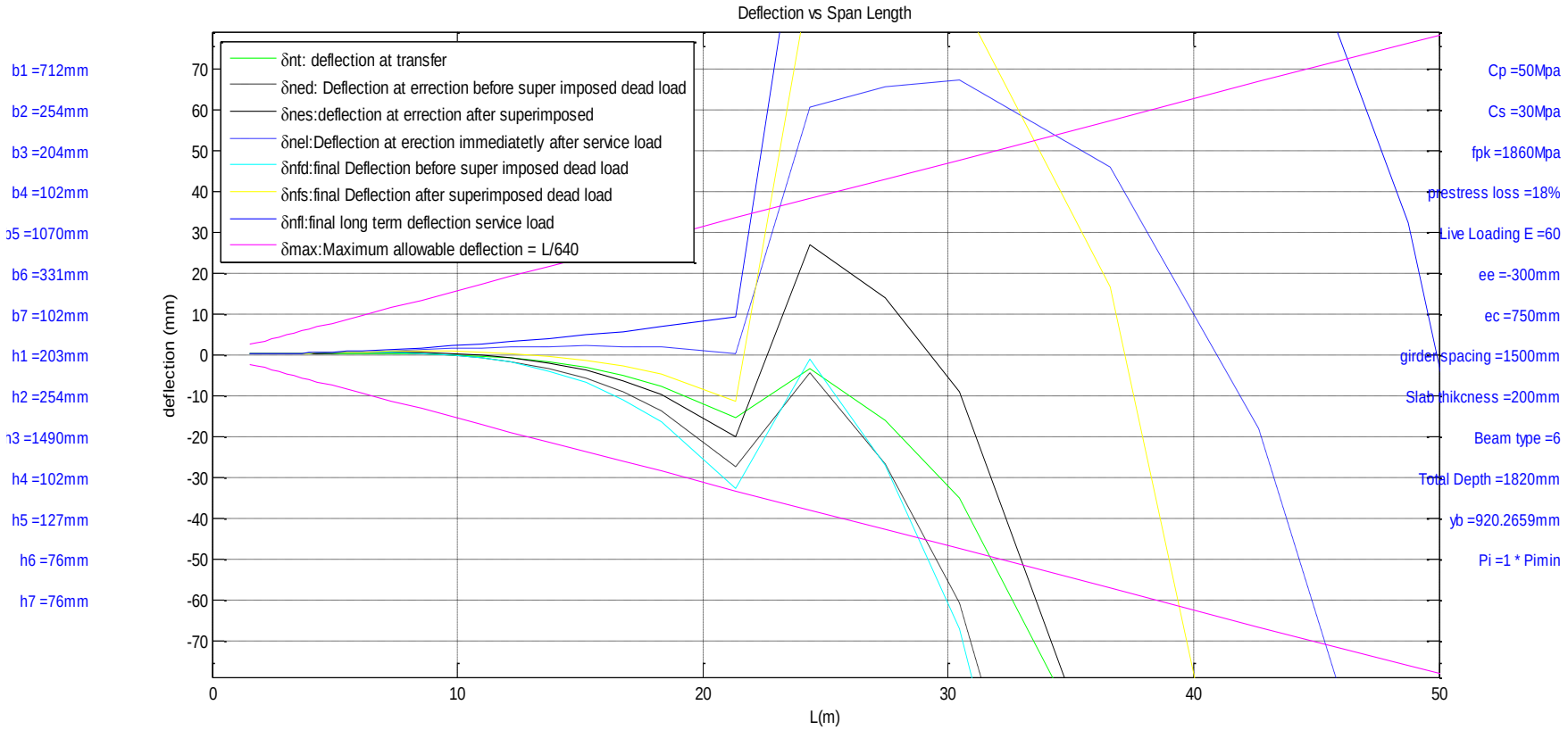


Figure 5-1 Typical deflection vs. Span Length Chart.

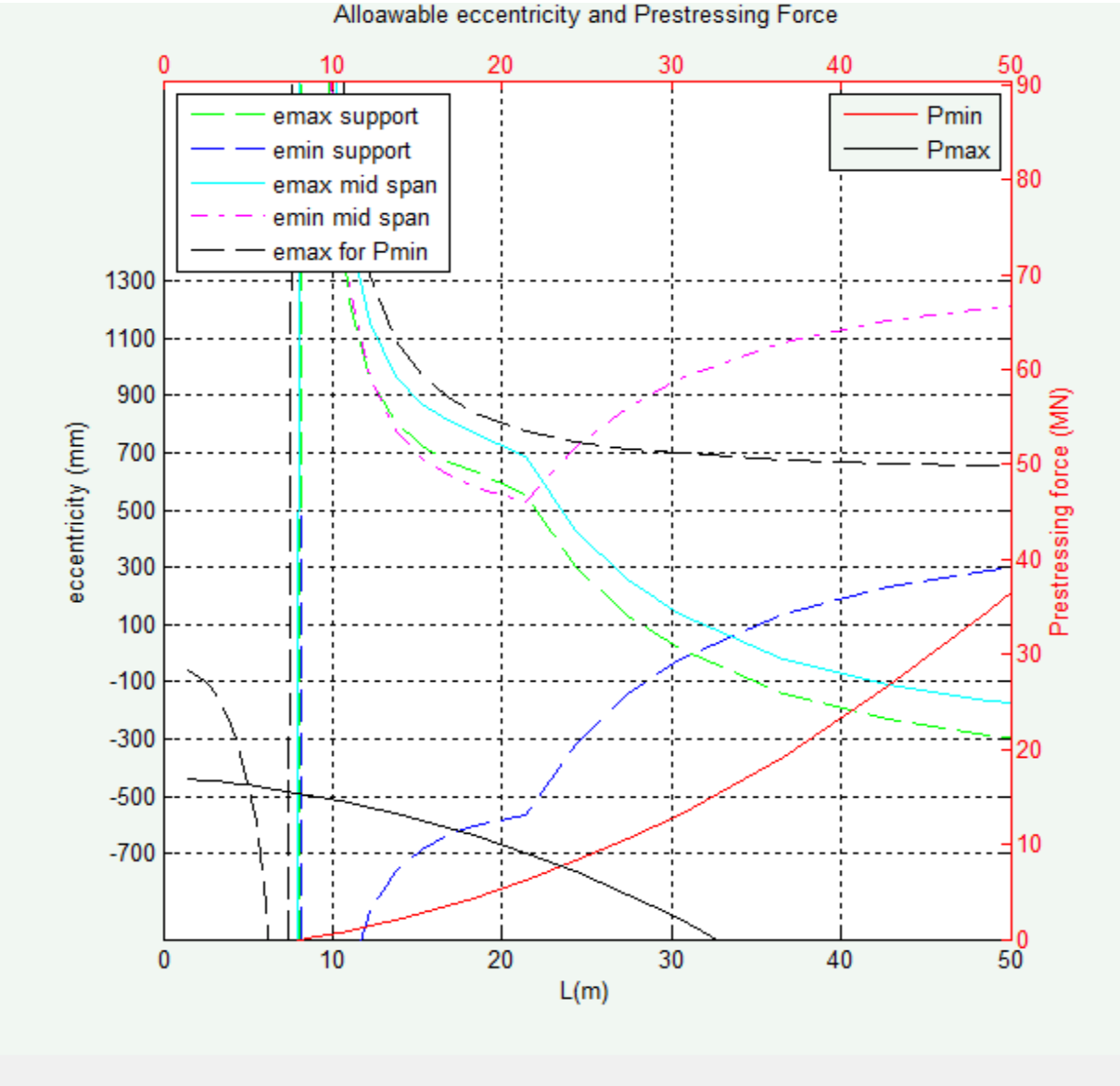


Figure 5-2 Allowable eccentricity and minimum prestressing force vs. span length for type 6 AASHTO I beam for Pi = 1.25*Pimin

As you can see in the above chart the maximum allowable mid-span eccentricity/emax mid-span (cyan color solid line) is higher than the minimum allowable mid-span eccentricity/emin mid-span (magenta broken line) in the range from 7.m – 19m. This means with this type of loading and material property AASHTO type 6 I beam works in the range of 7.0m to 19.0m.

When it comes to support eccentricity, the maximum allowable support eccentricity/emax support (the green dashed line) is again higher than the minimum allowable support

eccentricity/emin support (the blue dashed line) in the range about from 7m to 28m. Based on the support eccentricity allowable working range from 7m to 28m, however since the minimum range govern the working range of this beam is about form 7m to 19m. The above chart was developed for the ratio of applied prestressing force to the minimum prestressing force equal to 1.25 that is for Pi = 1.25*Pimin. Now let us see for Pi = Pimin

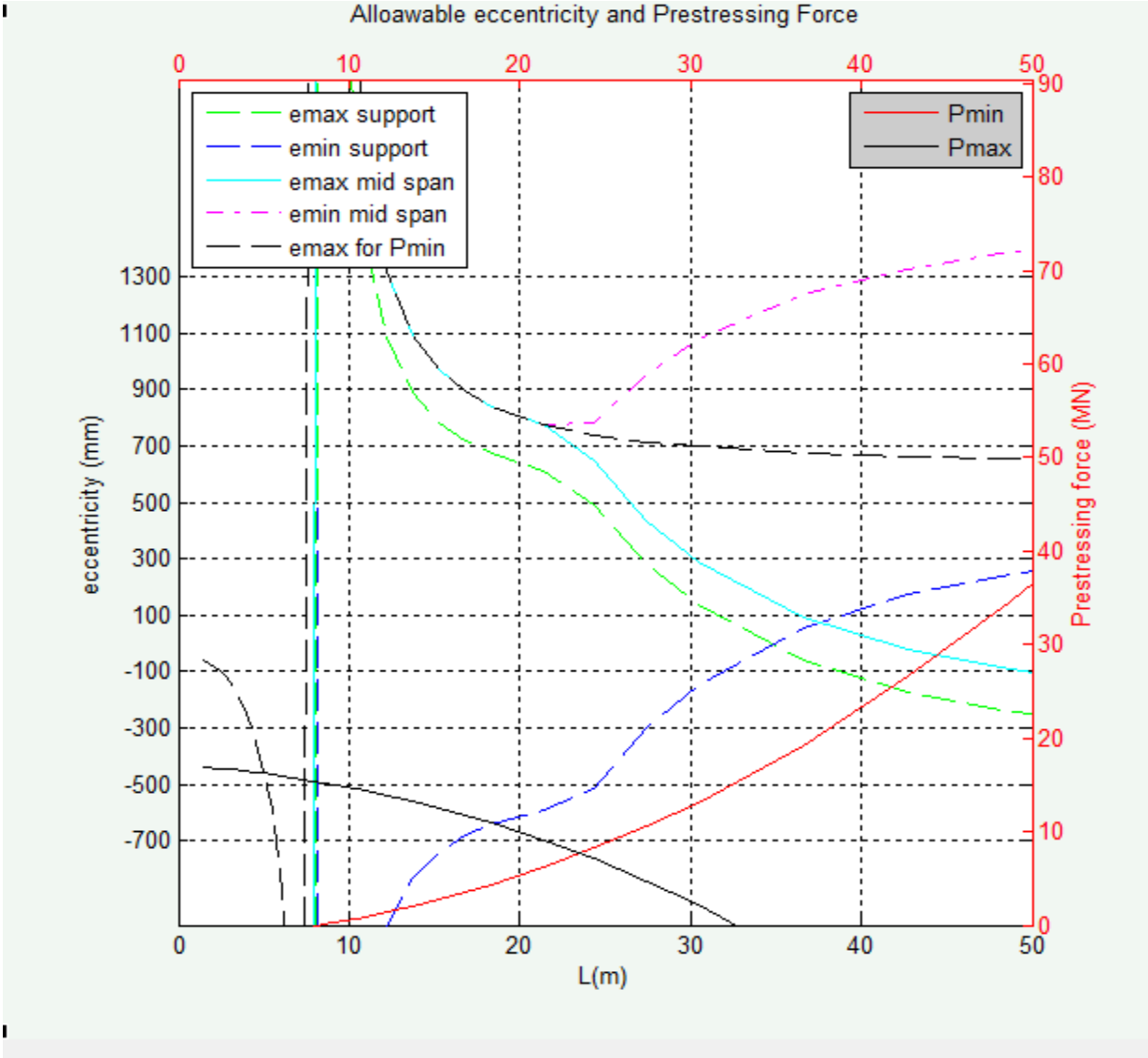


Figure 5-3 Allowable eccentricity and minimum prestressing force vs. span length for type 6 AASHTO I beam for Pi = Pimin

As you can see in the above graph the three eccentricities, maximum allowable mid span eccentricity (***emaxm***),the minimum allowable mid-span eccentricity (***eminm***) and the maximum

mid-span eccentricity corresponding to the minimum prestressing force (***emaxc***) is the same for a working range. That means there is one possible tendon profile at mid span if we are going to use the minimum possible prestressing force as the applied one.

As applied prestressing force increases the upward deflection (camber) increases proportionally and the overall downward deflection decreases; and vice versa.

When the mid-span eccentricity increases to the positive value the upward deflection (camber) increases proportionally and the overall downward deflection decreases, and vice versa.

When the support eccentricity increases to the positive value positive value the upward deflection (camber) decreases proportionally and the overall downward deflection increases; and vice versa.

When the support eccentricity increases in magnitude to the negative value the upward deflection (camber) increases proportionally and the overall downward deflection decreases; and vice versa.

6 Conclusion

The objective of the thesis is achieved in which a simple user friendly and versatile program is developed that plot a deflection chart for simply supported prestressed railway girder.

One can control the deflection or camber of the prestressed member with the magnitude of the applied load and varying the eccentricity of the tendon profile both at the support and at the mid-span. Especially at the support where we have a large range where we can fix the centroid of our tendon.

$$\delta = \frac{P * L^2}{48 * EI} * (-5 * e1 + e2)$$

And from the charts I developed the following points concluded as follows.

- As applied prestressing force increases the upward deflection (camber) increases proportionally and the overall downward deflection decreases; and vice versa.
- When the mid-span eccentricity increases to the positive value the upward deflection increases proportionally and the overall downward deflection decreases, and vice versa.
- When the support eccentricity increases to the positive value positive value the upward deflection decreases proportionally and the overall downward deflection increases; and vice versa.
- When the support eccentricity increases in magnitude to the negative value the upward deflection increases proportionally and the overall downward deflection decreases; and vice versa.

For a given load and a given cross-sectional geometry a prestressed member has not only a maximum span length limit but also a minimum span length limit in order to serve the intended function without failure. This minimum limit is due to the applied axial compression force and upward deflection. The applied prestressing axial force is counteracted by the self-weight of the member at transfer and the super imposed dead and the live (service) load at service. However below certain length the total weight of the member which increases with the span length may not be high enough to counteract the applied axial prestressing force and limit the stresses at the section of a beam to allowable stress limits. In which compression failure will occur either on the top or bottom surface of the member. This intern limits the minimum workable span length.
On the other hand, the maximum span length limit is governed by the applied service load and the downward deflection imposed by service load. After a certain span length the concrete will face tensile failure either on the top or bottom surface of a member.

7 Topics for Further Study

The following points could be interesting topic for further study.

i. Questioning ACI method of deflection calculation. One can use simulation or other method of deflection calculation and compare the results with the values developed on the chart. Especially on the long term method of deflection calculation ACI recommends using PCI method of computation. However PCI bridge manual itself question the accuracy of the method for composite member.

ii. Analyzing the effect of partial prestressing in deflection and overall design of prestressed member. That is the effect of adding non prestressing reinforcement in a prestressed member on the deflection calculation and overall design of a member.

iii. Providing a deflection chart for continues I beam. As this thesis is for simply supported it would be interesting what happens to those results when the beam is continuous.

8 Reference

1. **Benaim, Robert.** *The Design of Prestressed Concrete Bridges, Concepts and Principles* . London and New York : Tylor and Francis Group , 2008.

2. **Raju, N Krishna.** *Prestrssed Concrete, fourth edition.* New Delhi : Tata McGraw-Hill Publishing Company Limited, 2007.

3. *An Intoduction to Prestressed Concrete.* **A J Threlfall BEng, DIC.** s.l. : British Cement Association , 2002.

4. *Civil Engineering Design, Prestressed Concrete.* **Caprani, Dr. Colin.** 2006/07.

5. *Prestressed Concrete Design, (SAB 4323)* . **Ahmad, Assoc. Prof. Baderul Hisham.** s.l. : University Technolgy Malaysia.

6. ***Control of Deflection in Concrete Structure ACI 435R-95.*** **2000.**

7. *Building Code Requiremnts for Structural Concrete (ACI 318-08) and Commentary.* s.l. : American Concrete Institute, 2008.

8. *PCI Bridge Manual* . s.l. : Precast/Prestressed Concrete Institute, 1997.

9. *AREMA Manual for Railway Enigneering.* s.l. : American Railway Engineering and Maintainance of way Association. , 2010.

10. *Prestressed Concrete Railway Bridges, Lecture Note.* Asnake, Dr.

11. *AASHTO LRFD BRIDGE DESIGN SPECIFCATION* . s.l. : American Association of State Highway and Transportation Officials, 2012.

12. *AASHTO LRFD Bridge Design Specification Third Edition* . s.l. : American Asscoiation of State Highway and Transport Officials, 2005.

13. F K Kong and R H Evans. *Reinforced and Prestrssed Concrete, 3rd edition* . London : Chapman and HAll, 1993.

14. Brian H.Hahn and Daniel T.Valentine. *Essential Matlab for Engineers and Scientists, 5th edition.* s.l. : Elsevier, 2013.

15. http://www.mathworks.com. *http://www.mathworks.com.* [Online]

Appendix A: code for MATLAB Program Language

```
% Material
disp('insert the cubic strength of the prestressed concrete:in
Mpa' )
c=input('c=');
disp('insert the cubic strength of the cast insitu slab
concrete: in Mpa' )
cs = input('cs =');
disp('insert the the tensile strength of ptresstress tendon: in
Mpa ' );
fpk=input('fpk=');
Ep=196500;
disp('insert the expected prestress loss percentage:' );
prl=input('prl=');
disp('insert the ratio of applied force to the minimum
prestressing force (Pi/Pimn):' );
pff=input('pff=');
disp('choose the type of live loading: enter 80 for E-80 loading
and 60 for E-60 loading' );
cooperl = input('cooperE-');
eta=1-prl/100;
%concrete
gammac=25;
fck=0.8*c;
Ecp=4700*sqrt(c);
Ecs = 4700*sqrt(cs);
mrs = Ecp/Ecs;
mrp = Ep/Ecp;
fct=0.6*fck;
ftt=0.21*fck^(2/3);
fcw=0.5*fck;
ftw=0.75*ftt;
ftr=fcw+eta*ftt;
fbr=fct+eta*ftw;
fr=0.623*sqrt(c);

% prestressing Tendon
fpy=0.9*fpk;
fpit=0.8*fpk;
fpi=0.75*fpk;
```

```
fpew=0.8*fpi;
fpe=0.6*fpk;



%   GEOMETRY
disp('insert the type of the beam enter value 1 - 6 for AASHTO I
beams and 7 for custom I beam:' );
beamType=input('beamType=');

switch(beamType)
    case 1
       H = 712.00;
b1=407.00; b2=127.00; b3=153.00; b4=076.00; b5=305.00; b6 =0.0;
b7 =0.0;

h1=127.00; h2=127.00; h3=483.00; h4=076.00; h5=102.00; h6=0.0;
h7 =0.0;

    case 2
       H = 920.00;
b1=459; b2=153; b3=153; b4=076; b5=305; b6 =0.0; b7 =0.0;

h1=153; h2=153; h3=607; h4=76; h5=160; h6=0.0; h7 =0.0;

    case 3
        H = 1150.00;
b1=560; b2=190; b3=180; b4=115; b5=410; b6 =0.0; b7 =0.0;

h1=180; h2=190; h3=790; h4=115; h5=180; h6=0.0; h7 =0.0;

    case 4
        H = 1372.00;
b1=664; b2=230; b3=204; b4=153; b5=510; b6 =0.0; b7 =0.0;

h1=203; h2=230; h3=966; h4=153; h5=203; h6=0.0; h7 =0.0;

    case 5
        H = 1600.00;
b1=712; b2=254; b3=204; b4=102;  b5=1070; b6=331;  b7=102;

h1=203; h2=254; h3=1270; h4=102; h5=127; h6=076; h7=076;
    case 6
```

```
      H = 1820.00;
b1=712; b2=254; b3=204; b4=102; b5=1070; b6=331; b7=102;

h1=203; h2=254; h3=1490; h4=102; h5=127; h6=076; h7=076;
    otherwise
        bx=[4 12.7088 12.7088 9.6020 9.6020 10.8594 14.8983
14.8983 1.8106 1.8106 5.8494 7.0970 7.0970 4.0000 4];
        by=[4 4 6.483 9.5898 19.8938 21.0874 22.0170 23.5704
23.5704 22.0170 21.0874 19.8374 9.5898 6.4830 4];
        plot(bx,by);
        axis ([0 30 0 30])
        title ('I Beam')
        set(gca,'Xtick',0:40:40)
        set(gca,'Ytick',0:40:40)
        text(8.335,3,'------b1------
','fontsize',9,'HorizontalAlignment','center','color','blue')

text(13.0,5.22,'h1','fontsize',9,'HorizontalAlignment','left','c
olor','blue')
        text(13.5,4,'----
','fontsize',9,'HorizontalAlignment','right','color','blue')
        text(13.5,6.483,'----
','fontsize',9,'HorizontalAlignment','right','color','blue')
        text(11.15,6.483,'--b2--
','fontsize',9,'HorizontalAlignment','right','color','blue')

text(13.0,8.037,'h2','fontsize',9,'HorizontalAlignment','left','
color','blue')
        text(13.5,6.483,'----
','fontsize',9,'HorizontalAlignment','right','color','blue')
        text(13.5,9.6,'----
','fontsize',9,'HorizontalAlignment','right','color','blue')
        text(8.35,14.74,'--b3--
','fontsize',9,'HorizontalAlignment','center','color','blue')

text(10.75,14.74,'h3','fontsize',9,'HorizontalAlignment','center
','color','blue')
        text(11.2,19.89,'----
','fontsize',9,'HorizontalAlignment','right','color','blue')
        text(11,9.6,'----
','fontsize',9,'HorizontalAlignment','right','color','blue')
        text(10.23,21.09,'--b4--
','fontsize',9,'HorizontalAlignment','center','color','blue')
```

```
text(11,20.5,'h4','fontsize',9,'HorizontalAlignment','center','c
olor','blue')
        text(12.88,21.09,'--b6--
','fontsize',9,'HorizontalAlignment','center','color','blue')

text(14.9,21.55,'h6','fontsize',9,'HorizontalAlignment','center'
,'color','blue')
        text(15.8,22.02,'------
','fontsize',9,'HorizontalAlignment','right','color','blue')
        text(15.8,23.57,'------
','fontsize',9,'HorizontalAlignment','right','color','blue')

text(15.6,22.80,'h5','fontsize',9,'HorizontalAlignment','right',
'color','blue')
        text(8.36,24.22,'--b5--
','fontsize',9,'HorizontalAlignment','center','color','blue')
        text(15.1,21.09,'------
','fontsize',9,'HorizontalAlignment','right','color','blue')


        hold off
```

```
       disp('enter the dimesion of a beam in mm according to the 
following diagram')
disp('IIIIIIIIIIIIIIIIIIIIIIII b5 IIIIIIIIIIIIIIIIIIIIIIIII');
disp('IIIIIIIIIIIIIIIIIIIIIIIIIIIIIIIIIIIIIIIIIIIIIIIIIII h5');
disp('IIIIIIIIIIIIIIIIIIIIIIIIIIIIIIIIIIIIIIIIIIIIIIIIIIIII');
disp('      IIIIIIIIIIIIIIIIIIIIIIIIIIIIIIIIIIIIIIIIII xxx  ');
disp('          IIIIIIIIIIIIIIIIIIIIIIIIIIIIIIIIII      h6   ');
disp('              IIIIIIIIIIIIIIIIIIIIIIIIII      xxxxxx  ');
disp('                IIIIIIIIIIIIIIIIIIIIII            h4 
');
disp('                  IIIIIIIIIIIIIIIIII          xxxx    ');
disp('                      IIIIIIIIII              ');
disp('                      IIIIIIIIII              ');
disp('                      IIIIIIIIII         h3   ');
disp('                      IIIIIIIIII              ');
disp('                      IIIIIIIIII              ');
disp('                      IIIIIIIIII              ');
disp('                      IIIIIIIIII              ');
disp('                      IIIIIIIIII              ');
disp('                      IIIIIIIIII              ');
disp('                    IIIIIIIIIIIIII              xxxx 
');
disp('                 IIIIIIIIIIIIIIIIIIII       h2        ');
disp('          IIIIIIIIIIIIIIIIIIIIIIIIIIIIIIIIII  xxxx 
');
disp('IIIIIIIIIIIIIIIIIIIIIIIIIIIIIIIIIIIIIIIIIIIIIIIIIIIII ');
disp('IIIIIIIIIIIIIIIIIIIIIIIIIIIIIIIIIIIIIIIIIIIIIIIIIII h1');
disp('IIIIIIIIIIIIIIIIIIIIIIII b1 IIIIIIIIIIIIIIIIIIIIIIIII ');

disp('enter total depth')
H=input('H=');
disp('enter b1')
b1=input('b1=');
disp('enter h1')
h1=input('h1=');
disp('enter h2')
h2=input('h2=');
disp('enter h4')
h4=input('h4=');
disp('enter h5')
h5=input('h5=');
disp('enter b5');
b5 = input('b5=');
```

```
disp('enter h6')
h6 = input('h6=');
b2=h2;
b3 = b1-2*b2;
h3 = H - h1 -h5;
b4 =h4;
b6 = (b5 -b3-2*b4)/2;
h7 =h6;
b7 = b4;
end

% calculation
a1 = b1*h1;
a2=b2*h2;
a3=b3*h3;
a4=b4*h4;
a5=b5*h5;
a6 = b6*h6;
a7 = 2*b7*h7;


ac =(a1+a2+a3+a4+a5+a6+a7);

y1=h1/2;
y2=h1+h2/3;
y3=h1+h3/2;
y4=H-h5-h7-h4/3;
y5=H-h5/2;
y6 =H -h5-h6/3;
y7= H - h5 -h7/2;



AY = a1*y1+a2*y2+a3*y3+a4*y4+a5*y5+a6*y6+a7*y7;
Y = AY/ac;
yb = Y;
yt = H-yb;
Io =
(b1*h1^3/12+b2*h2^3/18+b3*h3^3/12+b4*h4^3/18+b5*h5^3/12+b6*h6^3/
18+b7*h7^3/6);
Ig = Io + (a1*(y1-Y)^2+a2*(y2-Y)^2+a3*(y3-Y)^2+a4*(y4-
Y)^2+a5*(y5-Y)^2+a6*(y6-Y)^2+a7*(y7-Y)^2);
zt = Ig/yt;
```

```
zb = Ig/yb;

epaxis = roundn((yb+500),2);
enaxis = roundn((-yt+200),2);

% Tendon profile input
str=['practical limit of eccentricity from the neutral axis in
mm',num2str(-yt), 'and' ,num2str(yb)];
disp(str)
disp('enter the ecentricity at support in mm');
ee = input ('ee =');
disp('enter the ecentricity at midspan in mm');
ec = input ('ec =');
disp('insert the center to center disatnce beteween girder:mm'
);
ccb=input('ccb=');
disp('insert the depth of cast in-situ slab in mm:' );
ts=input('ts=');
disp('insert 1 if the beam is edge beam and 0 if it is inetrior
beam:' );
edgeGirder=input('edgeGirder=');

if(edgeGirder==1)
    disp('insert the cantilever edge section in mm:' );
    edg=input('edg=');
else
    edg=0;
end
wg=edg;
 if (wg~=0)
wsd=10.625+25*(ts/1000)*((ccb/2000)+edg/1000)+19*0.3*(edg/1000+c
cb/2000);
 else
     wsd=10.625+25*(ts/1000)*(ccb/1000)+19*0.3*ccb/1000;
 end
wsl = wsd;
wdl=25*ac/(1e6);

%hold on
grid on


p=0
```

```
% length array for copper e-80 loading
% L =[1.52 1.83 2.13 2.44 2.74  3.05 3.35 3.66 3.96 4.27 4.88
5.49 6.10 7.32 8.53 9.75 10.97 12.19 13.72 15.24 16.76 18.29
21.34 24.38 27.43 30.48  36.58 42.67 48.77 54.86 60.96 76.20
91.44 106.68 121.92];
% ml =[67.79 81.35 94.91 108.46 127.30 152.53 178.10 216.93
257.60 298.28 379.62 460.97 559.27 773.38 991.06 1234.93 1487.72
1777.86 2170.91 2578.46 3027.64 3522.10 4630.06 5855.56 7238.75
8739.89 12507.80 16820.05 21568.07 26671.30 32148.73 47612.98
66163.04 88194.79 113616.04];
for i=1:1:35
    % Gemoetry for composite section
          t = 1*L(1,i)
        if (wg~=0)
            fw = max(b5/4,b3/2);
            le = ccb/2+min(L(1,i)*(1/8),min(6*ts+fw,wg));
        else

            fw = max(b5/2,b3);
            le = min( L(1,i)*(1/4), min(12*ts+fw,ccb));
        end
        lef = mrs*le;
        % MAximum allowable defelection
        dmax(1,i)= L(1,i)/640*1000;
        cmax(1,i)= L(1,i)/640*-1000;

     % Gemoetry for composite section

       b8 = lef; h8 = ts;
       a8 = b8*h8;
       y8 = H + h8/2;
       acc = ac+a8;
       Yc = (AY+a8*y8)/acc;
       ybg = Yc;
       ytg = H - ybg;
       yts = H+ts-ybg;

       Ioc = Io + (b8*h8^3)/12;
       Igc = Ioc +(a1*(y1-Yc)^2+a2*(y2-Yc)^2+a3*(y3-
Yc)^2+a4*(y4-Yc)^2+a5*(y5-Yc)^2+a6*(y6-Yc)^2+a7*(y7-
Yc)^2+a8*(y8-Yc)^2);

       ztp = Igc/ytg;
```

```
zts = Igc/yts;
zbp = Igc/ybg;
zbs = ztp;

% Moment
%Impact factor for live load
 if (L(1,i)<=4)
   k=1.6;
 elseif (L(1,i)<=39)
   k=1+(120/sqrt(L(1,i)))/100;
 else
   k=1.2;
 end;

% Momnet
 mdl= (wdl*L(1,i)^2/8)*1e6;
 msl= (wsd*L(1,i)^2/8)*1e6;
 mlt= k*ml(1,i) ;
 wl = 8*mlt/(L(1,i)^2);
 if (cooperl == 80);
     lf = 1;
 else
     lf =3/4;
 end;
 wll =wl*lf;
 mll = wll*L(1,i)^2/8*1e6 ;
 mtot=mdl+msl+mll;


%Minmum prestressing force and the corresponding maximum
%eccentricity.

finf = mtot/(eta*zb)-ftw/eta;
fsup = -mdl/zt - ftt;

Pimin = ac*(zb*finf+zt*fsup)/(zb+zt);
 asmin = Pimin/fpi;
 emaxp = zb*zt*(finf -fsup)/(ac*(finf*zb+fsup*zt));
Pmin(1,i) = Pimin/(1e6);
emaxc(1,i) =emaxp;
pi = pff*Pimin;
Aps =pi/fpi;
pe = eta*pi;
```

```
      Pimax = ac*(zb*fct+mdl+(1/eta*(zt*fcw-mtot)))/(zb+zt);
      Pmax(1,i) = Pimax/(1e6);
      Paxis(1,i) =0.45*Pimin/(1e6);
         %TENDON ECCENTRCITY AT THE END
     emaxs(1,i)=min(zt*ftt./pi+zt./ac,zb*fct./pi-zb./ac);
     emins(1,i)=max(-zt*fcw./(eta*pi)+zt./ac,-zb*ftw./(eta*pi)-
zb./ac);

     %TENDON ECCENTRCITY AT THE CENTER
     emaxm(1,i)=min(zt*ftt./pi+zt./ac+mdl./pi,zb*fct./pi-
zb./ac+mdl./pi);
     eminm(1,i)=max(-zt*fcw./(eta*pi)+zt./ac+mtot/(eta*pi),-
zb*ftw./(eta*pi)-zb./ac+mtot./(eta*pi));


       %A. AT TRANSFER
         % Mid Span
       ftim=pi/ac-pi*ec/zt+mdl/zt;
       fbim=pi/ac+pi*ec/zb-mdl/zb ;
         % At Support
       ftis=pi/ac-pi*ee/zt;
       fbis=pi/ac+pi*ee/zb;

       if((ftim>fct)||(fbim>fct)||(ftis>fct)||(fbis>fct))
           % concrete has crushed return 0
           caa=2;
       elseif((ftim<-fr)||(fbim<-fr)||(ftis<-fr)||(fbis<-fr))
          % concrete has cracked ..EFFECTIVE Ie METHOD
           caa=1;

       else
           %((-fr<=ftim<=fct) && (-fr<=fbim<=fct) && (-
fr<=ftis<=fct) &&
           %(-fr<=ftis<=fct))
           %ELASTIC METHOD
            caa=0;
       end

        %B. AFTER UN-SHORED SLAB IS CAST
         % Mid Span
       ftsm=pe/ac-pe*ec/zt+(mdl+msl)/zt;
       fbsm=pe/ac+pe*ec/zb-(mdl+msl)/zb ;
          % At Support
```

```
        ftss=pe/ac-pe*ee/zt;
        fbss=pe/ac+pe*ee/zb;

        if((ftsm>fct)||(fbsm>fct)||(ftss>fct)||(fbss>fct))
            % concrete has crushed return 0
            cab=2;
        elseif((ftsm<-fr)||(fbsm<-fr)||(ftss<-fr)||(fbss<-fr))
           % concrete has cracked ..EFFECTIVE Ie METHOD
            cab=1;

        else
            %((-fr<=ftsm<=fct) && (-fr<=fbsm<=fct) && (-
fr<=ftss<=fct) &&
            %(-fr<=ftss<=fct))
            %ELASTIC METHOD
             cab=0;
        end

        %c. AT SERVICE LOAD
        ftem=pe/ac-pe*ec/zt+(mdl+msl)/zt+mll/ztp;
        fbem=pe/ac+pe*ec/zb-(mdl+msl)/zb-mll/zbp ;

        if((ftem>fct)||(fbem>fct))
            % concrete has crushed return 0
            cac=2;
        elseif((ftem<-fr)||(fbem<-fr))
           % concrete has cracked ..EFFECTIVE Ie METHOD
            cac=1;

        else
            %((-fr<=ftem<=fct) && (-fr<=fbem<=fct))
            %ELASTIC METHOD
             cac=0;
        end

        %DEFLECTION  AND CAMBER CALCULATION.
         di(1,i)=(pi*1e6*L(1,i)^2*(-5*ec-ee))/(48*Ecp*Ig);
         Idi(1,i) = -1*di(1,i);
               b=ccb;
              dpp = ec+yt;
              dpc=ec+yt+ts;
              gamapp=Aps/(b*dpp);
              gamapc=Aps/(b*dpc);
```

```
            Icrp=mrp*Aps*dpp^2*(max(1-1.6*sqrt(mrp*gamapp),0));
            Icrc=mrp*Aps*dpc^2*(max(1-1.6*sqrt(mrp*gamapc),0));

        %DECISION A
      if( caa==0)
        ddl(1,i)=(5*wdl*L(1,i)^4)/(384*Ecp*Ig)*1e12;

             if(cab==0)
                 dsl(1,i) = (5*wsd*L(1,i)^4)/(384*Ecp*Ig)*1e12;

                     if(cac==0)
                           dll(1,i) =
(5*wll*L(1,i)^4)/(384*Ecp*Igc)*1e12;
                     else(cac==1)
                        fll = mll/zbp;
                          mr =max(1-(fbem-fr)/fll,0);
                         Ie = min(mr^3*(Igc -Icrc)+Icrc,Igc);
                         dll(1,i) =
(5*wll*L(1,i)^4)/(384*Ecp*Ie)*1e12;
                     %else
                     %dll = 0;
                     end
             else
                 %(cab==1)
                     fsl = msl/zb;
                     mr =max(1-(fbsm-fr)/fsl,0);
                     Ie = min(mr^3*(Ig -Icrp)+Icrp,Ig);
                     dsl(1,i) =
(5*wsl*L(1,i)^4)/(384*Ecp*Ie)*1e12;
                     %if(cac==2)
                         %dll = 0;
                     %else
                         fll = mll/zbp;
                          mr =max(1-(fbem-fr)/fll,0);
                         Ie = min(mr^3*(Igc -Icrc)+Icrc,Igc);
                         dll(1,i) =
(5*wll*L(1,i)^4)/(384*Ecp*Ie)*1e12;
             end       %end
             %else
                 %dsl=0;
                % dll =0;
             %end
```

```
     else
         %(caa==1)
             fdl = mdl/zb;
              mr =max(1-(fbim-fr)/fdl,0);
                    Ie = min(mr^3*(Ig -Icrp)+Icrp,Ig);
                    ddl(1,i) =
(5*wdl*L(1,i)^4)/(384*Ecp*Ie)*1e12;
                    %if(cab==2)
                     %   dsl =0;
                      %  dll =0;
                    %else
                        fsl = msl/zb;
                          mr =max(1-(fbsm-fr)/fsl,0);
                    Ie = min(mr^3*(Ig -Icrp)+Icrp,Ig);
                    dsl(1,i) =
(5*wdl*L(1,i)^4)/(384*Ecp*Ie)*1e12;


                   % if(cac==2)
                       % dll =0;
                   % else
                        fll = mll/zbp;
                         mr =max(1-(fbem-fr)/fll,0);
                        Ie = min(mr^3*(Igc -Icrc)+Icrc,Igc);
                        dll(1,i) =
(5*wll*L(1,i)^4)/(384*Ecp*Ie)*1e12;
                    %end
                   % end
         %else
             %if caa ==2
          %   ddl = 0;
           %  dsl =0;
            % dll =0;
     end

  % if((caa==2)||(cab==2)||(cac==2))
      % dnt= 0;
       %dned =0;
 %      dnes =0;
  %     dnel =0;
   %    dnfd=0;
    %   dnfs=0,
     %  dnfl=0;
```

```
  % else
   % Net Deflection at transfer
   dnt(1,i) = di(1,i)+ddl(1,i);
   % Net Deflection at errection before super imposed dead load
   dned(1,i) = 1.8*di(1,i)+1.85*ddl(1,i);
   % Net Deflection at erection immediately after superimposed
dead load
   dnes(1,i) =(1.8*di(1,i)+1.85*ddl(1,i)) + dsl(1,i);
   % Net Deflection at erection immediatetly after service load
   dnel(1,i) =  dnes(1,i) + dll(1,i);
   % Net final Deflection before super imposed dead load
   dnfd(1,i) = 2.2*di(1,i) + 2.4*ddl(1,i);
   % Net final Deflection after superimposed dead load
   dnfs(1,i) = dnfd(1,i) + 3*dsl(1,i);
   % Net final deflection at service load
   dnfl(1,i) = 2.2*di(1,i) + 2.4*ddl(1,i) + 3*dsl(1,i)+
dll(1,i);

   %end

       demin(1,i) =
min(dnt(1,i),min(dned(1,i),min(dnes(1,i),min(dnel(1,i),min(dnfd(
1,i),min(dnfs(1,i),dnfl(1,i)))))));
       demax(1,i) =
max(dnt(1,i),max(dned(1,i),max(dnes(1,i),max(dnel(1,i),max(dnfd(
1,i),max(dnfs(1,i),dnfl(1,i)))))));

    %plot(L(1,i),di)

       p=p+1
end

figure(1);
plot(L,dnt,'-g',L,dned,'--k',L,dnes,'-K',L,dnel,'--b',L,dnfd,'-
c',L,dnfs,'-y',L,dnfl,'-b',L,dmax,'-m',L,cmax,'-m')
%plot(L,ddl,'-bs'),plot(1,2,1),plot(L,dnt,'-
g^'),plot(1,2,1),plot(L,dnes,'-Kx'),plot(1,2,1),plot(L,dnfl,'-
mh')

axis ([0 50 -69 69])
xlabel('L(m)')
ylabel('deflection (mm)')
title ('Deflection vs Span Length')
```

```
set(gca,'Xtick',0:10:50)
set(gca,'Ytick',-70:10:70)

legend('\deltant: deflection at transfer','\deltaned: Deflection
at errection before super imposed dead
load','\deltanes:deflection at errection after
superimposed','\deltanel:Deflection at erection immediatetly
after service load','\deltanfd:final Deflection before super
imposed dead load','\deltanfs:final Deflection after
superimposed dead load','\deltanfl:final long term deflection
service load','\deltamax:Maximum allowable deflection =
L/640',2)
%str1(1)= {'center each line in the Uicontrol'};
%str1(2)= {'Also check out the text wrap function'};
%str2 = {'Cp =',num2str(c),'Mpa'};
%str3 = {'\deltadl:- deflection due to dead load
only',num2str(c),'Mpa'};
%str4= {'\deltant:- net deflection at
transfer',num2str(c),'Mpa'};
%str5 = {'\deltanes:-deflection at errection with super-imposed
dead load',num2str(c),'Mpa'};
%str6 = {'\deltanfl = final long term deflection at service
load',num2str(c),'Mpa'};
%uicontrol('style','text','position',[80 80 250
65],'string',str1);
%text(150,0
,str2,'fontsize',9,'HorizontalAlignment','right','color','black'
);
 text(55,70,['Cp
=',num2str(c),'Mpa'],'fontsize',9,'HorizontalAlignment','right',
'color','blue')
text(55,60,['Cs
=',num2str(cs),'Mpa'],'fontsize',9,'HorizontalAlignment','right'
,'color','blue')
text(55,50,['fpk
=',num2str(fpk),'Mpa'],'fontsize',9,'HorizontalAlignment','right
','color','blue')
text(55,40,['prestress loss
=',num2str(prl),'%'],'fontsize',9,'HorizontalAlignment','right',
'color','blue')
text(55,30,['Live Loading E
=',num2str(cooperl)],'fontsize',9,'HorizontalAlignment','right',
'color','blue')
```

```
text(55,20,['ee
=',num2str(ee),'mm'],'fontsize',9,'HorizontalAlignment','right',
'color','blue')
text(55,10,['ec
=',num2str(ec),'mm'],'fontsize',9,'HorizontalAlignment','right',
'color','blue')
text(55,0,['girder spacing
=',num2str(ccb),'mm'],'fontsize',9,'HorizontalAlignment','right'
,'color','blue')
text(55,-10,['Slab thikcness
=',num2str(ts),'mm'],'fontsize',9,'HorizontalAlignment','right',
'color','blue')
text(55,-20,['Beam type
=',num2str(beamType)],'fontsize',9,'HorizontalAlignment','right'
,'color','blue')
text(55,-30,['Total Depth
=',num2str(H),'mm'],'fontsize',9,'HorizontalAlignment','right','
color','blue')
text(55,-40,['yb
=',num2str(yb),'mm'],'fontsize',9,'HorizontalAlignment','right',
'color','blue')
text(55,-50,['Pi =',num2str(pff),' *
Pimin'],'fontsize',9,'HorizontalAlignment','right','color','blue
')
 text(-5,70,['b1
=',num2str(b1),'mm'],'fontsize',9,'HorizontalAlignment','right',
'color','blue')
text(-5,60,['b2
=',num2str(b2),'mm'],'fontsize',9,'HorizontalAlignment','right',
'color','blue')
text(-5,50,['b3
=',num2str(b3),'mm'],'fontsize',9,'HorizontalAlignment','right',
'color','blue')
text(-5,40,['b4
=',num2str(b4),'mm'],'fontsize',9,'HorizontalAlignment','right',
'color','blue')
text(-5,30,['b5
=',num2str(b5),'mm'],'fontsize',9,'HorizontalAlignment','right',
'color','blue')
text(-5,20,['b6
=',num2str(b6),'mm'],'fontsize',9,'HorizontalAlignment','right',
'color','blue')
```

```
text(-5,10,['b7
=',num2str(b7),'mm'],'fontsize',9,'HorizontalAlignment','right',
'color','blue')
text(-5,0,['h1
=',num2str(h1),'mm'],'fontsize',9,'HorizontalAlignment','right',
'color','blue')
text(-5,-10,['h2
=',num2str(h2),'mm'],'fontsize',9,'HorizontalAlignment','right',
'color','blue')
text(-5,-20,['h3
=',num2str(h3),'mm'],'fontsize',9,'HorizontalAlignment','right',
'color','blue')
text(-5,-30,['h4
=',num2str(h4),'mm'],'fontsize',9,'HorizontalAlignment','right',
'color','blue')
text(-5,-40,['h5
=',num2str(h5),'mm'],'fontsize',9,'HorizontalAlignment','right',
'color','blue')
text(-5,-50,['h6
=',num2str(h6),'mm'],'fontsize',9,'HorizontalAlignment','right',
'color','blue')
text(-5,-60,['h7
=',num2str(h7),'mm'],'fontsize',9,'HorizontalAlignment','right',
'color','blue')
grid on
hold off

   figure(2);
   subplot(1,2,1),plot(L,demin,'r',L,demax,'b',L,dmax,'-
m',L,cmax,'-m')
%plot(L,ddl,'-bs'),plot(1,2,1),plot(L,dnt,'-
g^'),plot(1,2,1),plot(L,dnes,'-Kx'),plot(1,2,1),plot(L,dnfl,'-
mh')
axis ([0 50 -78 78])
xlabel('L(m)')
ylabel('deflection (mm)')
title ('Deflection Envelope vs Span Length')
set(gca,'Xtick',0:10:50)
set(gca,'Ytick',-70:10:70)

legend('delta emax: maximum camber envelope ','delta emax:
maximum deflection envelope ','\deltamax:Maximum allowable
deflection = L/640');
```

```
grid on


text(58,70,['Cp
=',num2str(c),'Mpa'],'fontsize',9,'HorizontalAlignment','right',
'color','blue')
text(58,60,['Cs
=',num2str(cs),'Mpa'],'fontsize',9,'HorizontalAlignment','right'
,'color','blue')
text(58,50,['fpk
=',num2str(fpk),'Mpa'],'fontsize',9,'HorizontalAlignment','right
','color','blue')
text(58,40,['prestress loss
=',num2str(prl),'%'],'fontsize',9,'HorizontalAlignment','right',
'color','blue')
text(58,30,['Live Loading E
=',num2str(cooperl)],'fontsize',9,'HorizontalAlignment','right',
'color','blue')
text(58,20,['ee
=',num2str(ee),'mm'],'fontsize',9,'HorizontalAlignment','right',
'color','blue')
text(58,10,['ec
=',num2str(ec),'mm'],'fontsize',9,'HorizontalAlignment','right',
'color','blue')
text(58,0,['girder spacing
=',num2str(ccb),'mm'],'fontsize',9,'HorizontalAlignment','right'
,'color','blue')
text(58,-10,['Slab thikcness
=',num2str(ts),'mm'],'fontsize',9,'HorizontalAlignment','right',
'color','blue')
text(58,-20,['Beam type
=',num2str(beamType)],'fontsize',9,'HorizontalAlignment','right'
,'color','blue')
text(58,-30,['Total Depth
=',num2str(H),'mm'],'fontsize',9,'HorizontalAlignment','right','
color','blue')
text(58,-40,['yb
=',num2str(yb),'mm'],'fontsize',9,'HorizontalAlignment','right',
'color','blue')
text(58,-50,['Pi =',num2str(pff),' *
Pimin'],'fontsize',9,'HorizontalAlignment','right','color','blue
')
```

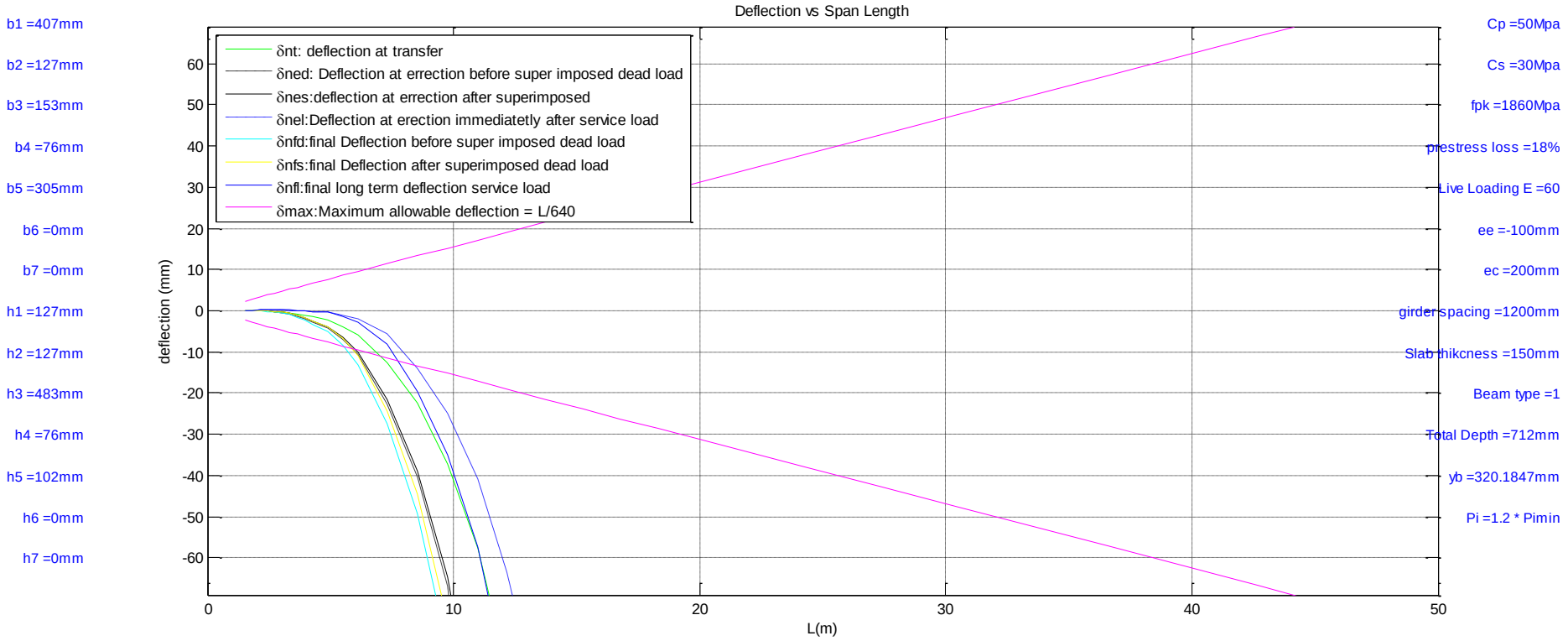
```
 text(-5,70,['b1
=',num2str(b1),'mm'],'fontsize',9,'HorizontalAlignment','right',
'color','blue')
text(-5,60,['b2
=',num2str(b2),'mm'],'fontsize',9,'HorizontalAlignment','right',
'color','blue')
text(-5,50,['b3
=',num2str(b3),'mm'],'fontsize',9,'HorizontalAlignment','right',
'color','blue')
text(-5,40,['b4
=',num2str(b4),'mm'],'fontsize',9,'HorizontalAlignment','right',
'color','blue')
text(-5,30,['b5
=',num2str(b5),'mm'],'fontsize',9,'HorizontalAlignment','right',
'color','blue')
text(-5,20,['b6
=',num2str(b6),'mm'],'fontsize',9,'HorizontalAlignment','right',
'color','blue')
text(-5,10,['b7
=',num2str(b7),'mm'],'fontsize',9,'HorizontalAlignment','right',
'color','blue')
text(-5,0,['h1
=',num2str(h1),'mm'],'fontsize',9,'HorizontalAlignment','right',
'color','blue')
text(-5,-10,['h2
=',num2str(h2),'mm'],'fontsize',9,'HorizontalAlignment','right',
'color','blue')
text(-5,-20,['h3
=',num2str(h3),'mm'],'fontsize',9,'HorizontalAlignment','right',
'color','blue')
text(-5,-30,['h4
=',num2str(h4),'mm'],'fontsize',9,'HorizontalAlignment','right',
'color','blue')
text(-5,-40,['h5
=',num2str(h5),'mm'],'fontsize',9,'HorizontalAlignment','right',
'color','blue')
text(-5,-50,['h6
=',num2str(h6),'mm'],'fontsize',9,'HorizontalAlignment','right',
'color','blue')
text(-5,-60,['h7
=',num2str(h7),'mm'],'fontsize',9,'HorizontalAlignment','right',
'color','blue')
```

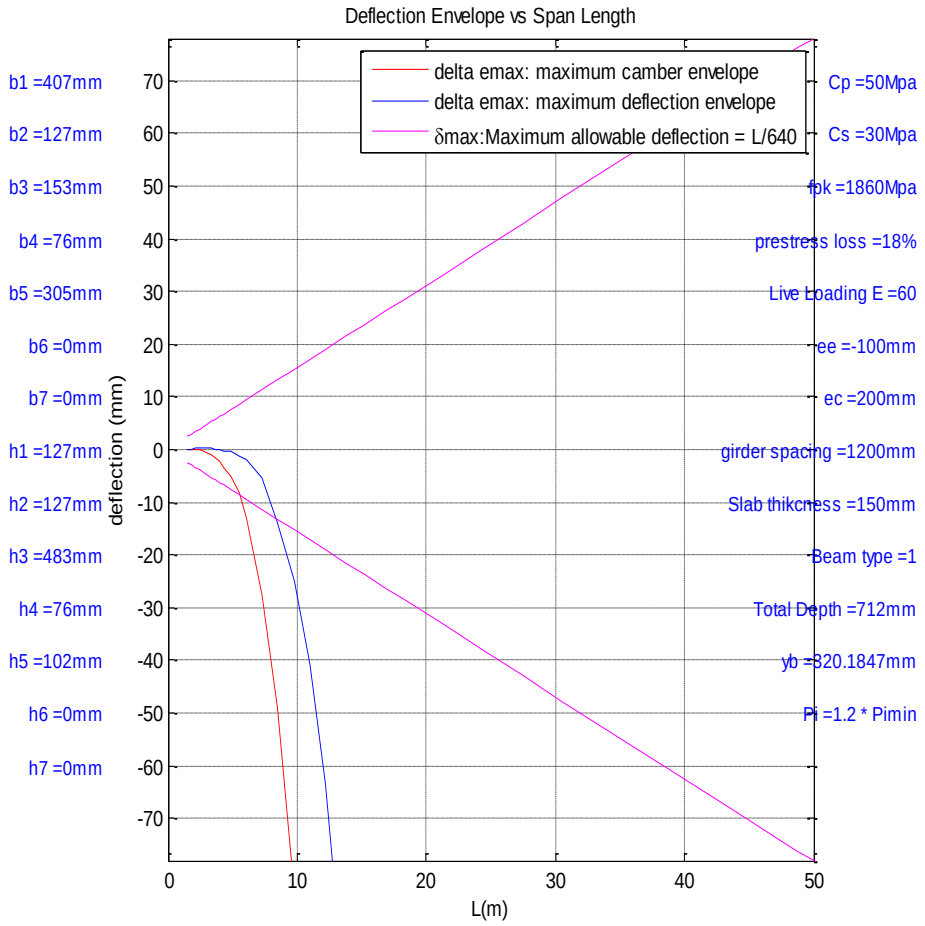
```
    %plotyy(L,emaxc,L,Pmin,'-rp','bs')
 subplot(1,2,2),plot(L, emaxs, '--g',L, emins, '--b',L, emaxm,
'-c',L, eminm,'-.m',L,emaxc,'--k')
 axis ([0 50 enaxis epaxis])
 ax1 = gca;
 set(ax1,'XColor','k','YColor','K')
    xlabel('L(m)')
    ylabel('eccentricity (mm)')
    %y=ylabel('eccentricity (mm)')
    %set(y,'position',get(y,'position')-[-2,0,0]);

    title('Alloawable eccentricity and Prestressing Force')
    set(gca,'Xtick',0:10:50)
    set(gca,'Ytick',enaxis:200:epaxis)
    legend('emax support','emin support','emax mid span','emin
mid span','emax for Pmin',2)
    %text(100,950,'s: for
suport','fontsize',9,'HorizontalAlignment','right','color','red'
)
    %text(100,850,'m: for mid-
span','fontsize',9,'HorizontalAlignment','right','color','red')
    %text(100,750,'c: coresponding to
pmin','fontsize',9,'HorizontalAlignment','right','color','red')
    grid on

ax2 = axes('Position',get(ax1,'Position'),...
    'XAxisLocation','top',...
    'YAxisLocation','right',...
    'Color','none',...
    'XColor','r','YColor','r');
    imax = find(max(Paxis)==Paxis);
    axis ([0 50 0 Paxis(imax)])
    set(gca,'Xtick',0:50:50)
    set(gca,'Ytick',0:10:Pmin(imax))
    ylabel('Prestressing force (MN)')
hl2 = line(L,Pmin,'Color','r','Parent',ax2);
hl3 = line(L,Pmax,'Color','black','Parent',ax2);
legend('Pmin','Pmax',1)

 hold off
```

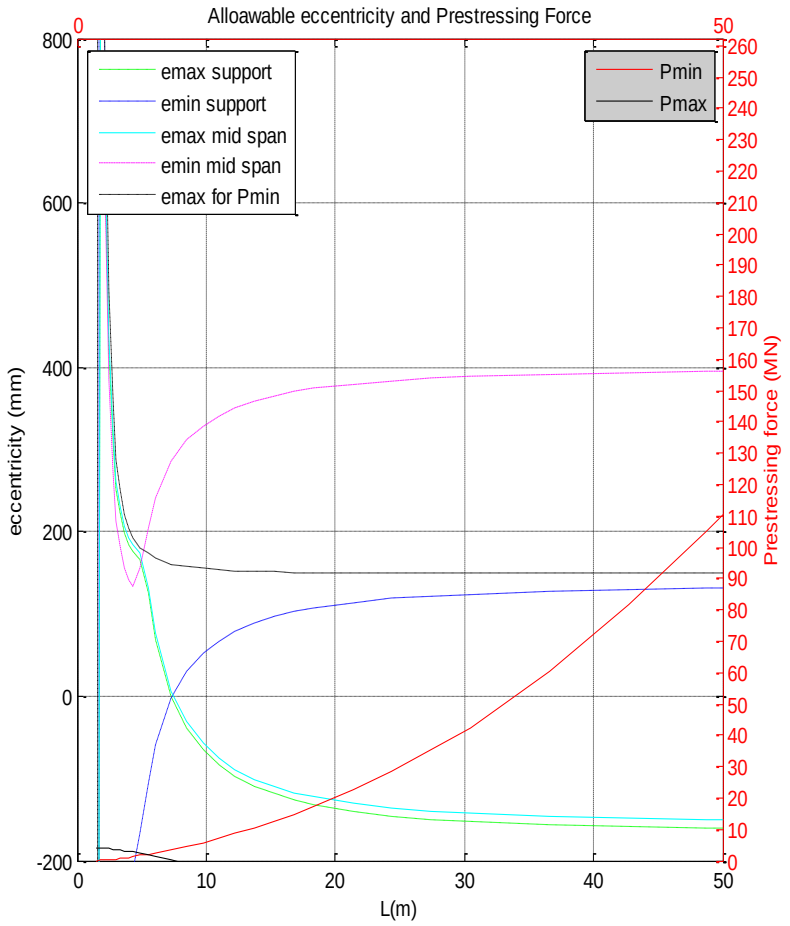
Appendix B: Sample Charts plotted using the program

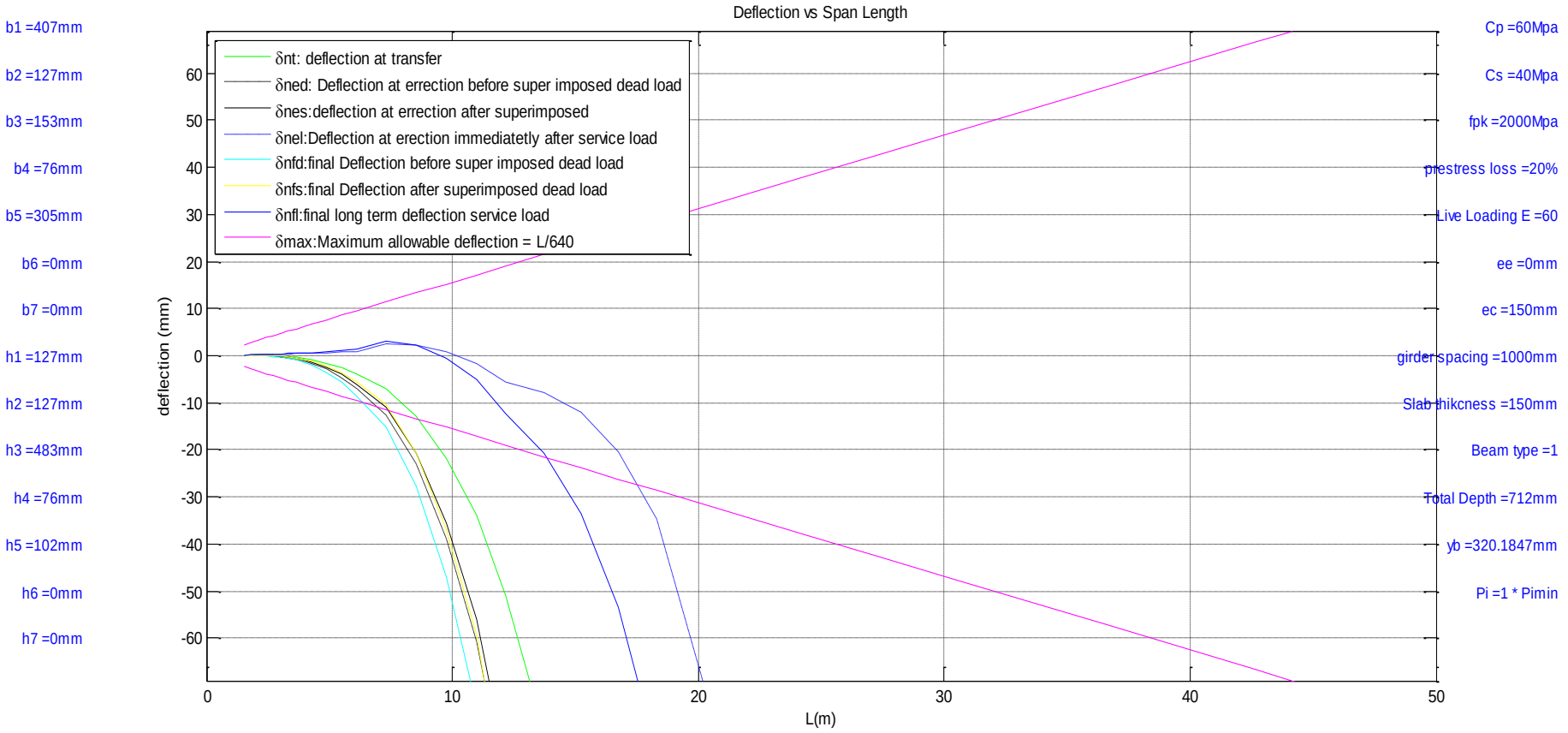
Note: - the deflection data after the break (after the minimum allowable eccentricity become higher than that of the maximum) is just numerical has no engineering value.

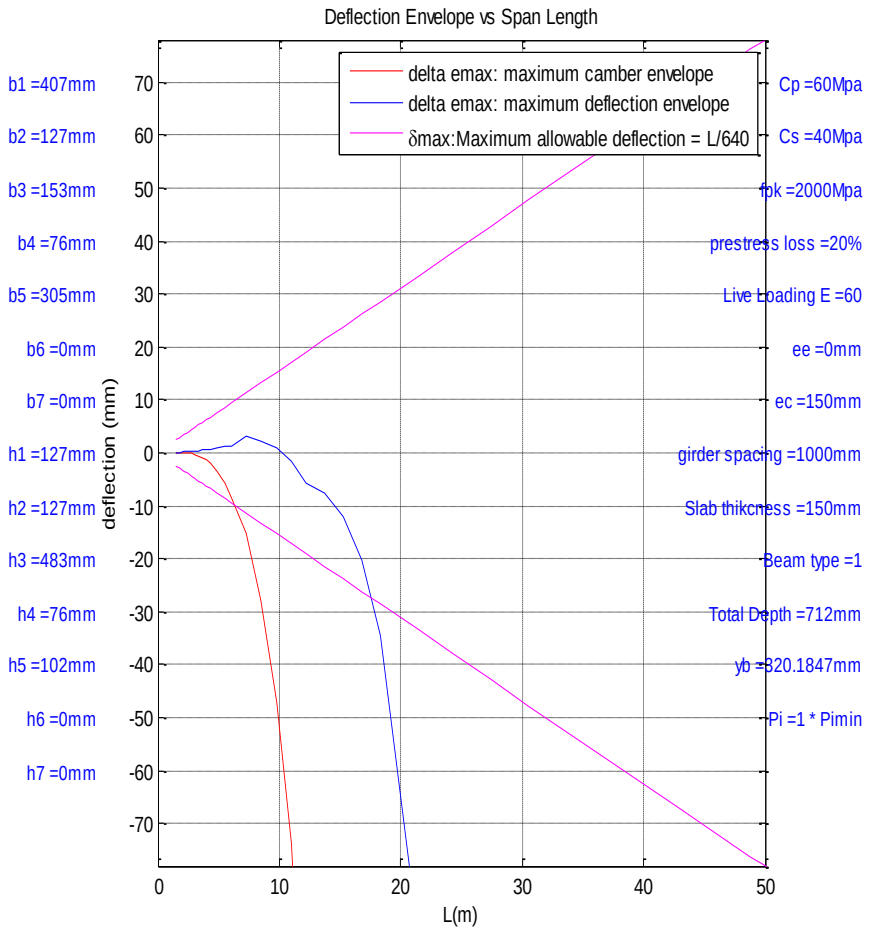
AASHTO/PCI I beam Type I

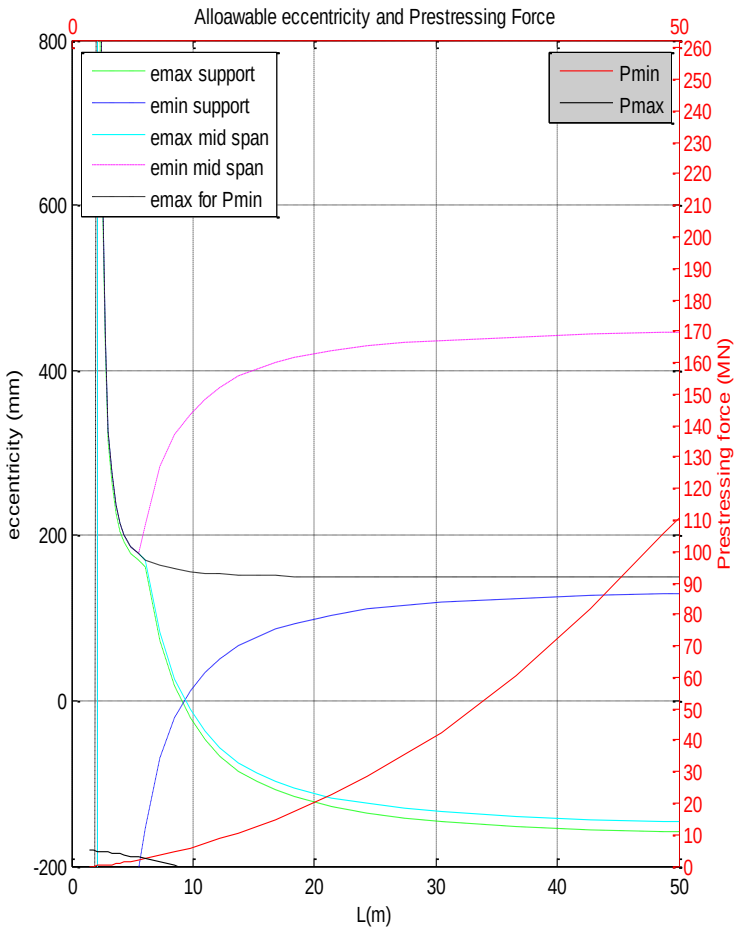


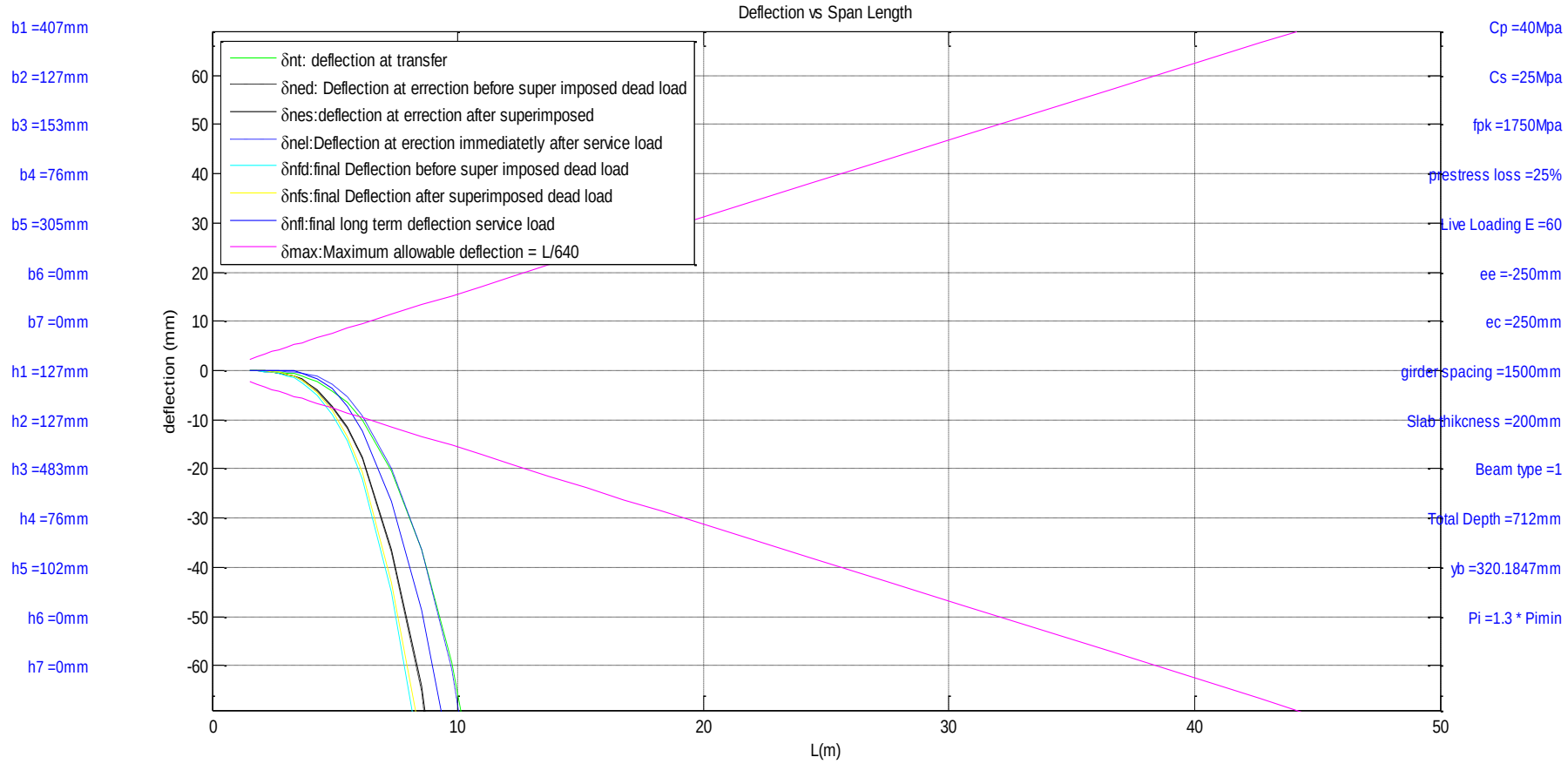

Deflection Envelope vs Span Length
delta emax: maximum camber envelope
delta emax: maximum deflection envelope
δmax:Maximum allowable deflection = L/640
b1 =407mm
b2 =127mm
b3 =153mm
b4 =76mm
b5 =305mm
b6 =0mm
b7 =0mm
h1 =127mm
h2 =127mm
h3 =483mm
h4 =76mm
h5 =102mm
h6 =0mm
h7 =0mm
Cp =50Mpa
Cs =30Mpa
fpk =1860Mpa
prestress loss =18%
Live Loading E =60
ee =-100mm
ec =200mm
girder spacing =1200mm
Slab thikcness =150mm
Beam type =1
Total Depth =712mm
yb =320.1847mm
Pi =1.2 * Pimin
deflection (mm)
L(m)

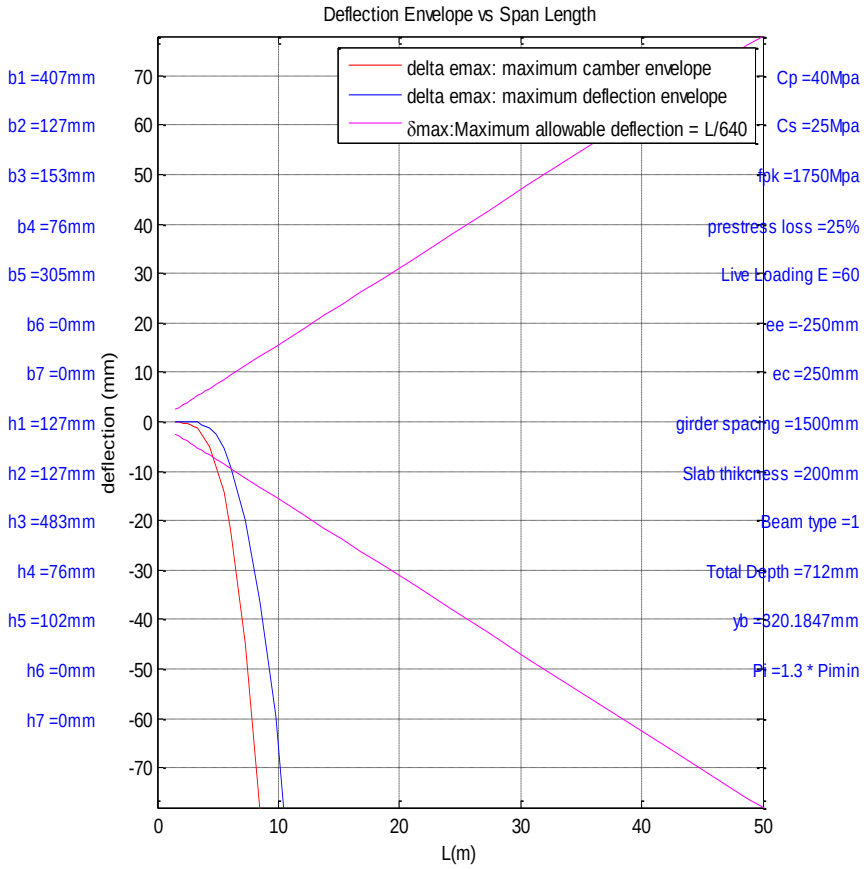

Alloawable eccentricity and Prestressing Force
emax support
emin support
emax mid span
emin mid span
emax for Pmin
Pmin
Pmax
eccentricity (mm)
Prestressing force (MN)
L(m)

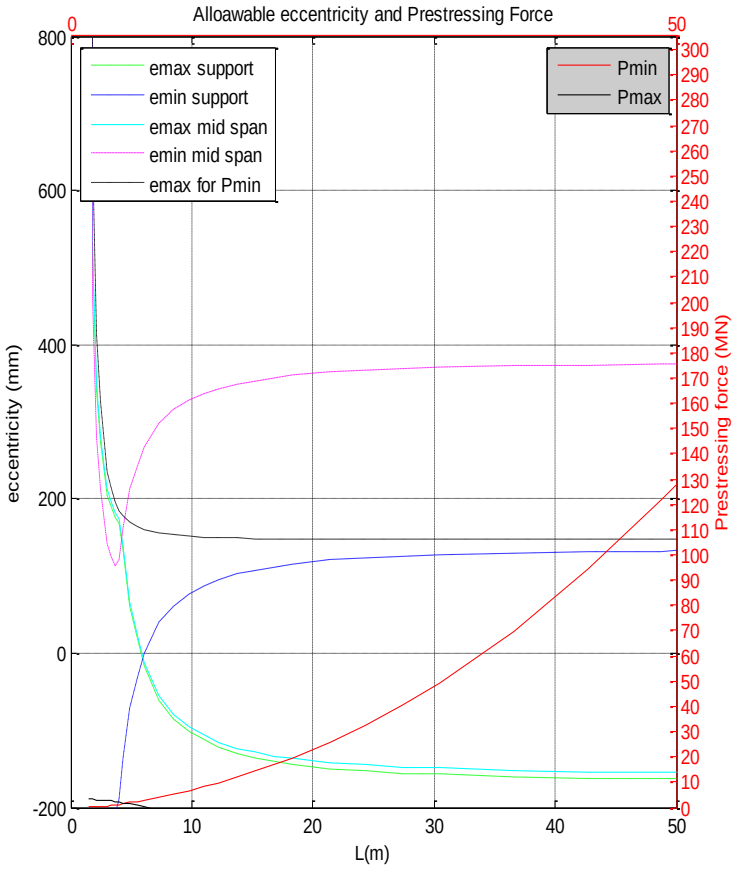

Deflection vs Span Length
δnt: deflection at transfer
δned: Deflection at errection before super imposed dead load
δnes:deflection at errection after superimposed
δnel:Deflection at erection immediatetly after service load
δnfd:final Deflection before super imposed dead load
δnfs:final Deflection after superimposed dead load
δnfl:final long term deflection service load
δmax:Maximum allowable deflection = L/640
deflection (mm)
L(m)
b1 =407mm
b2 =127mm
b3 =153mm
b4 =76mm
b5 =305mm
b6 =0mm
b7 =0mm
h1 =127mm
h2 =127mm
h3 =483mm
h4 =76mm
h5 =102mm
h6 =0mm
h7 =0mm
Cp =60Mpa
Cs =40Mpa
fpk =2000Mpa
prestress loss =20%
Live Loading E =60
ee =0mm
ec =150mm
girder spacing =1000mm
Slab thikcness =150mm
Beam type =1
Total Depth =712mm
yb =320.1847mm
Pi =1 * Pimin


Deflection Envelope vs Span Length
delta emax: maximum camber envelope
delta emax: maximum deflection envelope
δmax:Maximum allowable deflection = L/640
b1 =407mm
b2 =127mm
b3 =153mm
b4 =76mm
b5 =305mm
b6 =0mm
b7 =0mm
h1 =127mm
h2 =127mm
h3 =483mm
h4 =76mm
h5 =102mm
h6 =0mm
h7 =0mm
Cp =60Mpa
Cs =40Mpa
fpk =2000Mpa
prestress loss =20%
Live Loading E =60
ee =0mm
ec =150mm
girder spacing =1000mm
Slab thikcness =150mm
Beam type =1
Total Depth =712mm
yb =320.1847mm
Pi =1 * Pimin
deflection (mm)
L(m)

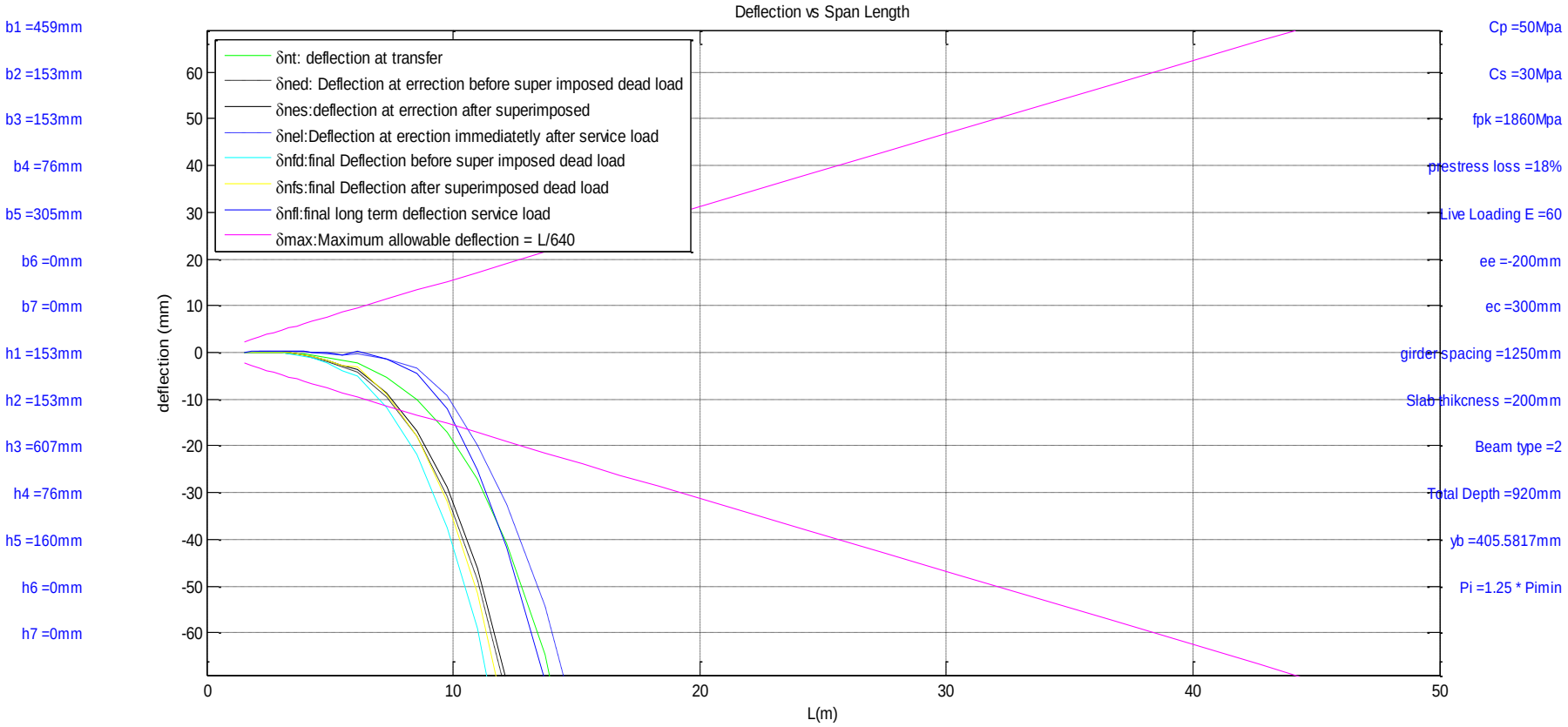

Alloawable eccentricity and Prestressing Force
emax support
emin support
emax mid span
emin mid span
emax for Pmin
Pmin
Pmax
eccentricity (mm)
Prestressing force (MN)
L(m)

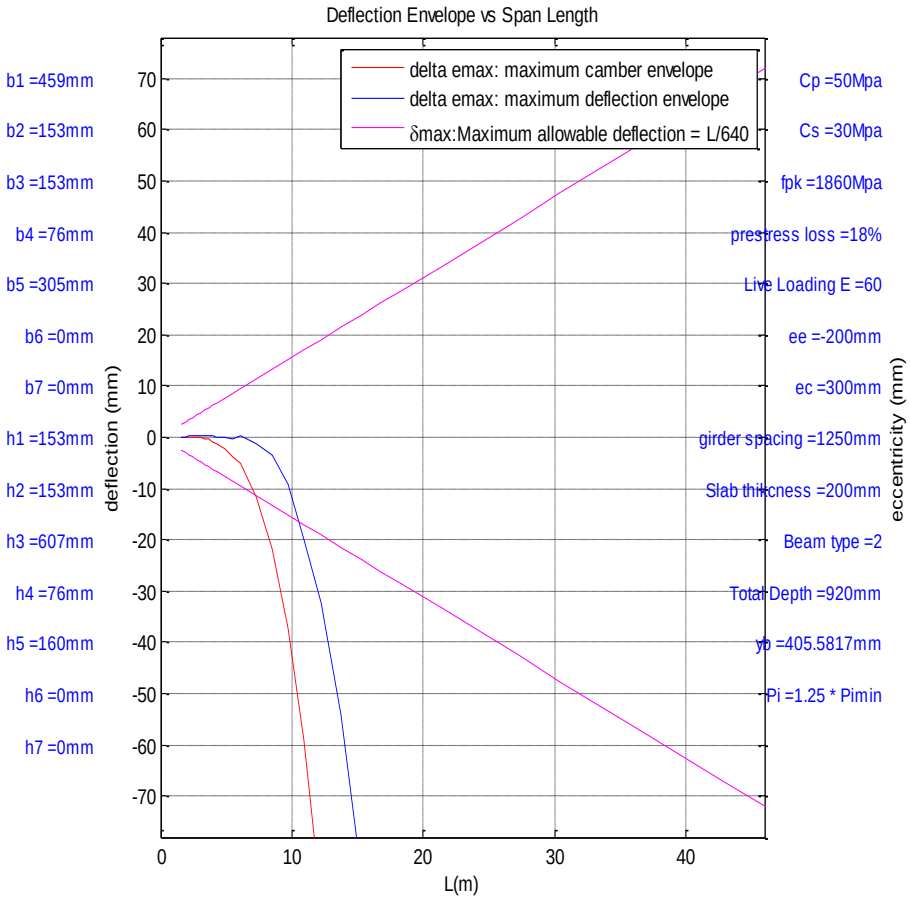

Deflection vs Span Length
b1 =407mm
b2 =127mm
b3 =153mm
b4 =76mm
b5 =305mm
b6 =0mm
b7 =0mm
h1 =127mm
h2 =127mm
h3 =483mm
h4 =76mm
h5 =102mm
h6 =0mm
h7 =0mm
δnt: deflection at transfer
δned: Deflection at errection before super imposed dead load
δnes:deflection at errection after superimposed
δnel:Deflection at erection immediatetly after service load
δnfd:final Deflection before super imposed dead load
δnfs:final Deflection after superimposed dead load
δnfl:final long term deflection service load
δmax:Maximum allowable deflection = L/640
deflection (mm)
L(m)
Cp =40Mpa
Cs =25Mpa
fpk =1750Mpa
prestress loss =25%
Live Loading E =60
ee =-250mm
ec =250mm
girder spacing =1500mm
Slab thikcness =200mm
Beam type =1
Total Depth =712mm
yb =320.1847mm
Pi =1.3 * Pimin

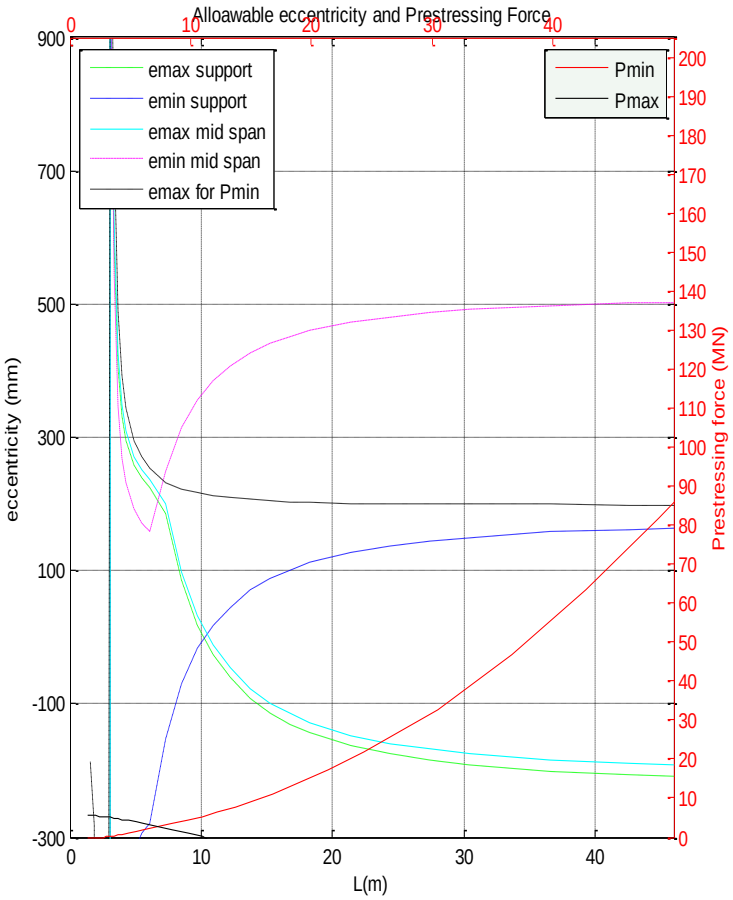

Deflection Envelope vs Span Length
delta emax: maximum camber envelope
delta emax: maximum deflection envelope
δmax:Maximum allowable deflection = L/640
b1 =407mm
b2 =127mm
b3 =153mm
b4 =76mm
b5 =305mm
b6 =0mm
b7 =0mm
h1 =127mm
h2 =127mm
h3 =483mm
h4 =76mm
h5 =102mm
h6 =0mm
h7 =0mm
Cp =40Mpa
Cs =25Mpa
fpk =1750Mpa
prestress loss =25%
Live Loading E =60
ee =-250mm
ec =250mm
girder spacing =1500mm
Slab thikcness =200mm
Beam type =1
Total Depth =712mm
yb =820.1847mm
Pi =1.3 * Pimin
deflection (mm)
L(m)

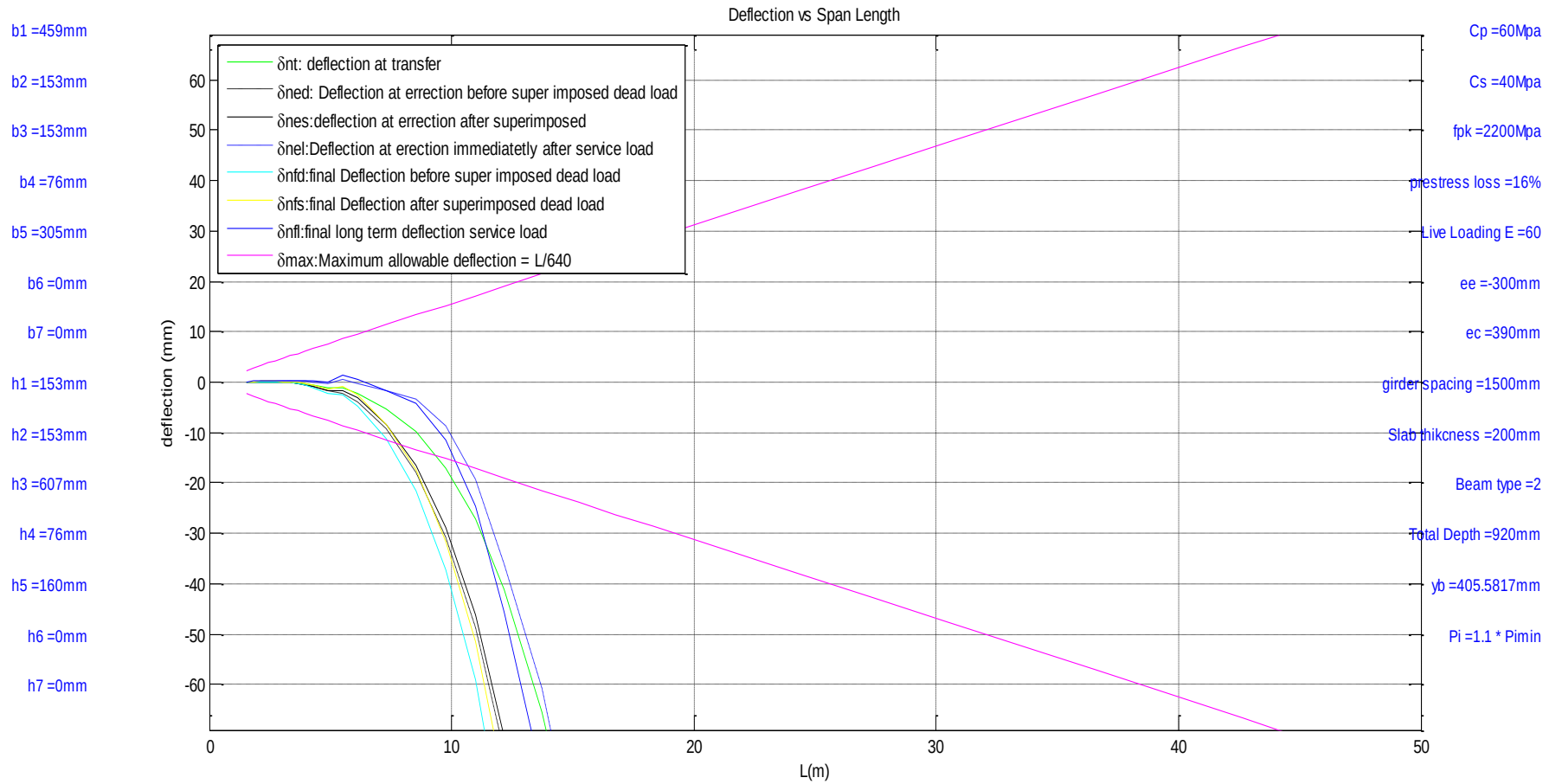

Alloawable eccentricity and Prestressing Force
emax support
emin support
emax mid span
emin mid span
emax for Pmin
Pmin
Pmax
eccentricity (mm)
Prestressing force (MN)
L(m)

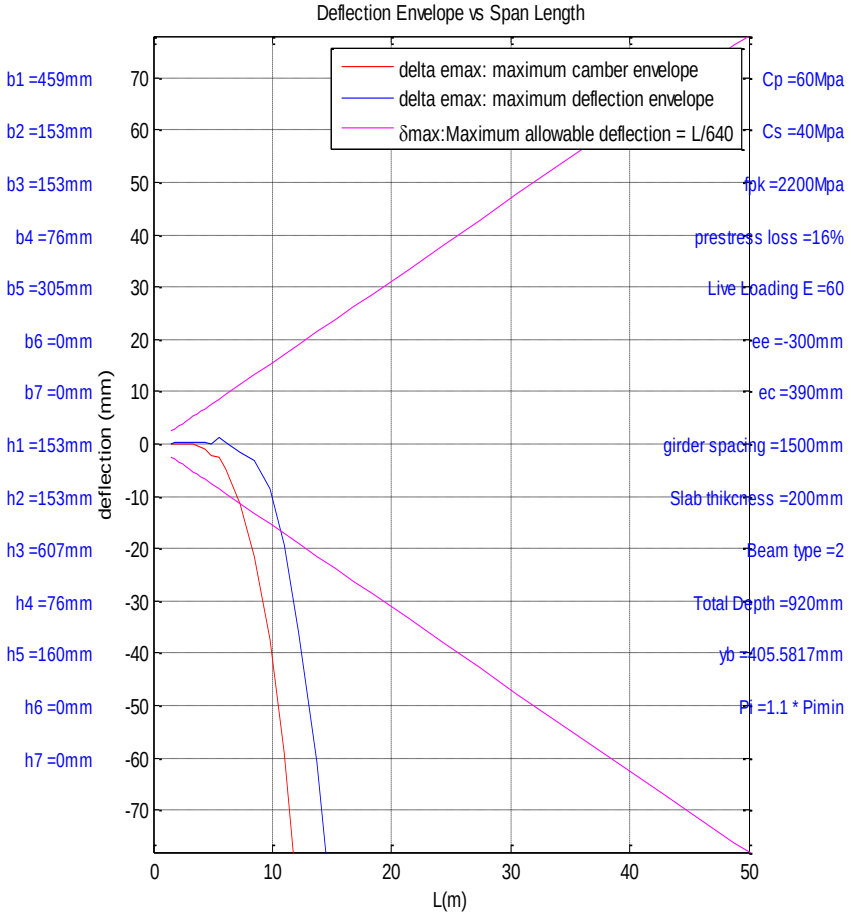
AASHTO/PCI I beam Type II

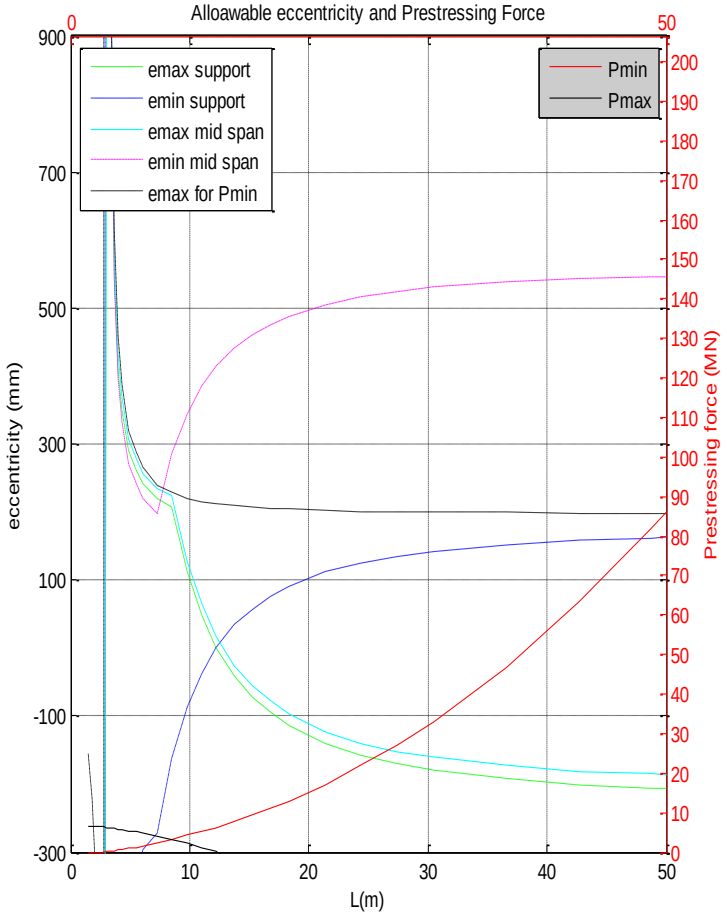


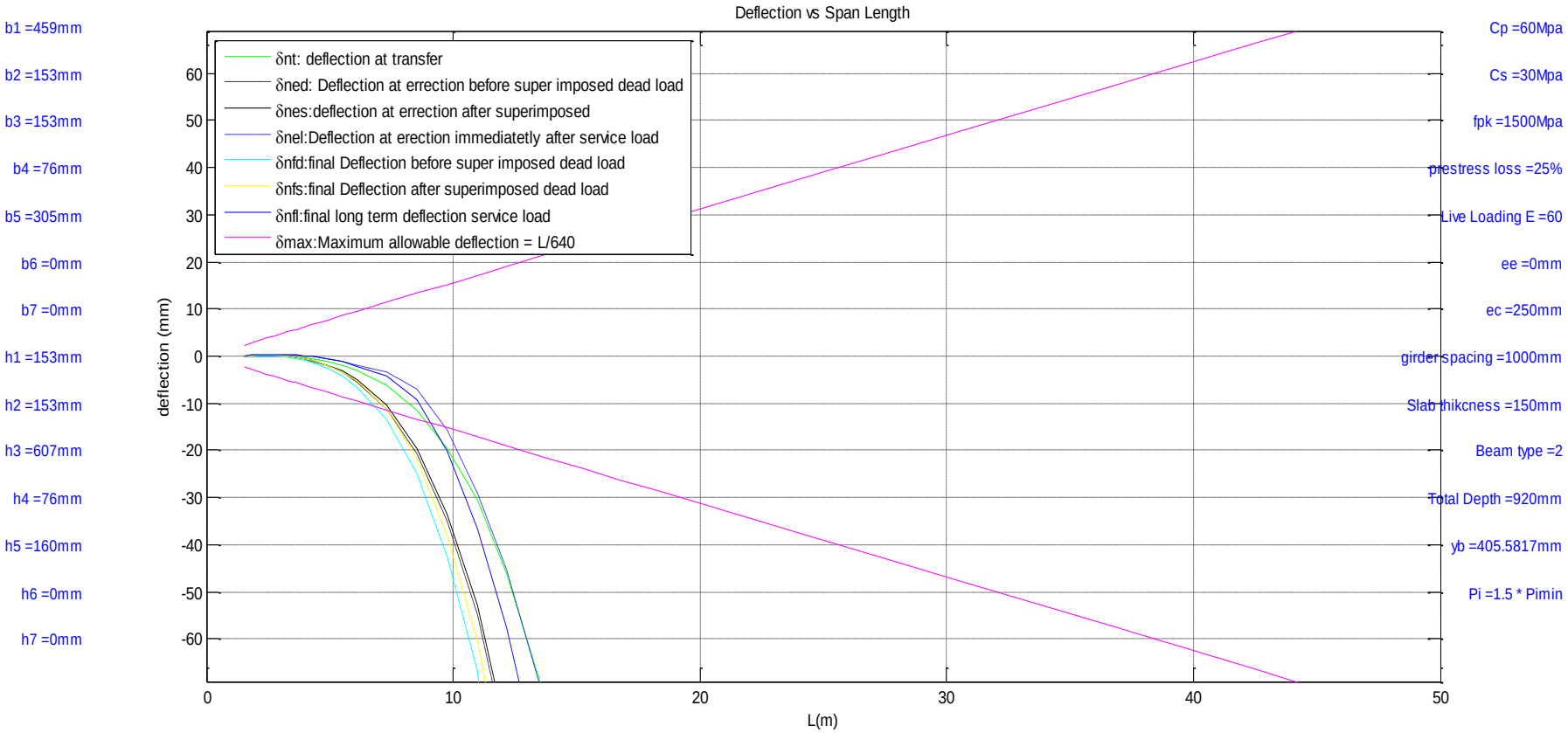

Deflection Envelope vs Span Length
delta emax: maximum camber envelope
delta emax: maximum deflection envelope
δmax:Maximum allowable deflection = L/640
b1 =459mm
b2 =153mm
b3 =153mm
b4 =76mm
b5 =305mm
b6 =0mm
b7 =0mm
h1 =153mm
h2 =153mm
h3 =607mm
h4 =76mm
h5 =160mm
h6 =0mm
h7 =0mm
Cp =50Mpa
Cs =30Mpa
fpk =1860Mpa
prestress loss =18%
Live Loading E =60
ee =-200mm
ec =300mm
girder spacing =1250mm
Slab thickness =200mm
Beta type =2
Total Depth =920mm
yb =405.5817mm
Pi =1.25 * Pimin
deflection (mm)
L(m)

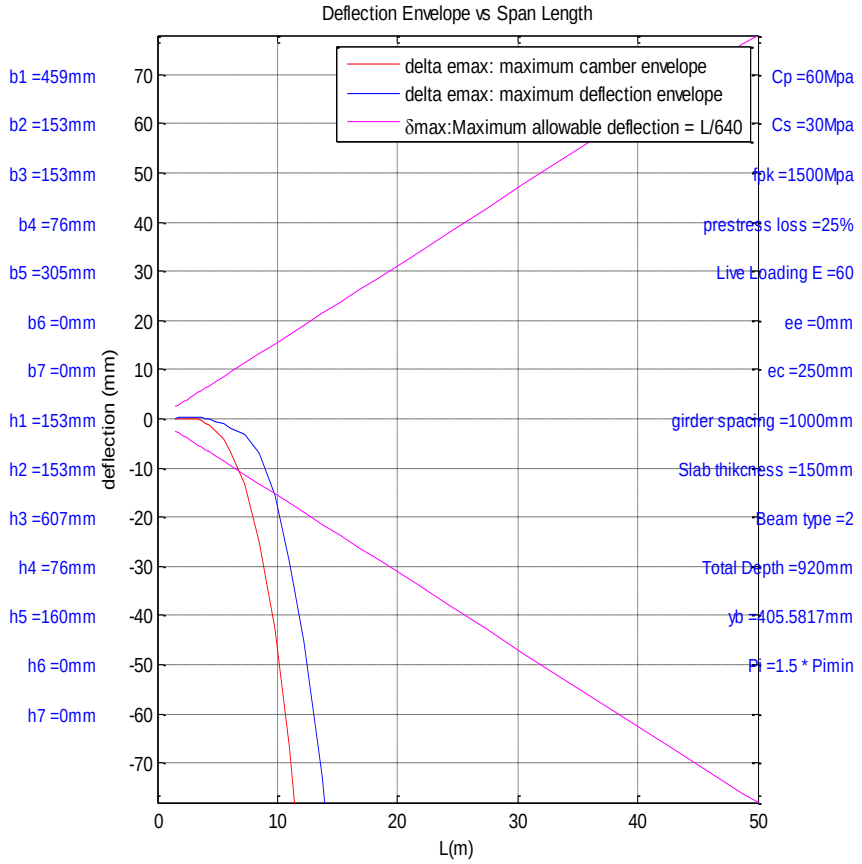

Alloawable eccentricity and Prestressing Force
emax support
emin support
emax mid span
emin mid span
emax for Pmin
Pmin
Pmax
eccentricity (mm)
Prestressing force (MN)
L(m)

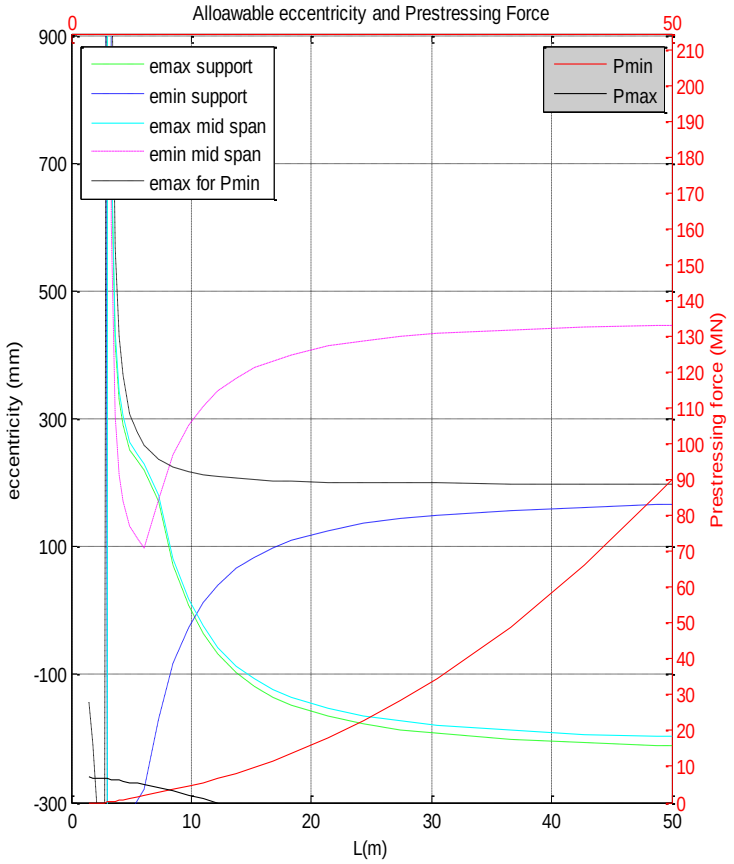

Deflection vs Span Length
b1 =459mm
b2 =153mm
b3 =153mm
b4 =76mm
b5 =305mm
b6 =0mm
b7 =0mm
h1 =153mm
h2 =153mm
h3 =607mm
h4 =76mm
h5 =160mm
h6 =0mm
h7 =0mm
δnt: deflection at transfer
δned: Deflection at errection before super imposed dead load
δnes:deflection at errection after superimposed
δnel:Deflection at erection immediatetly after service load
δnfd:final Deflection before super imposed dead load
δnfs:final Deflection after superimposed dead load
δnfl:final long term deflection service load
δmax:Maximum allowable deflection = L/640
deflection (mm)
L(m)
Cp =60Mpa
Cs =40Mpa
fpk =2200Mpa
prestress loss =16%
Live Loading E =60
ee =-300mm
ec =390mm
girder spacing =1500mm
Slab thikcness =200mm
Beam type =2
Total Depth =920mm
yb =405.5817mm
Pi =1.1 * Pimin


Deflection Envelope vs Span Length
delta emax: maximum camber envelope
delta emax: maximum deflection envelope
δmax:Maximum allowable deflection = L/640
b1 =459mm
b2 =153mm
b3 =153mm
b4 =76mm
b5 =305mm
b6 =0mm
b7 =0mm
h1 =153mm
h2 =153mm
h3 =607mm
h4 =76mm
h5 =160mm
h6 =0mm
h7 =0mm
Cp =60Mpa
Cs =40Mpa
fpk =2200Mpa
prestress loss =16%
Live Loading E =60
ee =-300mm
ec =390mm
girder spacing =1500mm
Slab thikcness =200mm
Beam type =2
Total Depth =920mm
yb =405.5817mm
Pi =1.1 * Pimin
deflection (mm)
L(m)

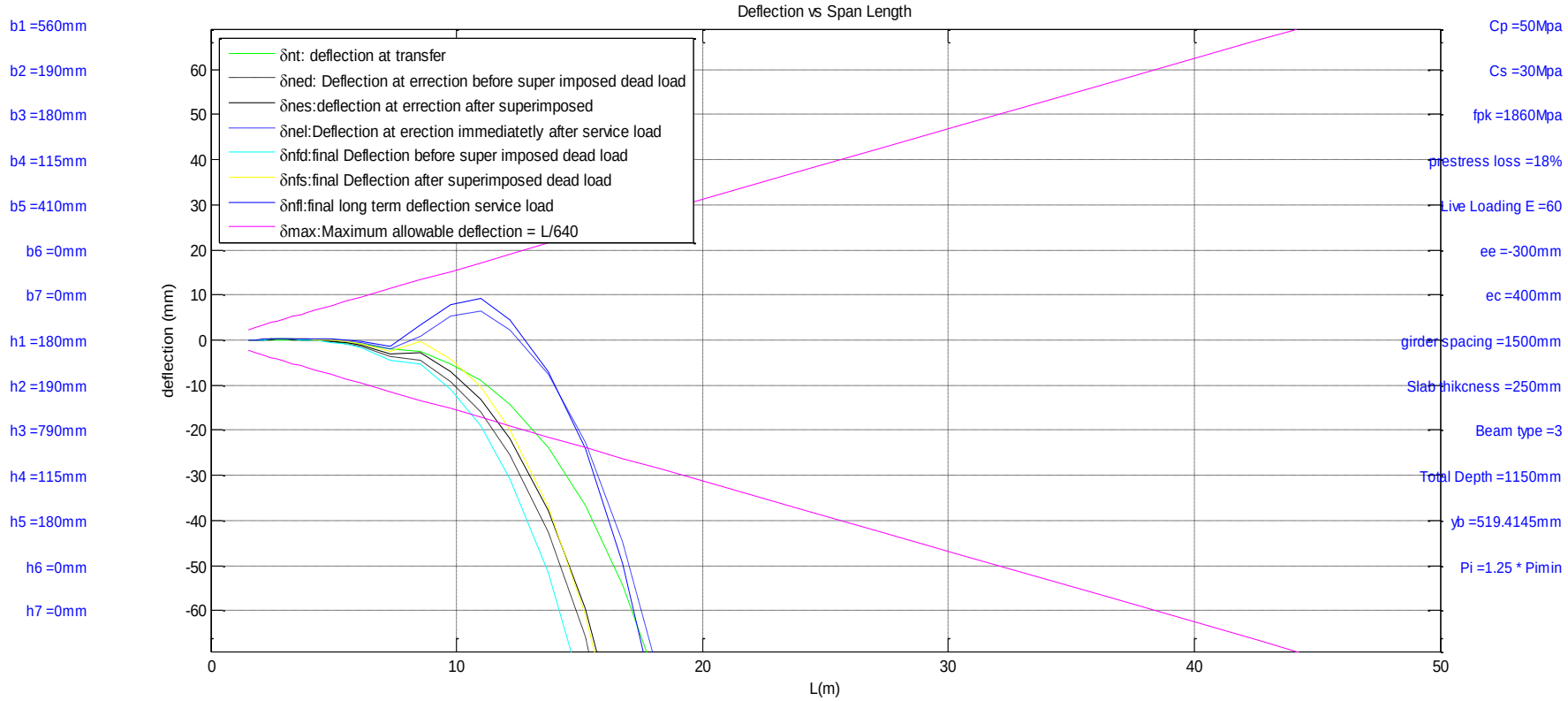

Alloawable eccentricity and Prestressing Force
emax support
emin support
emax mid span
emin mid span
emax for Pmin
Pmin
Pmax
eccentricity (mm)
Prestressing force (MN)
L(m)

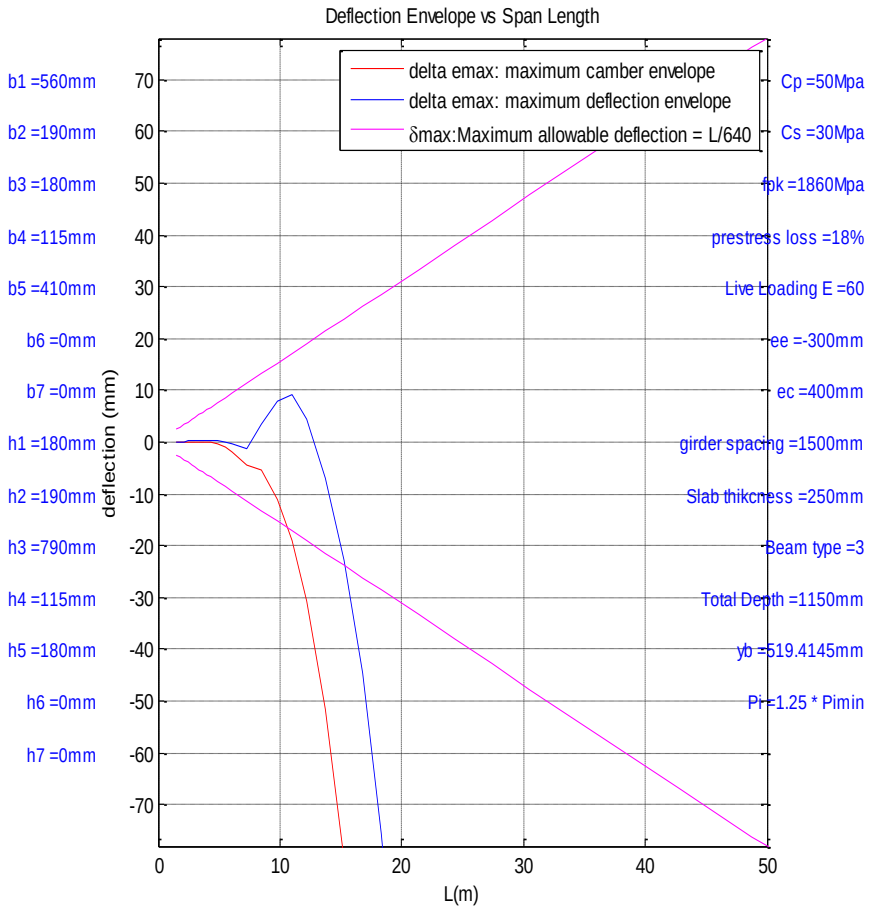

Deflection vs Span Length
δnt: deflection at transfer
δned: Deflection at erection before super imposed dead load
δnes:deflection at erection after superimposed
δnel:Deflection at erection immediatetly after service load
δnfd:final Deflection before super imposed dead load
δnfs:final Deflection after superimposed dead load
δnfl:final long term deflection service load
δmax:Maximum allowable deflection = L/640
deflection (mm)
L(m)
b1 =459mm
b2 =153mm
b3 =153mm
b4 =76mm
b5 =305mm
b6 =0mm
b7 =0mm
h1 =153mm
h2 =153mm
h3 =607mm
h4 =76mm
h5 =160mm
h6 =0mm
h7 =0mm
Cp =60Mpa
Cs =30Mpa
fpk =1500Mpa
prestress loss =25%
Live Loading E =60
ee =0mm
ec =250mm
girder spacing =1000mm
Slab thikcness =150mm
Beam type =2
Total Depth =920mm
yb =405.5817mm
Pi =1.5 * Pimin

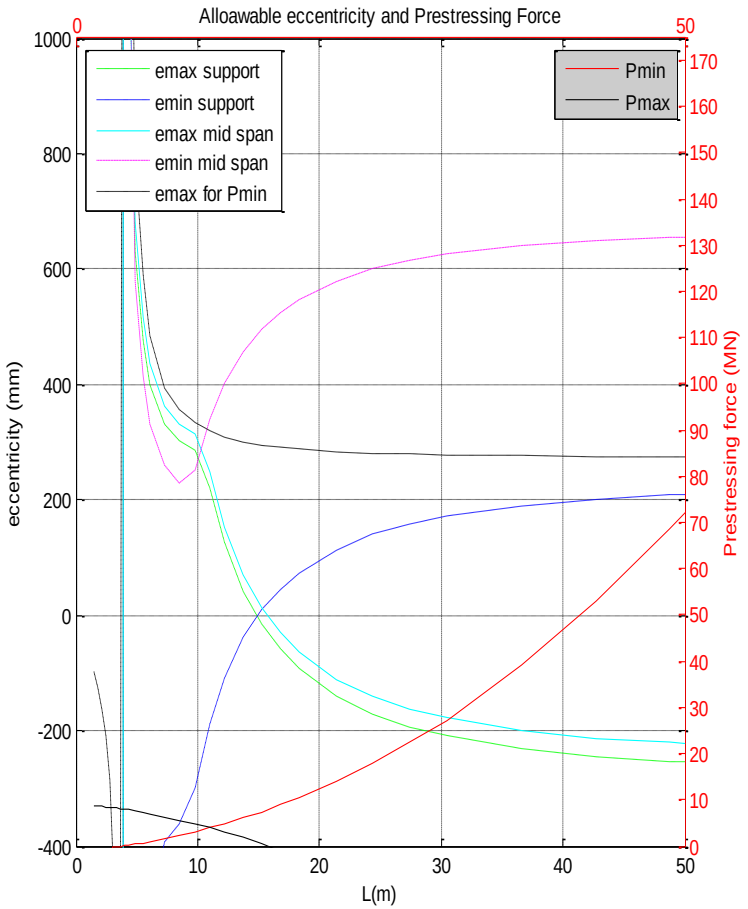

Deflection Envelope vs Span Length
delta emax: maximum camber envelope
delta emax: maximum deflection envelope
δmax:Maximum allowable deflection = L/640
b1 =459mm
b2 =153mm
b3 =153mm
b4 =76mm
b5 =305mm
b6 =0mm
b7 =0mm
h1 =153mm
h2 =153mm
h3 =607mm
h4 =76mm
h5 =160mm
h6 =0mm
h7 =0mm
Cp =60Mpa
Cs =30Mpa
fpk =1500Mpa
prestress loss =25%
Live Loading E =60
ee =0mm
ec =250mm
girder spacing =1000mm
Slab thikcness =150mm
Beam type =2
Total Depth =920mm
yb =405.5817mm
Pi =1.5 * Pimin
deflection (mm)
L(m)

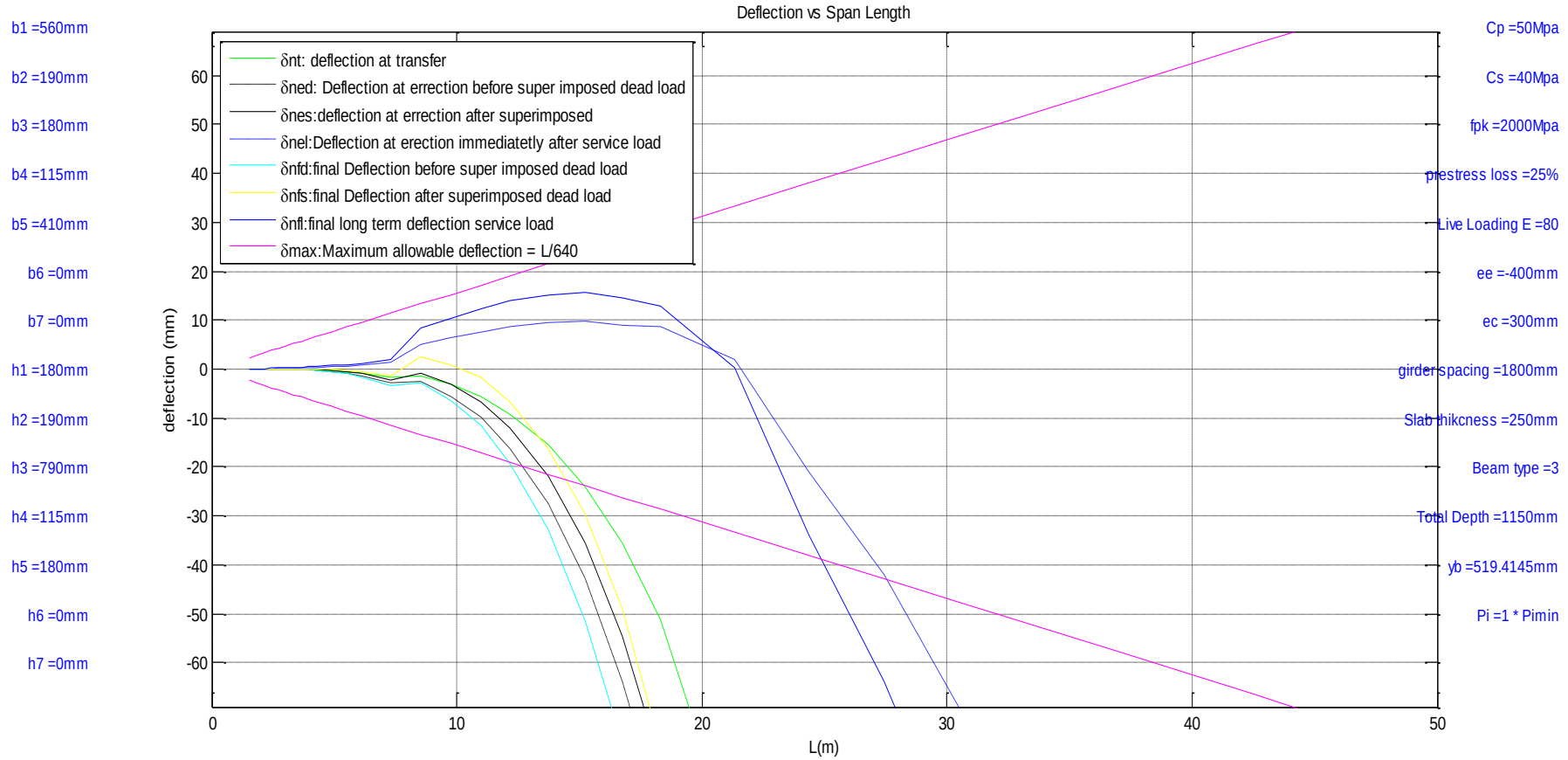

Alloawable eccentricity and Prestressing Force
emax support
emin support
emax mid span
emin mid span
emax for Pmin
Pmin
Pmax
eccentricity (mm)
Prestressing force (MN)
L(m)

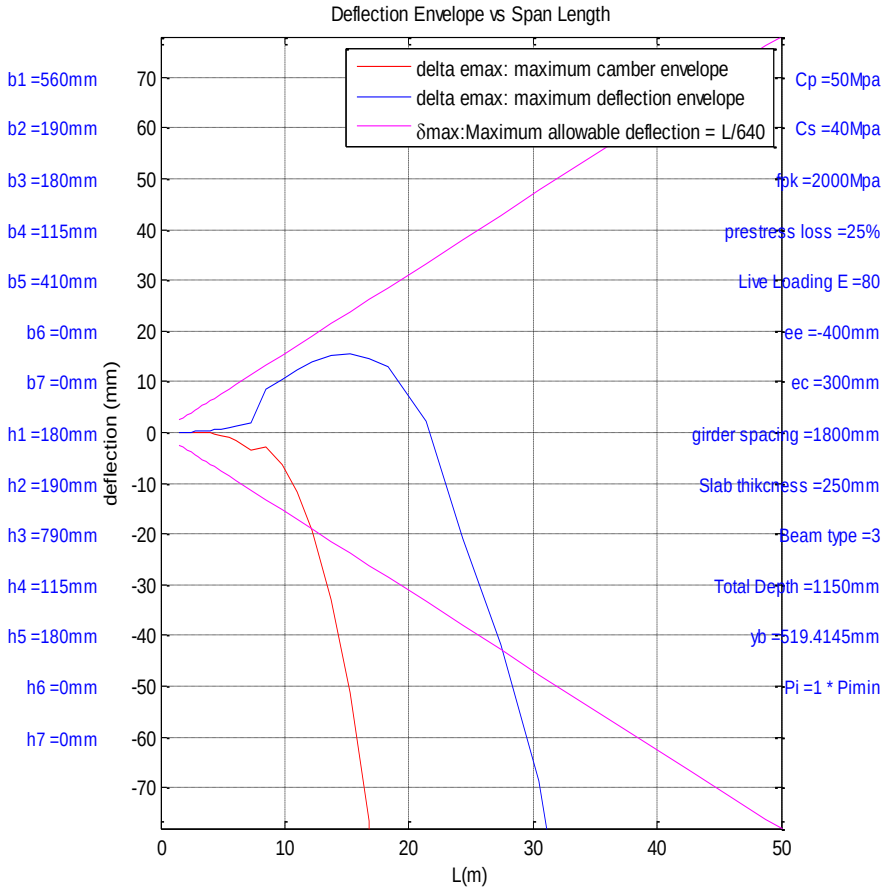
AASHTO/PCI I beam Type III

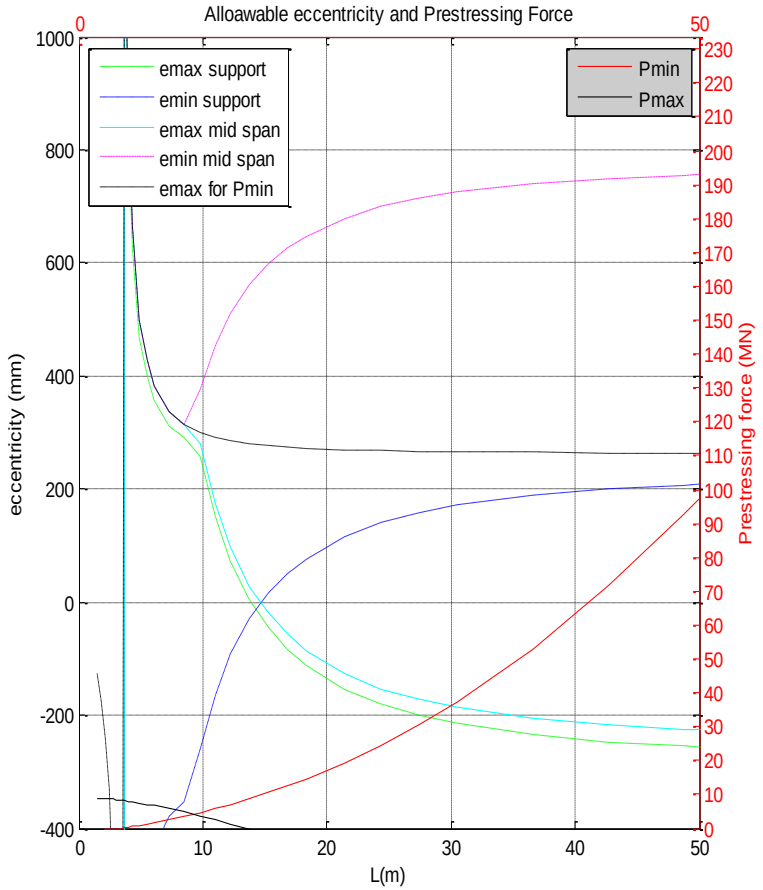


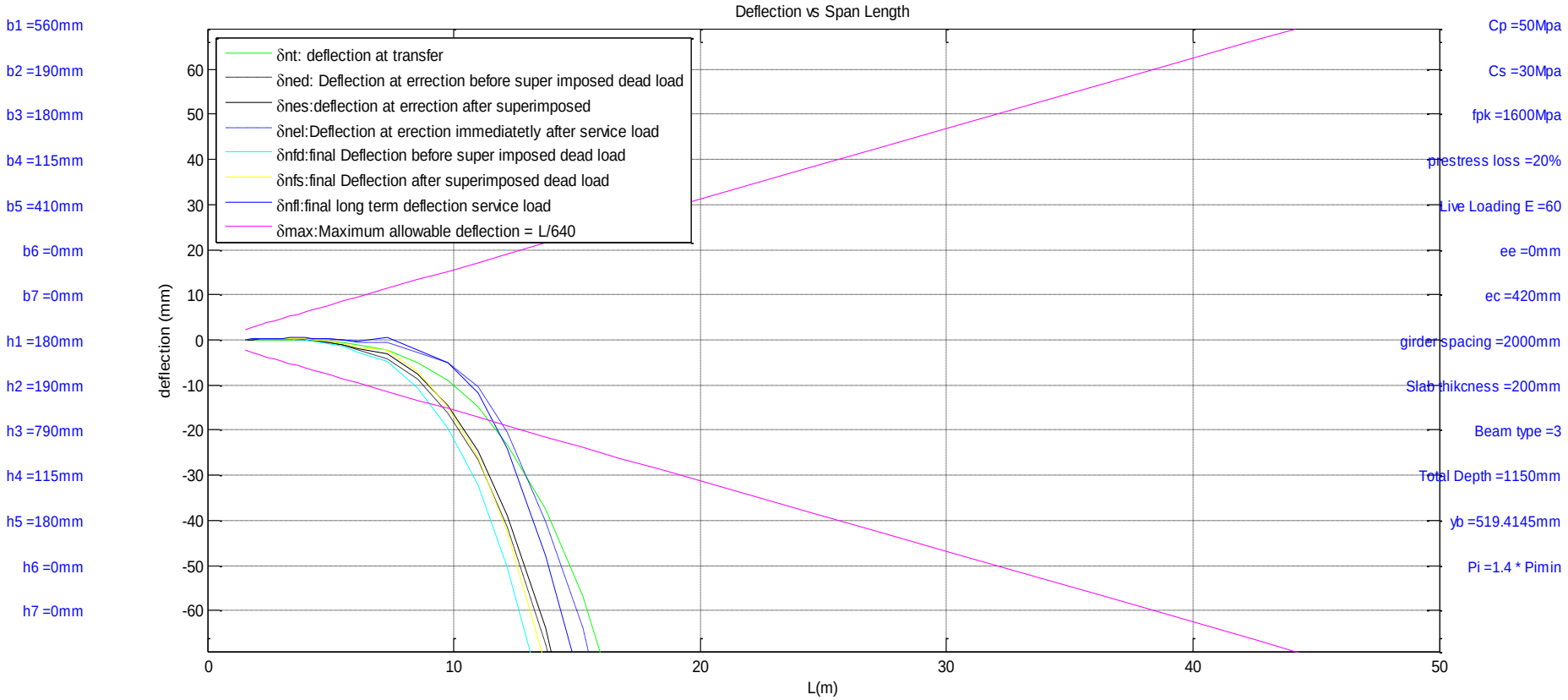

Deflection Envelope vs Span Length
delta emax: maximum camber envelope
delta emax: maximum deflection envelope
δmax:Maximum allowable deflection = L/640
b1 =560mm
b2 =190mm
b3 =180mm
b4 =115mm
b5 =410mm
b6 =0mm
b7 =0mm
h1 =180mm
h2 =190mm
h3 =790mm
h4 =115mm
h5 =180mm
h6 =0mm
h7 =0mm
Cp =50Mpa
Cs =30Mpa
fpk =1860Mpa
prestress loss =18%
Live Loading E =60
ee =-300mm
ec =400mm
girder spacing =1500mm
Slab thikcness =250mm
Beam type =3
Total Depth =1150mm
yb =519.4145mm
Pi=1.25 * Pimin
deflection (mm)
L(m)

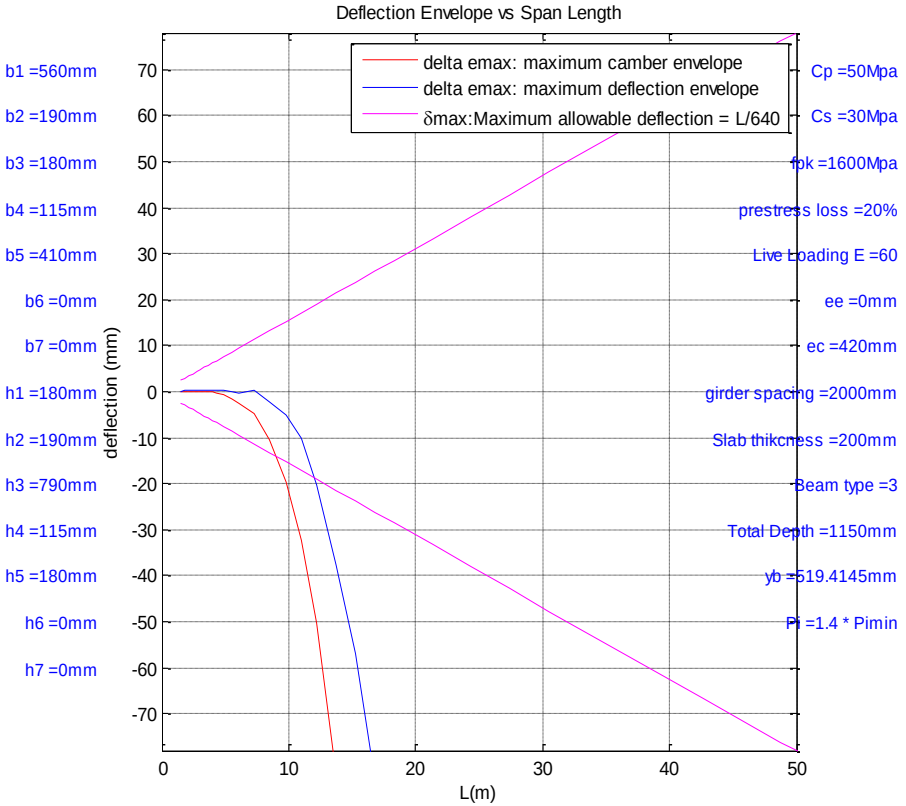

Alloawable eccentricity and Prestressing Force
emax support
emin support
emax mid span
emin mid span
emax for Pmin
Pmin
Pmax
eccentricity (mm)
Prestressing force (MN)
L(m)

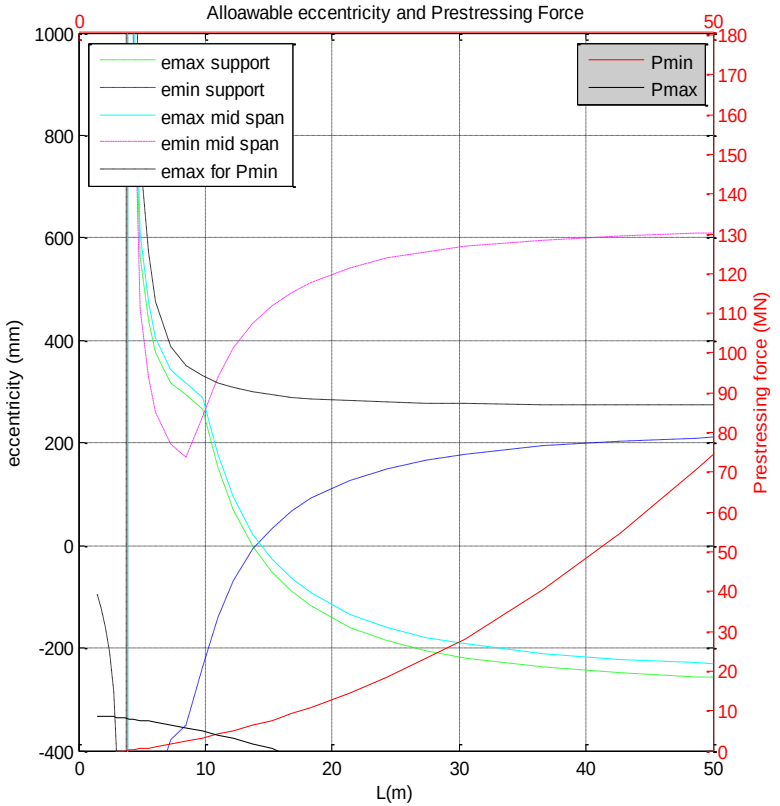

Deflection vs Span Length
b1 =560mm
b2 =190mm
b3 =180mm
b4 =115mm
b5 =410mm
b6 =0mm
b7 =0mm
h1 =180mm
h2 =190mm
h3 =790mm
h4 =115mm
h5 =180mm
h6 =0mm
h7 =0mm
δnt: deflection at transfer
δned: Deflection at errection before super imposed dead load
δnes:deflection at errection after superimposed
δnel:Deflection at erection immediatetly after service load
δnfd:final Deflection before super imposed dead load
δnfs:final Deflection after superimposed dead load
δnfl:final long term deflection service load
δmax:Maximum allowable deflection = L/640
deflection (mm)
L(m)
Cp =50Mpa
Cs =40Mpa
fpk =2000Mpa
prestress loss =25%
Live Loading E =80
ee =-400mm
ec =300mm
girder spacing =1800mm
Slab thikcness =250mm
Beam type =3
Total Depth =1150mm
yb =519.4145mm
Pi =1 * Pimin


Deflection Envelope vs Span Length
delta emax: maximum camber envelope
delta emax: maximum deflection envelope
δmax:Maximum allowable deflection = L/640
b1 =560mm
b2 =190mm
b3 =180mm
b4 =115mm
b5 =410mm
b6 =0mm
b7 =0mm
h1 =180mm
h2 =190mm
h3 =790mm
h4 =115mm
h5 =180mm
h6 =0mm
h7 =0mm
Cp =50Mpa
Cs =40Mpa
fpk =2000Mpa
prestress loss =25%
Live Loading E =80
ee =-400mm
ec =300mm
girder spacing =1800mm
Slab thikcness =250mm
Beam type =3
Total Depth =1150mm
yb =519.4145mm
Pi =1 * Pimin
deflection (mm)
L(m)

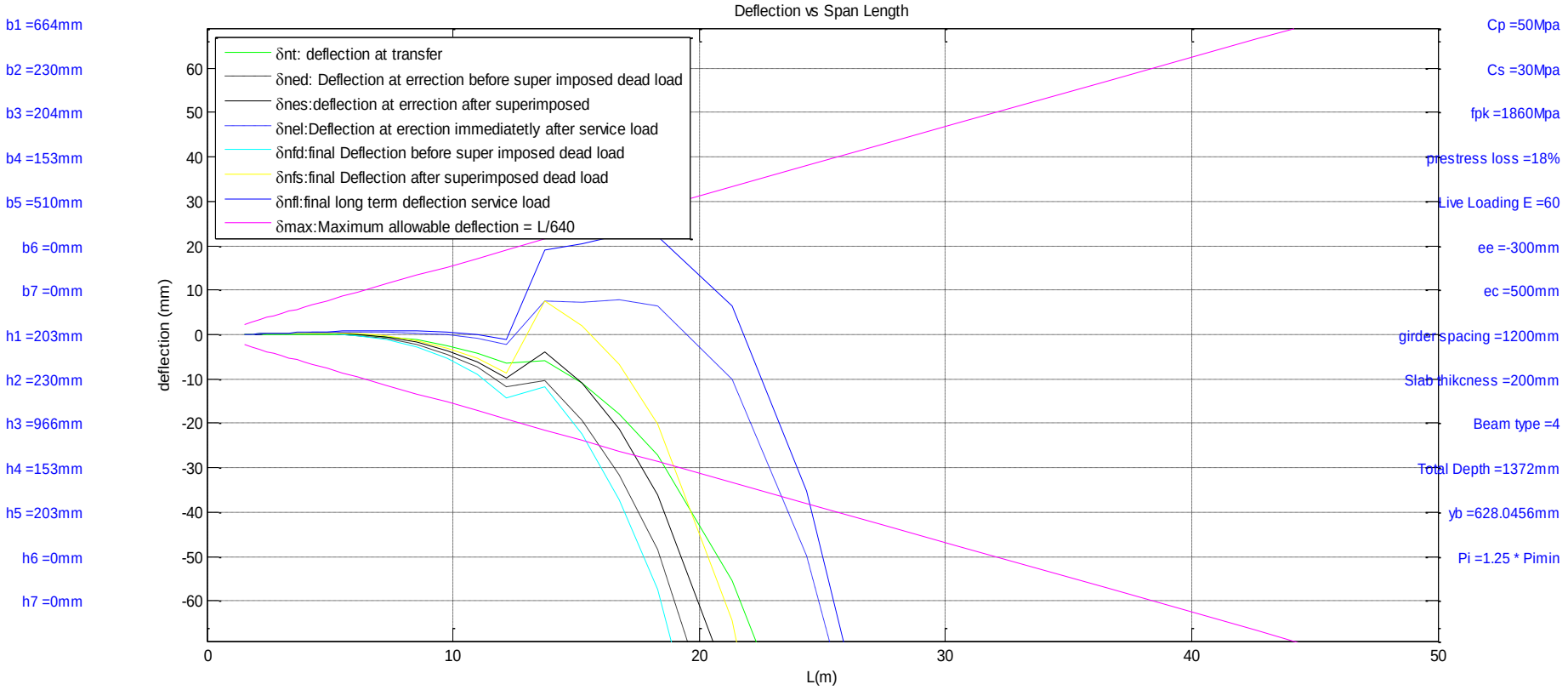

Alloawable eccentricity and Prestressing Force
emax support
emin support
emax mid span
emin mid span
emax for Pmin
Pmin
Pmax
eccentricity (mm)
Prestressing force (MN)
L(m)

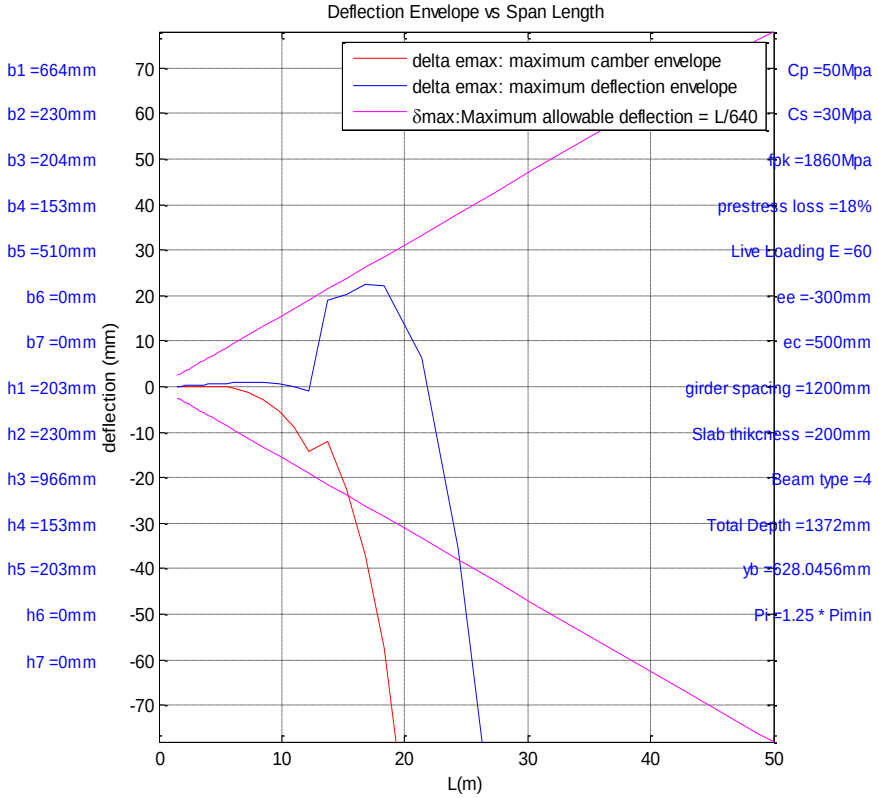

Deflection vs Span Length
δnt: deflection at transfer
δned: Deflection at errection before super imposed dead load
δnes:deflection at errection after superimposed
δnel:Deflection at erection immediatetly after service load
δnfd:final Deflection before super imposed dead load
δnfs:final Deflection after superimposed dead load
δnfl:final long term deflection service load
δmax:Maximum allowable deflection = L/640
deflection (mm)
L(m)
b1 =560mm
b2 =190mm
b3 =180mm
b4 =115mm
b5 =410mm
b6 =0mm
b7 =0mm
h1 =180mm
h2 =190mm
h3 =790mm
h4 =115mm
h5 =180mm
h6 =0mm
h7 =0mm
Cp =50Mpa
Cs =30Mpa
fpk =1600Mpa
prestress loss =20%
Live Loading E =60
ee =0mm
ec =420mm
girder spacing =2000mm
Slab thikcness =200mm
Beam type =3
Total Depth =1150mm
yb =519.4145mm
Pi =1.4 * Pimin

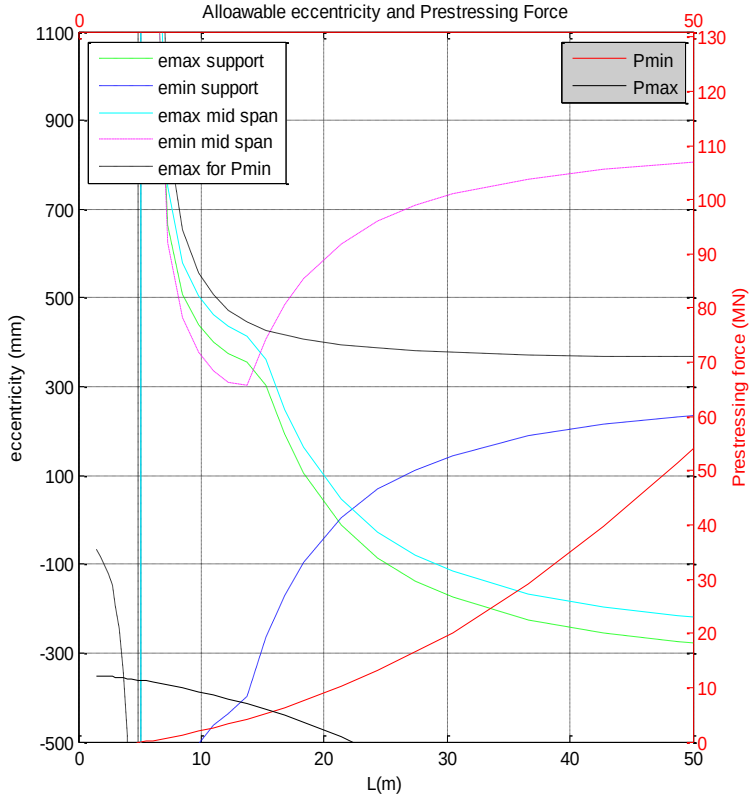

Deflection Envelope vs Span Length
delta emax: maximum camber envelope
delta emax: maximum deflection envelope
δmax:Maximum allowable deflection = L/640
b1 =560mm
b2 =190mm
b3 =180mm
b4 =115mm
b5 =410mm
b6 =0mm
b7 =0mm
h1 =180mm
h2 =190mm
h3 =790mm
h4 =115mm
h5 =180mm
h6 =0mm
h7 =0mm
Cp =50Mpa
Cs =30Mpa
fpk =1600Mpa
prestress loss =20%
Live Loading E =60
ee =0mm
ec =420mm
girder spacing =2000mm
Slab thikcness =200mm
Beam type =3
Total Depth =1150mm
yb =519.4145mm
Pi =1.4 * Pimin
deflection (mm)
L(m)

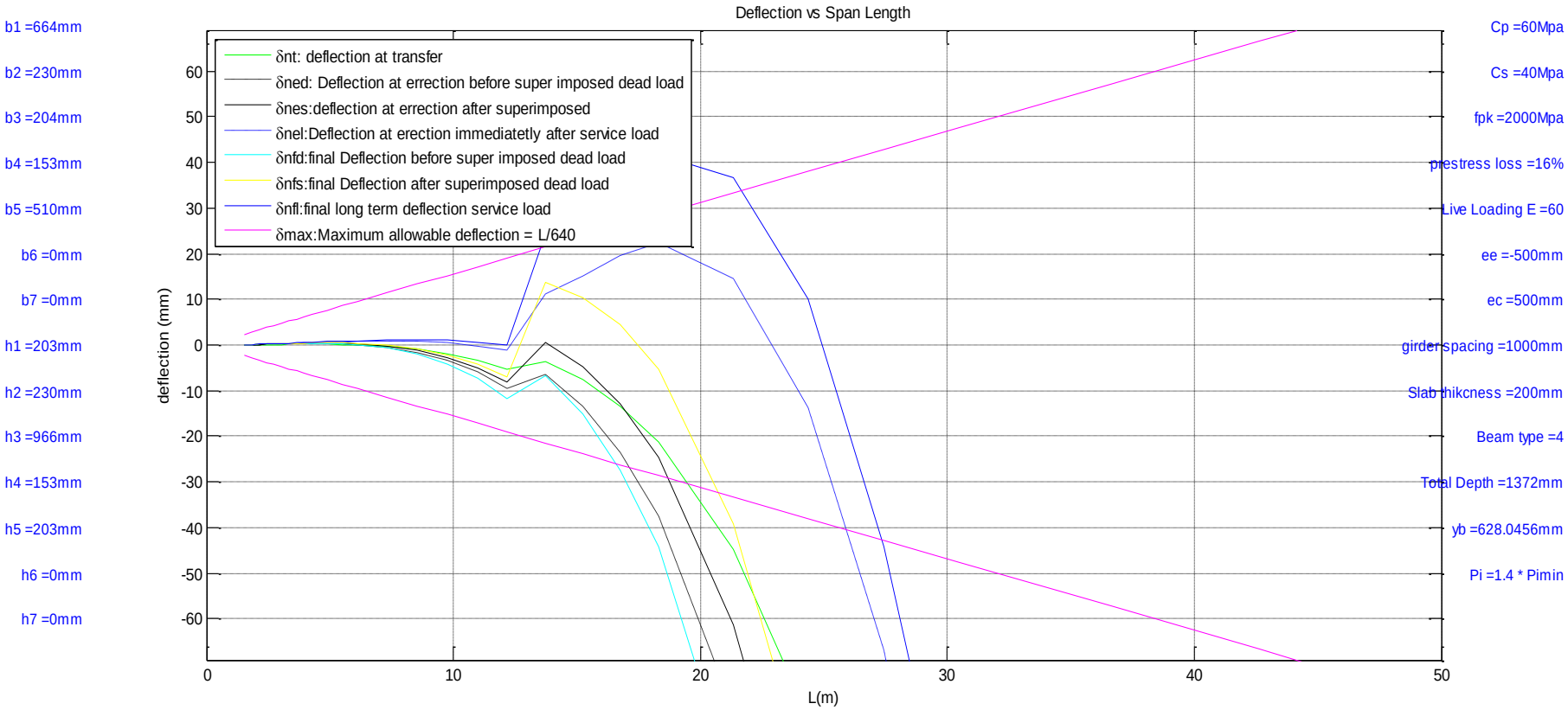

Alloawable eccentricity and Prestressing Force
emax support
emin support
emax mid span
emin mid span
emax for Pmin
Pmin
Pmax
eccentricity (mm)
Prestressing force (MN)
L(m)

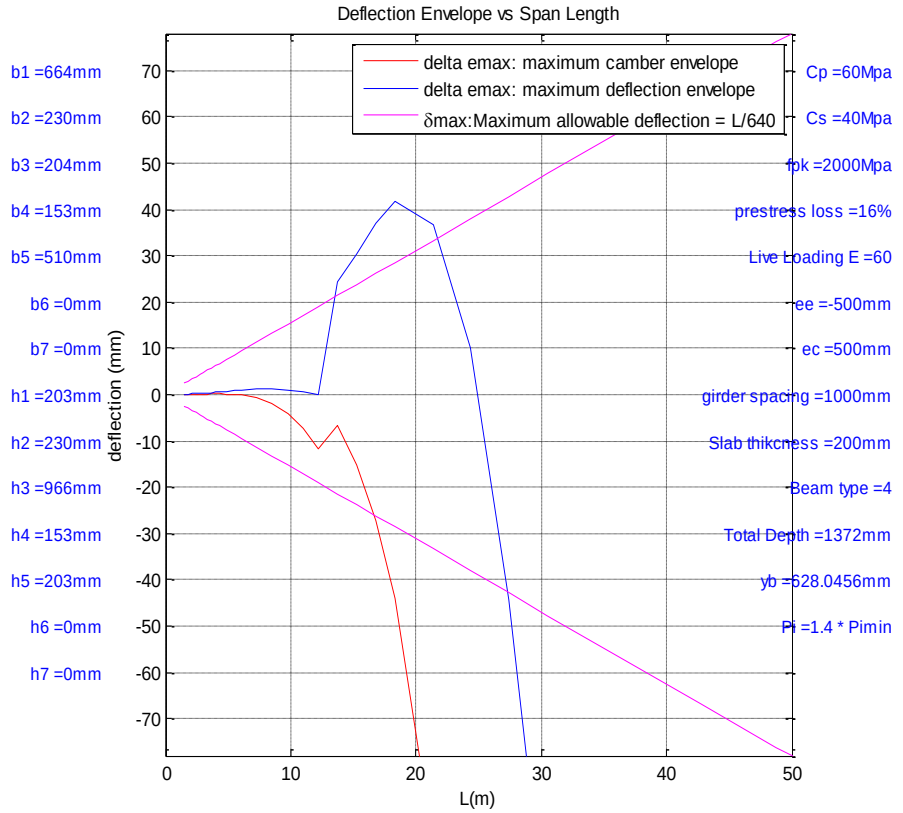
AASHTO/PCI I beam Type IV

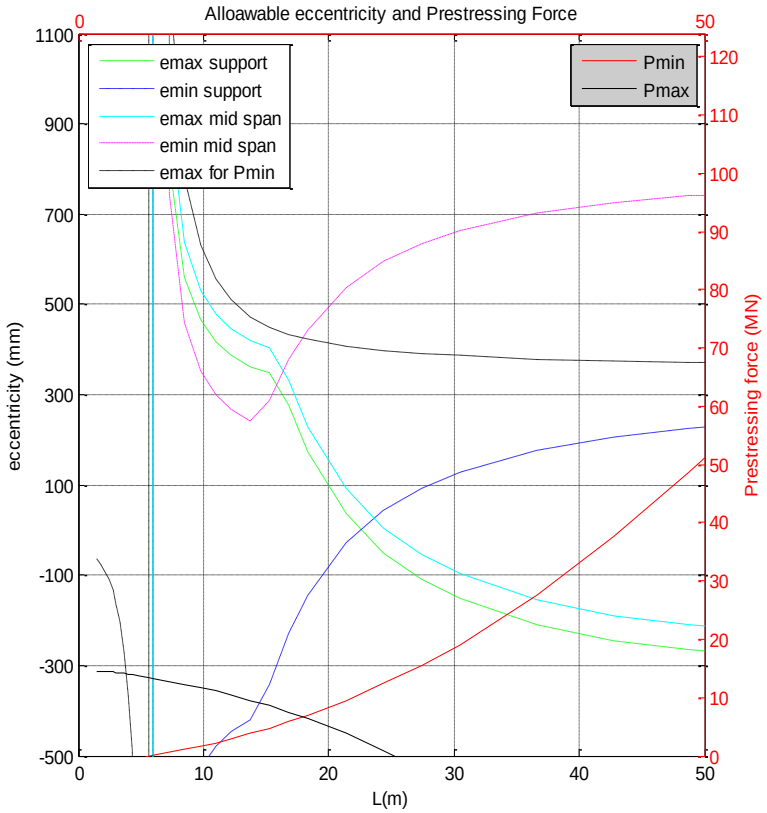


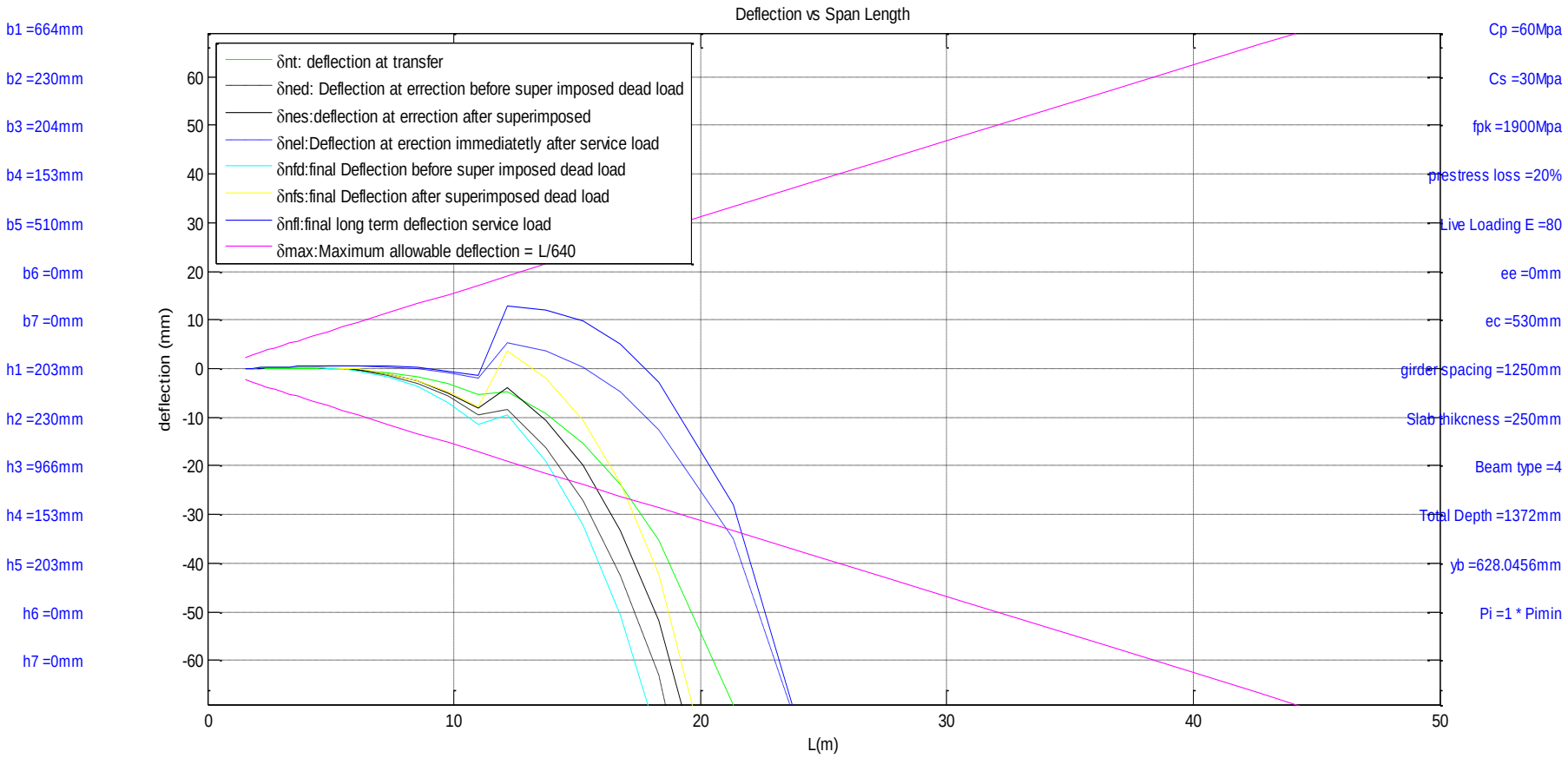

Deflection Envelope vs Span Length
delta emax: maximum camber envelope
delta emax: maximum deflection envelope
δmax:Maximum allowable deflection = L/640
b1 =664mm
b2 =230mm
b3 =204mm
b4 =153mm
b5 =510mm
b6 =0mm
b7 =0mm
h1 =203mm
h2 =230mm
h3 =966mm
h4 =153mm
h5 =203mm
h6 =0mm
h7 =0mm
Cp =50Mpa
Cs =30Mpa
fpk =1860Mpa
prestress loss =18%
Live Loading E =60
ee =-300mm
ec =500mm
girder spacing =1200mm
Slab thikcness =200mm
Beam type =4
Total Depth =1372mm
yb =628.0456mm
Pi=1.25 * Pimin
deflection (mm)
L(m)

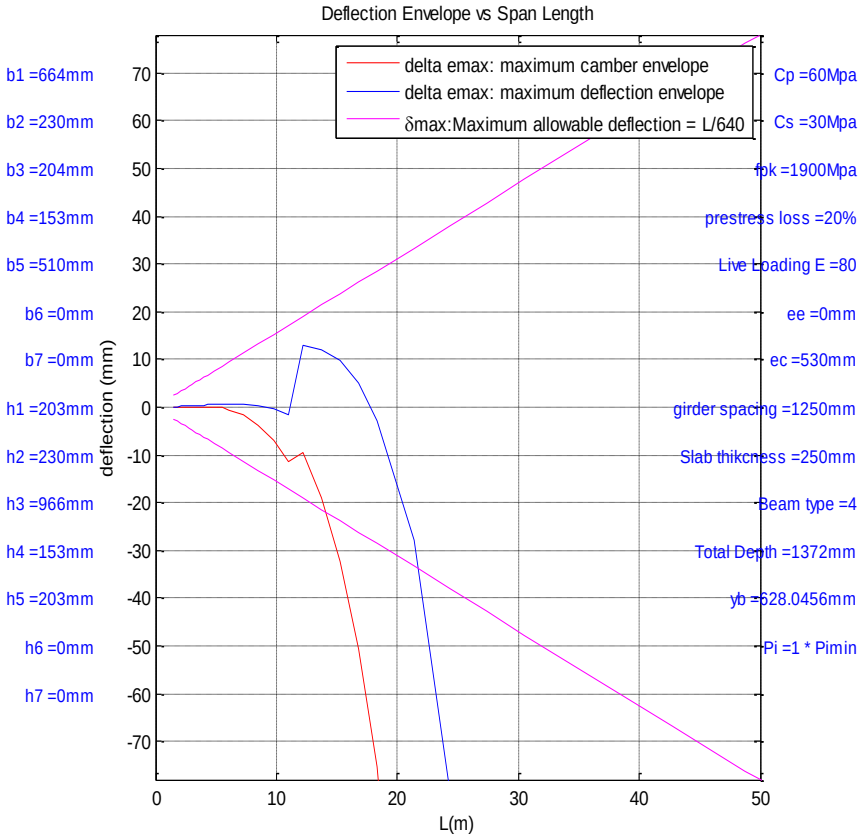

Alloawable eccentricity and Prestressing Force
emax support
emin support
emax mid span
emin mid span
emax for Pmin
Pmin
Pmax
eccentricity (mm)
Prestressing force (MN)
L(m)

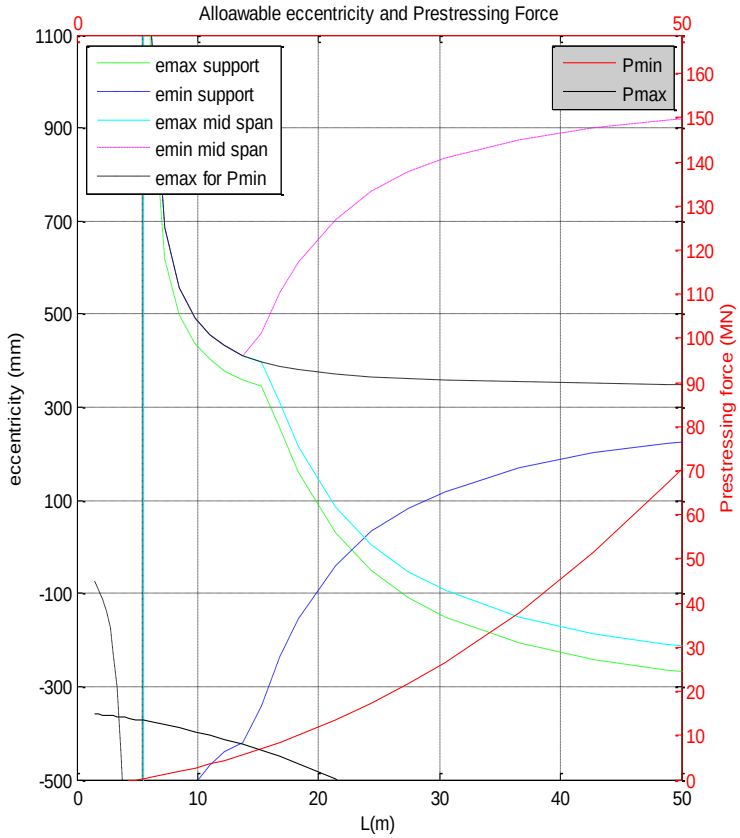

Deflection vs Span Length
δnt: deflection at transfer
δned: Deflection at errection before super imposed dead load
δnes:deflection at errection after superimposed
δnel:Deflection at erection immediatetly after service load
δnfd:final Deflection before super imposed dead load
δnfs:final Deflection after superimposed dead load
δnfl:final long term deflection service load
δmax:Maximum allowable deflection = L/640
deflection (mm)
L(m)
b1 =664mm
b2 =230mm
b3 =204mm
b4 =153mm
b5 =510mm
b6 =0mm
b7 =0mm
h1 =203mm
h2 =230mm
h3 =966mm
h4 =153mm
h5 =203mm
h6 =0mm
h7 =0mm
Cp =60Mpa
Cs =40Mpa
fpk =2000Mpa
prestress loss =16%
Live Loading E =60
ee =-500mm
ec =500mm
girder spacing =1000mm
Slab thikcness =200mm
Beam type =4
Total Depth =1372mm
yb =628.0456mm
Pi =1.4 * Pimin


Deflection Envelope vs Span Length
delta emax: maximum camber envelope
delta emax: maximum deflection envelope
δmax:Maximum allowable deflection = L/640
b1 =664mm
b2 =230mm
b3 =204mm
b4 =153mm
b5 =510mm
b6 =0mm
b7 =0mm
h1 =203mm
h2 =230mm
h3 =966mm
h4 =153mm
h5 =203mm
h6 =0mm
h7 =0mm
Cp =60Mpa
Cs =40Mpa
fpk =2000Mpa
prestress loss =16%
Live Loading E =60
ee =-500mm
ec =500mm
girder spacing =1000mm
Slab thikcness =200mm
Beam type =4
Total Depth =1372mm
yb =528.0456mm
Pi =1.4 * Pimin
deflection (mm)
L(m)

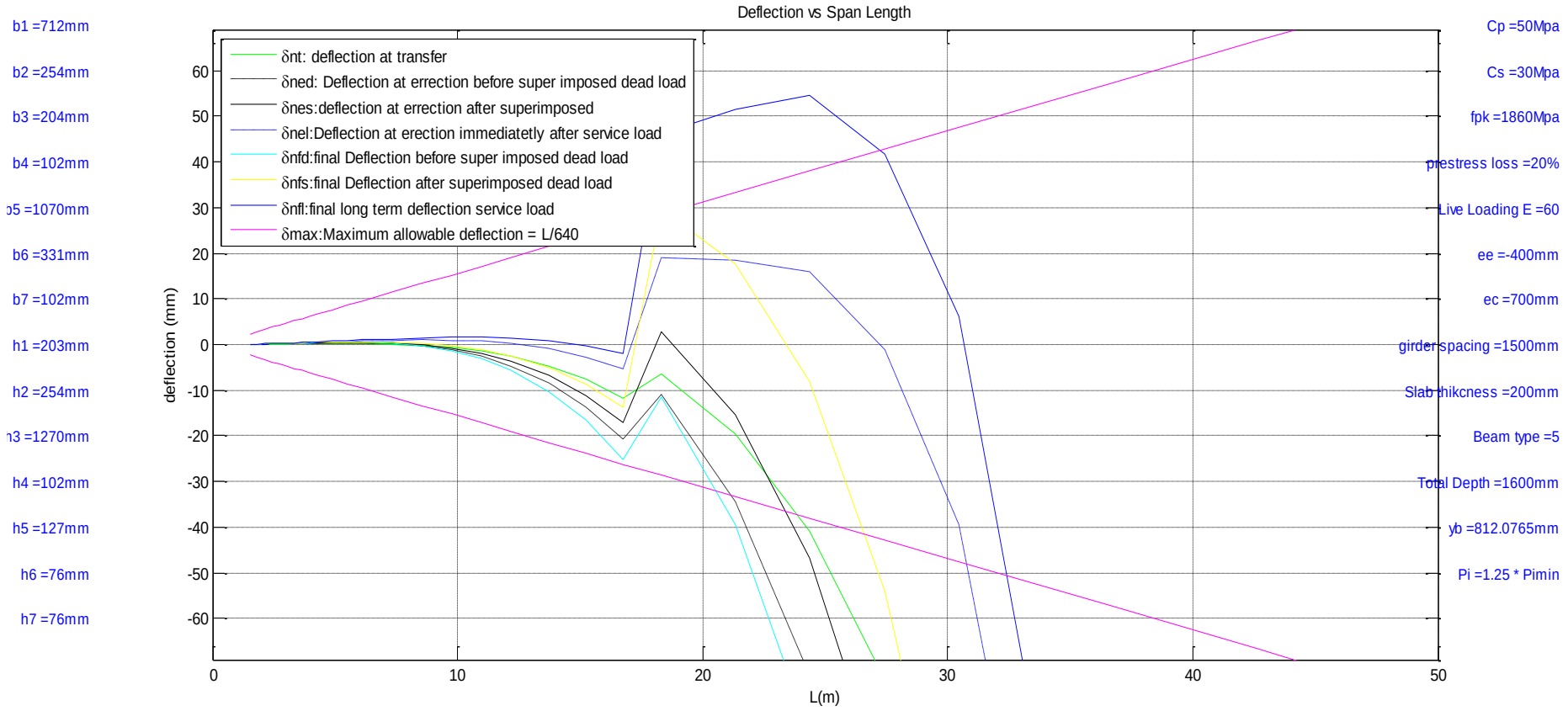

Alloawable eccentricity and Prestressing Force
emax support
emin support
emax mid span
emin mid span
emax for Pmin
Pmin
Pmax
eccentricity (mm)
Prestressing force (MN)
L(m)

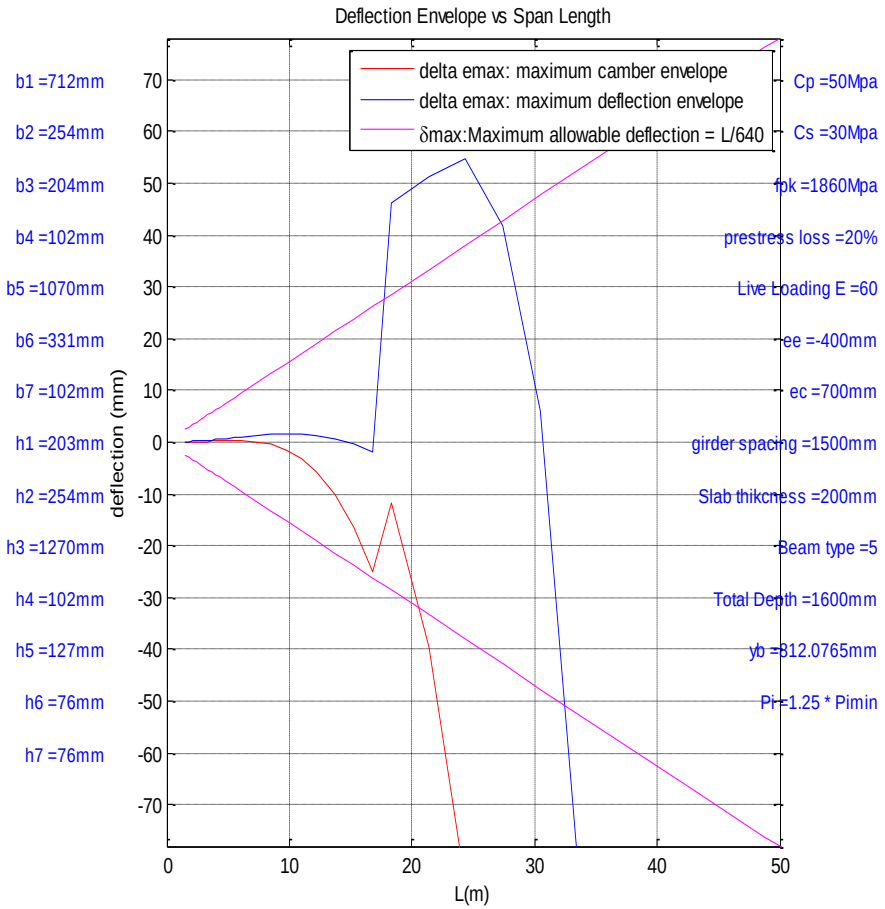

Deflection vs Span Length
b1 =664mm
b2 =230mm
b3 =204mm
b4 =153mm
b5 =510mm
b6 =0mm
b7 =0mm
h1 =203mm
h2 =230mm
h3 =966mm
h4 =153mm
h5 =203mm
h6 =0mm
h7 =0mm
δnt: deflection at transfer
δned: Deflection at errection before super imposed dead load
δnes:deflection at errection after superimposed
δnel:Deflection at erection immediatetly after service load
δnfd:final Deflection before super imposed dead load
δnfs:final Deflection after superimposed dead load
δnfl:final long term deflection service load
δmax:Maximum allowable deflection = L/640
deflection (mm)
L(m)
Cp =60Mpa
Cs =30Mpa
fpk =1900Mpa
prestress loss =20%
Live Loading E =80
ee =0mm
ec =530mm
girder spacing =1250mm
Slab thikcness =250mm
Beam type =4
Total Depth =1372mm
yb =628.0456mm
Pi =1 * Pimin

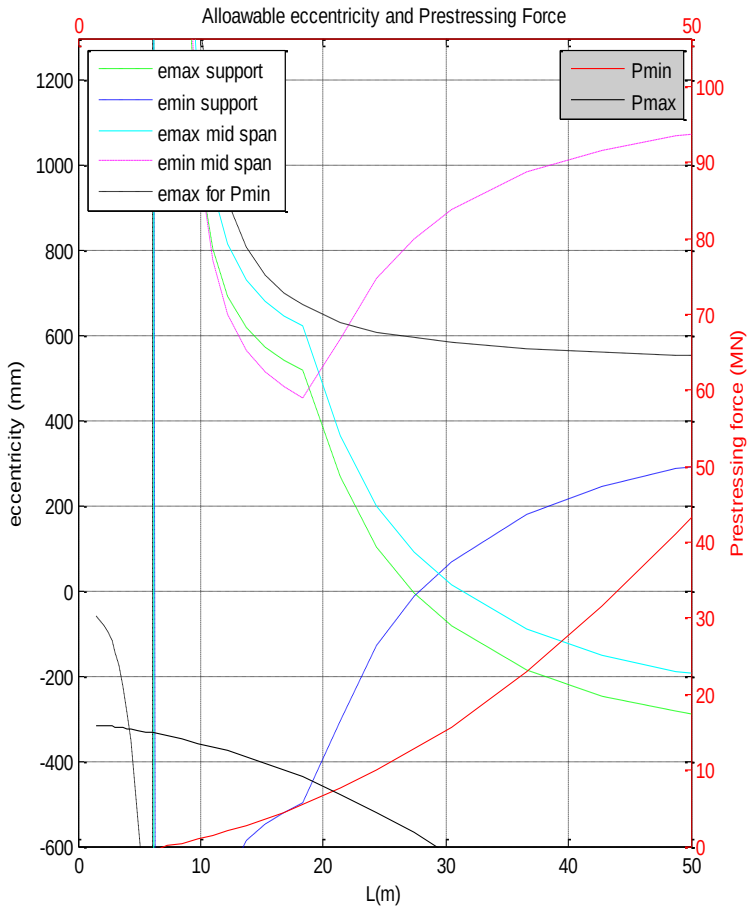

Deflection Envelope vs Span Length
delta emax: maximum camber envelope
delta emax: maximum deflection envelope
δmax:Maximum allowable deflection = L/640
b1 =664mm
b2 =230mm
b3 =204mm
b4 =153mm
b5 =510mm
b6 =0mm
b7 =0mm
h1 =203mm
h2 =230mm
h3 =966mm
h4 =153mm
h5 =203mm
h6 =0mm
h7 =0mm
Cp =60Mpa
Cs =30Mpa
fpk =1900Mpa
prestress loss =20%
Live Loading E =80
ee =0mm
ec =530mm
girder spacing =1250mm
Slab thikcness =250mm
Beam type =4
Total Depth =1372mm
yb =628.0456mm
Pi =1 * Pimin
deflection (mm)
L(m)

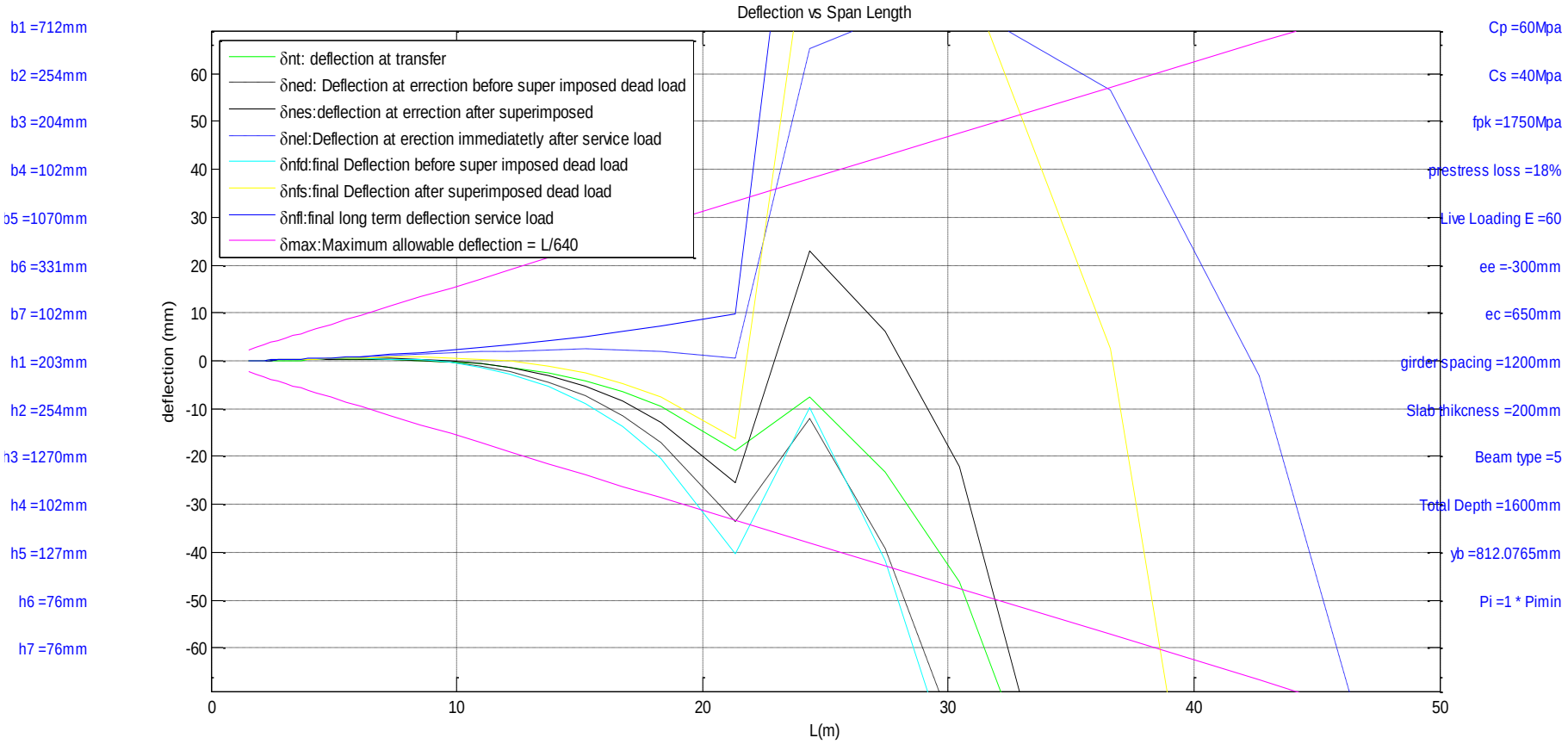

Alloawable eccentricity and Prestressing Force
emax support
emin support
emax mid span
emin mid span
emax for Pmin
Pmin
Pmax
eccentricity (mm)
Prestressing force (MN)
L(m)

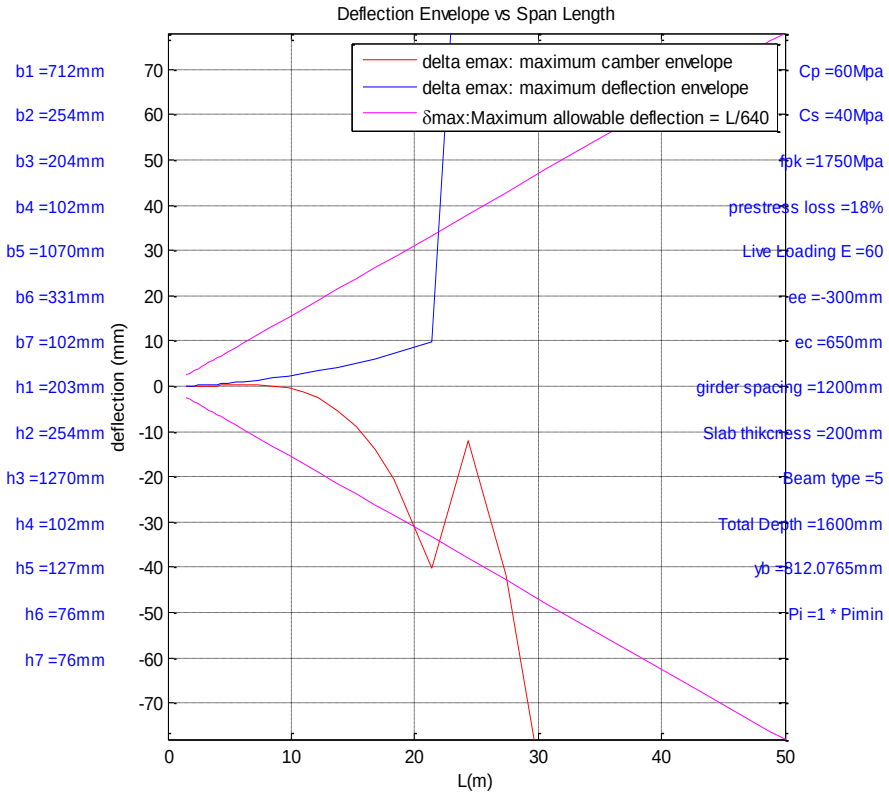
AASHTO/PCI I beam Type V

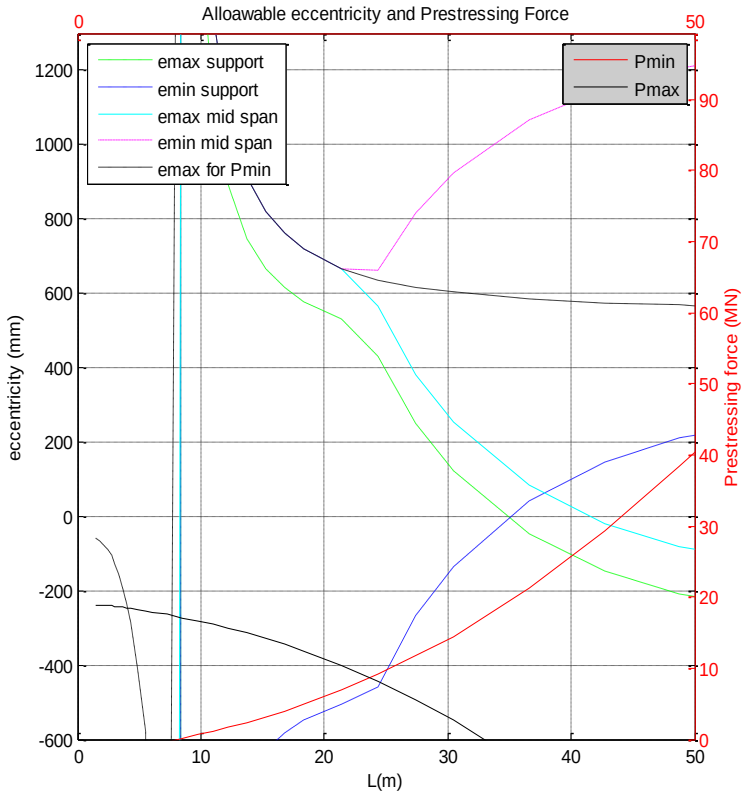


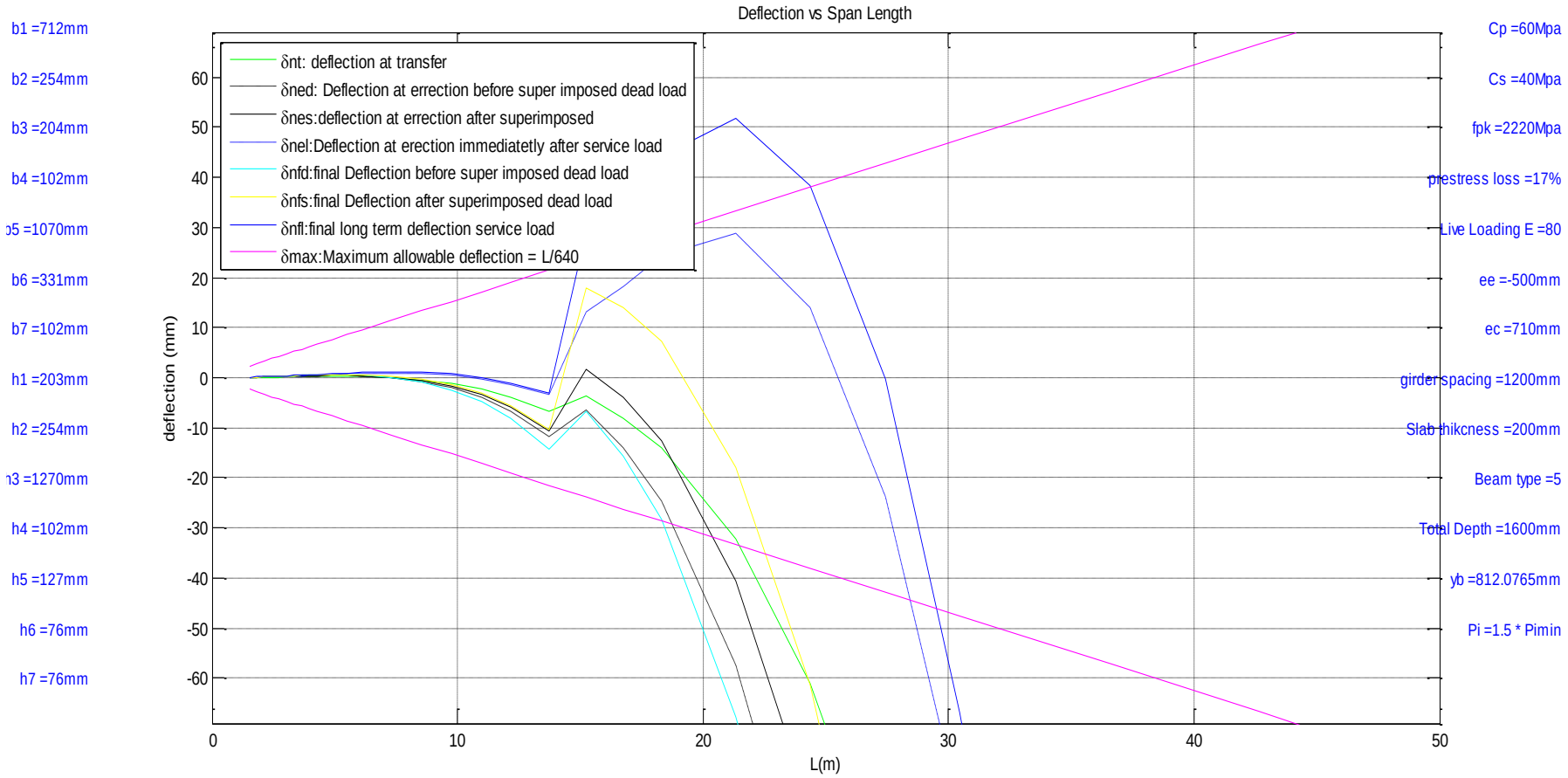

Deflection Envelope vs Span Length
delta emax: maximum camber envelope
delta emax: maximum deflection envelope
δmax:Maximum allowable deflection = L/640
b1 =712mm
b2 =254mm
b3 =204mm
b4 =102mm
b5 =1070mm
b6 =331mm
b7 =102mm
h1 =203mm
h2 =254mm
h3 =1270mm
h4 =102mm
h5 =127mm
h6 =76mm
h7 =76mm
Cp =50Mpa
Cs =30Mpa
fpk =1860Mpa
prestress loss =20%
Live Loading E =60
ee =-400mm
ec =700mm
girder spacing =1500mm
Slab thikcness =200mm
Beam type =5
Total Depth =1600mm
yb =812.0765mm
Pi =1.25 * Pimin
deflection (mm)
L(m)

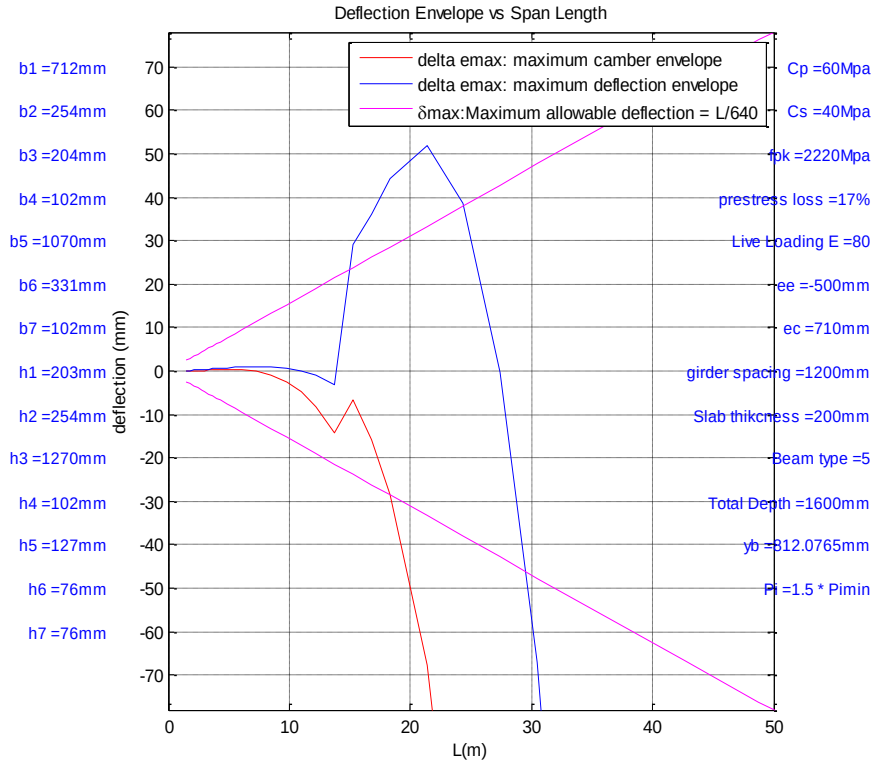

Alloawable eccentricity and Prestressing Force
emax support
emin support
emax mid span
emin mid span
emax for Pmin
Pmin
Pmax
eccentricity (mm)
Prestressing force (MN)
L(m)

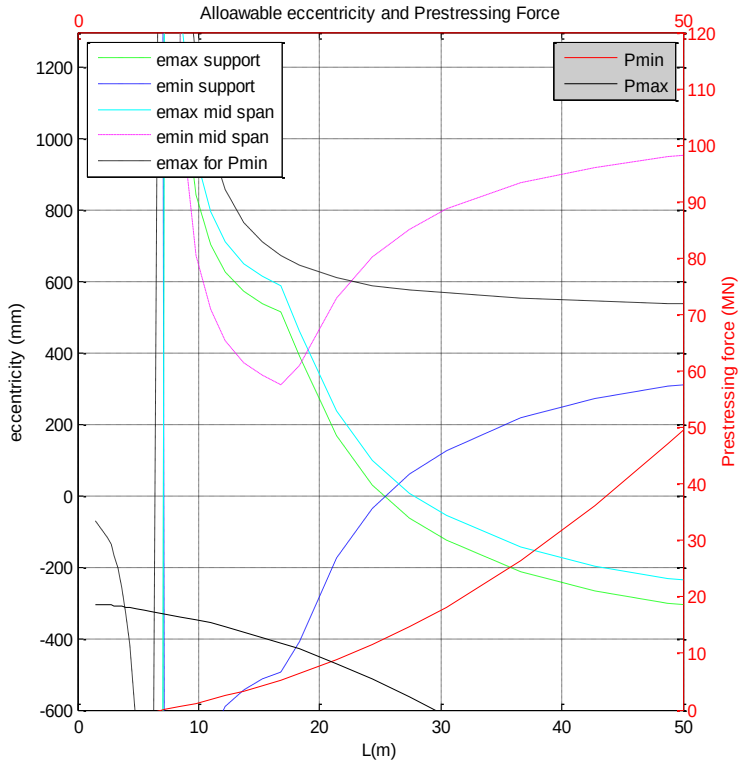

Deflection vs Span Length
δnt: deflection at transfer
δned: Deflection at errection before super imposed dead load
δnes:deflection at errection after superimposed
δnel:Deflection at erection immediatetly after service load
δnfd:final Deflection before super imposed dead load
δnfs:final Deflection after superimposed dead load
δnfl:final long term deflection service load
δmax:Maximum allowable deflection = L/640
deflection (mm)
L(m)
b1 =712mm
b2 =254mm
b3 =204mm
b4 =102mm
b5 =1070mm
b6 =331mm
b7 =102mm
h1 =203mm
h2 =254mm
h3 =1270mm
h4 =102mm
h5 =127mm
h6 =76mm
h7 =76mm
Cp =60Mpa
Cs =40Mpa
fpk =1750Mpa
prestress loss =18%
Live Loading E =60
ee =-300mm
ec =650mm
girder spacing =1200mm
Slab thikcness =200mm
Beam type =5
Total Depth =1600mm
yb =812.0765mm
Pi =1 * Pimin


Deflection Envelope vs Span Length
delta emax: maximum camber envelope
delta emax: maximum deflection envelope
δmax:Maximum allowable deflection = L/640
b1 =712mm
b2 =254mm
b3 =204mm
b4 =102mm
b5 =1070mm
b6 =331mm
b7 =102mm
h1 =203mm
h2 =254mm
h3 =1270mm
h4 =102mm
h5 =127mm
h6 =76mm
h7 =76mm
Cp =60Mpa
Cs =40Mpa
fpk =1750Mpa
prestress loss =18%
Live Loading E =60
ee =-300mm
ec =650mm
girder spacing =1200mm
Slab thikcness =200mm
Beam type =5
Total Depth =1600mm
yb =812.0765mm
Pi =1 * Pimin
deflection (mm)
L(m)

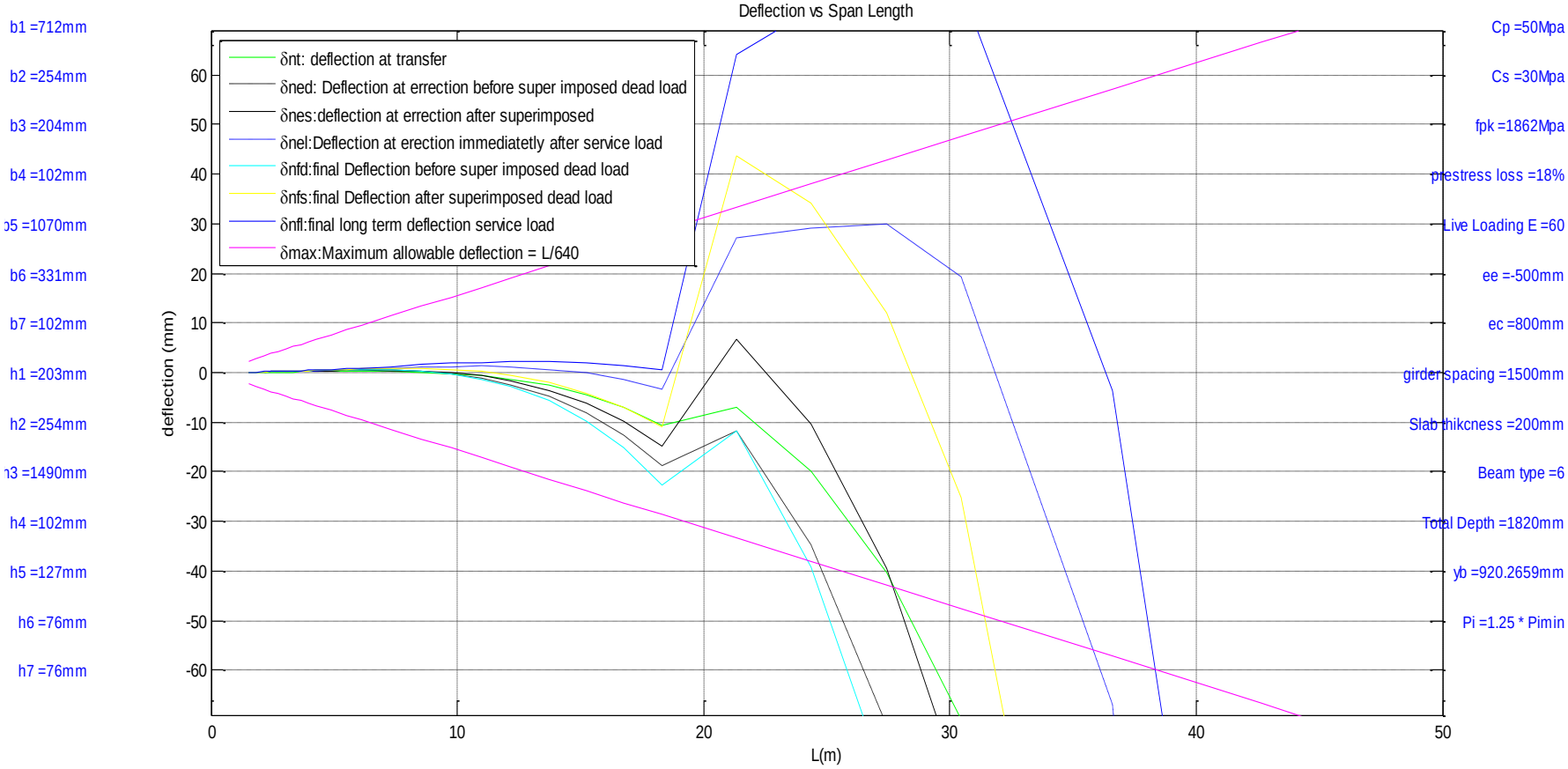

Alloawable eccentricity and Prestressing Force
emax support
emin support
emax mid span
emin mid span
emax for Pmin
Pmin
Pmax
eccentricity (mm)
Prestressing force (MN)
L(m)

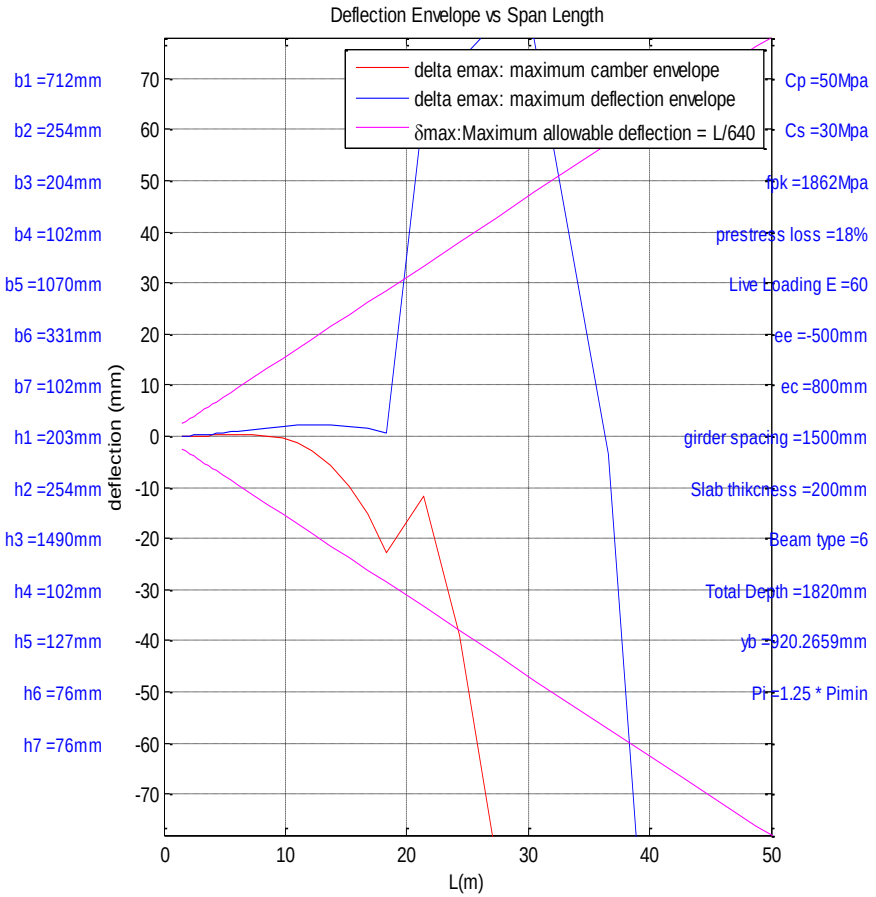

Deflection vs Span Length
δnt: deflection at transfer
δned: Deflection at errection before super imposed dead load
δnes:deflection at errection after superimposed
δnel:Deflection at erection immediatetly after service load
δnfd:final Deflection before super imposed dead load
δnfs:final Deflection after superimposed dead load
δnfl:final long term deflection service load
δmax:Maximum allowable deflection = L/640
deflection (mm)
L(m)
b1 =712mm
b2 =254mm
b3 =204mm
b4 =102mm
b5 =1070mm
b6 =331mm
b7 =102mm
h1 =203mm
h2 =254mm
h3 =1270mm
h4 =102mm
h5 =127mm
h6 =76mm
h7 =76mm
Cp =60Mpa
Cs =40Mpa
fpk =2220Mpa
prestress loss =17%
Live Loading E =80
ee =-500mm
ec =710mm
girder spacing =1200mm
Slab thikcness =200mm
Beam type =5
Total Depth =1600mm
yb =812.0765mm
Pi =1.5 * Pimin

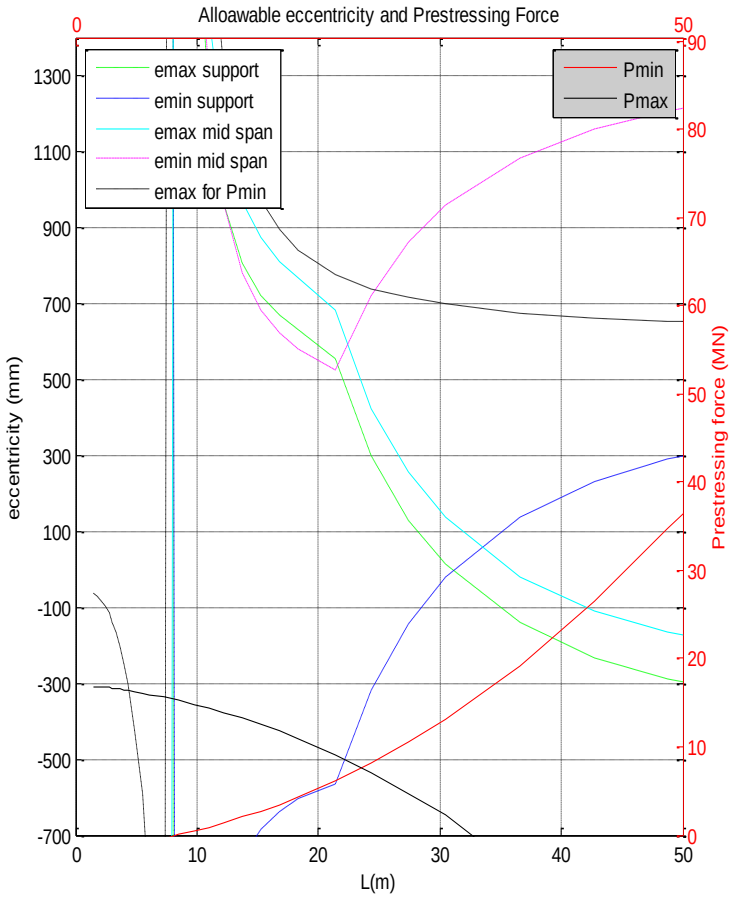

Deflection Envelope vs Span Length
delta emax: maximum camber envelope
delta emax: maximum deflection envelope
δmax:Maximum allowable deflection = L/640
b1 =712mm
b2 =254mm
b3 =204mm
b4 =102mm
b5 =1070mm
b6 =331mm
b7 =102mm
h1 =203mm
h2 =254mm
h3 =1270mm
h4 =102mm
h5 =127mm
h6 =76mm
h7 =76mm
Cp =60Mpa
Cs =40Mpa
fpk =2220Mpa
prestress loss =17%
Live Loading E =80
ee =-500mm
ec =710mm
girder spacing =1200mm
Slab thikcness =200mm
Beam type =5
Total Depth =1600mm
yb =812.0765mm
Pi =1.5 * Pimin
deflection (mm)
L(m)

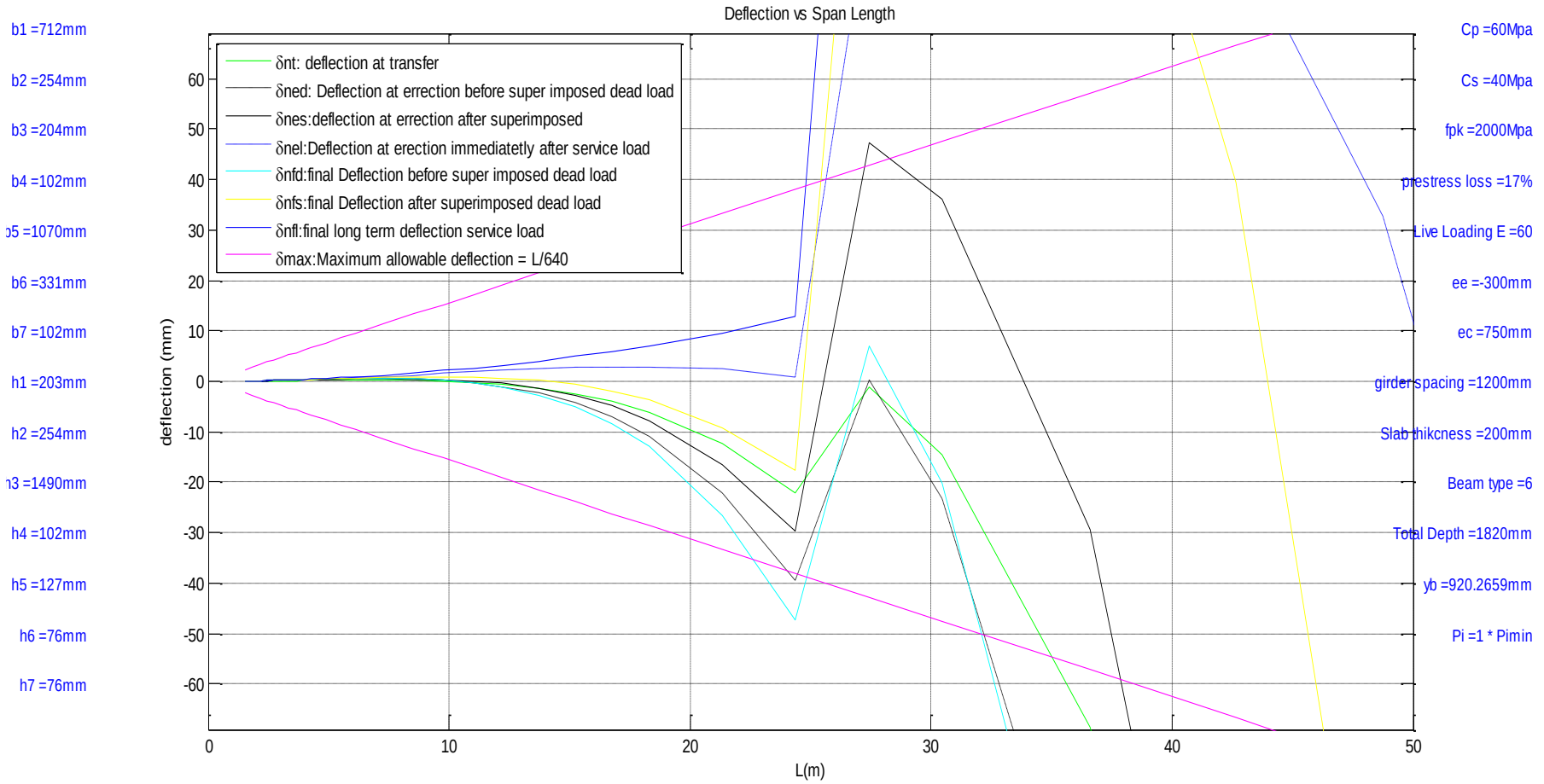

Alloawable eccentricity and Prestressing Force
emax support
emin support
emax mid span
emin mid span
emax for Pmin
Pmin
Pmax
eccentricity (mm)
Prestressing force (MN)
L(m)

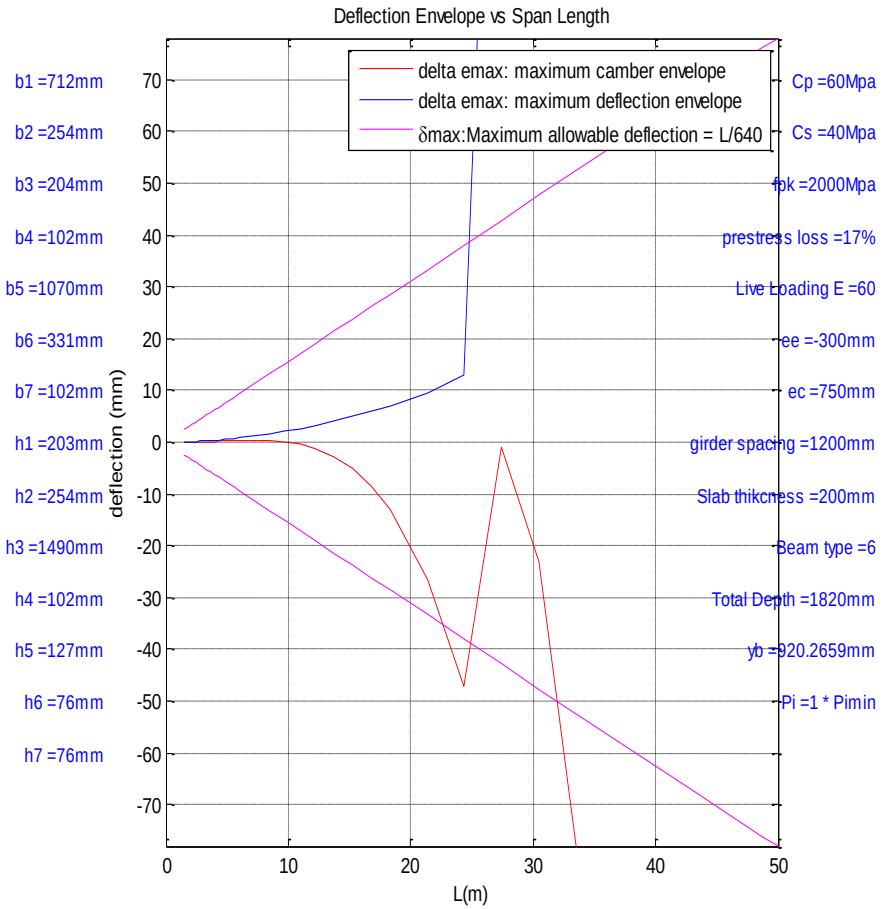
AASHTO/PCI I beam Type VI

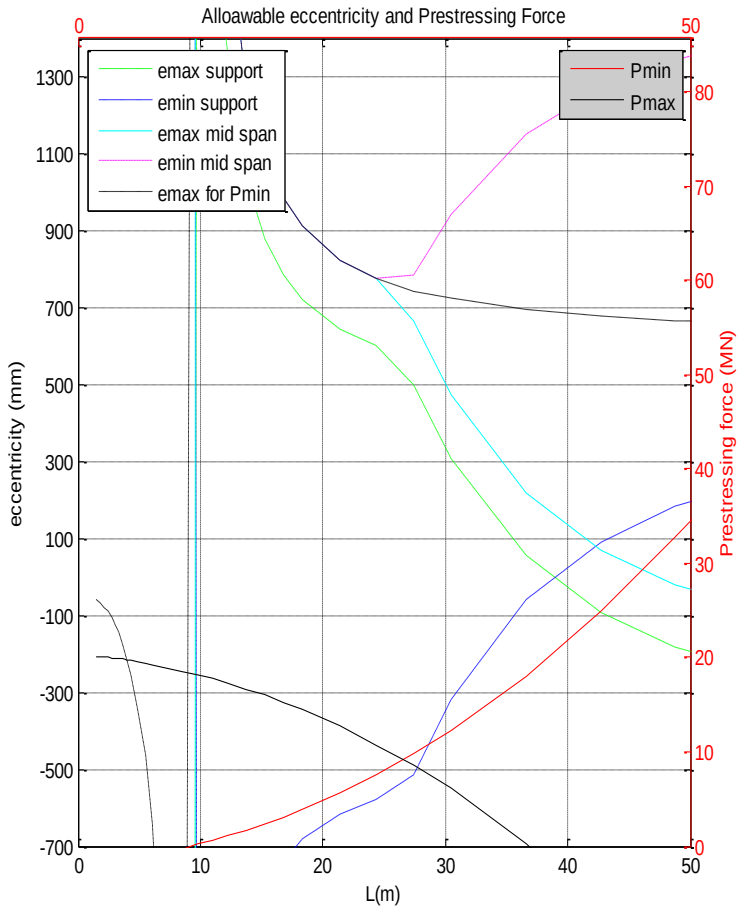


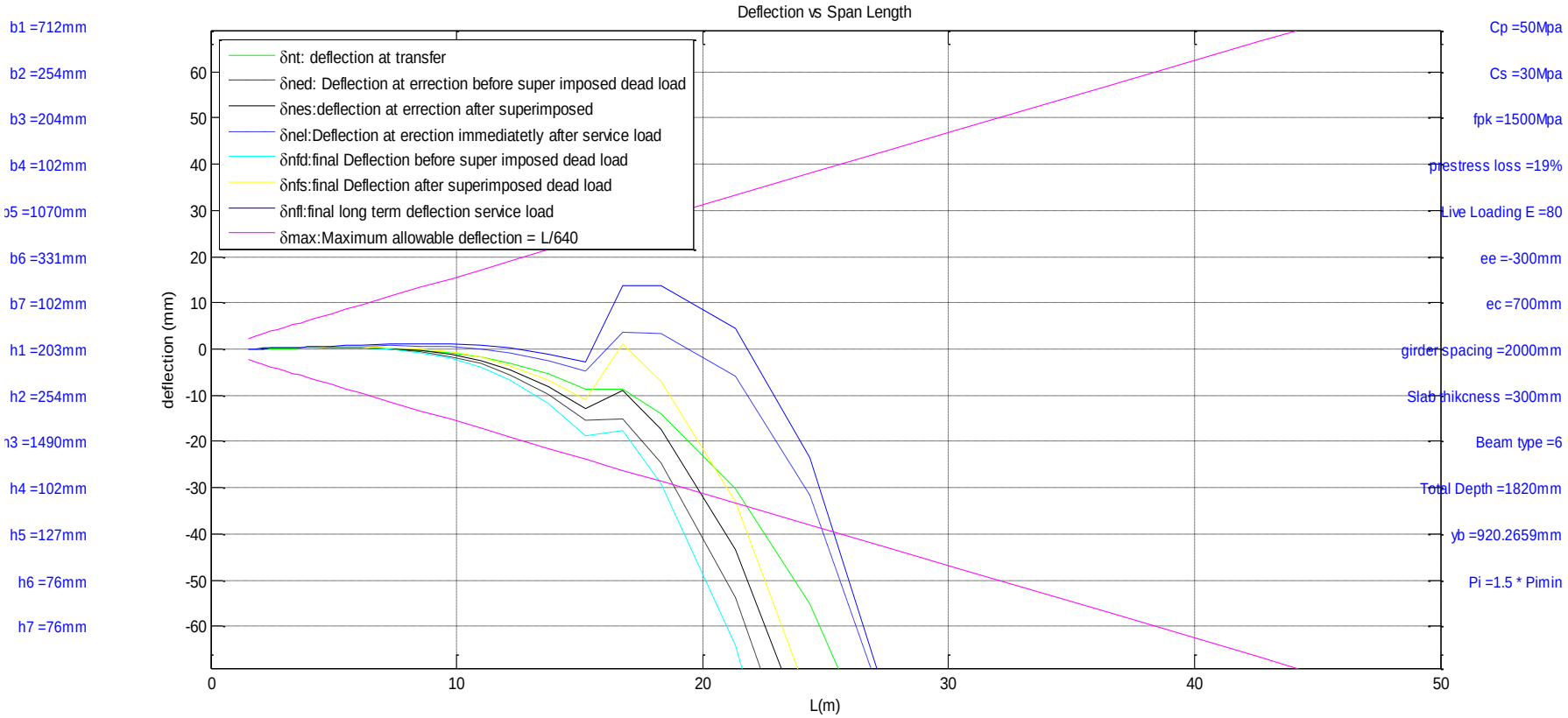

Deflection Envelope vs Span Length
delta emax: maximum camber envelope
delta emax: maximum deflection envelope
δmax:Maximum allowable deflection = L/640
b1 =712mm
b2 =254mm
b3 =204mm
b4 =102mm
b5 =1070mm
b6 =331mm
b7 =102mm
h1 =203mm
h2 =254mm
h3 =1490mm
h4 =102mm
h5 =127mm
h6 =76mm
h7 =76mm
Cp =50Mpa
Cs =30Mpa
fpk =1862Mpa
prestress loss =18%
Live Loading E =60
ee =-500mm
ec =800mm
girder spacing =1500mm
Slab thikcness =200mm
Beam type =6
Total Depth =1820mm
yb =920.2659mm
Pi=1.25 * Pimin
deflection (mm)
L(m)

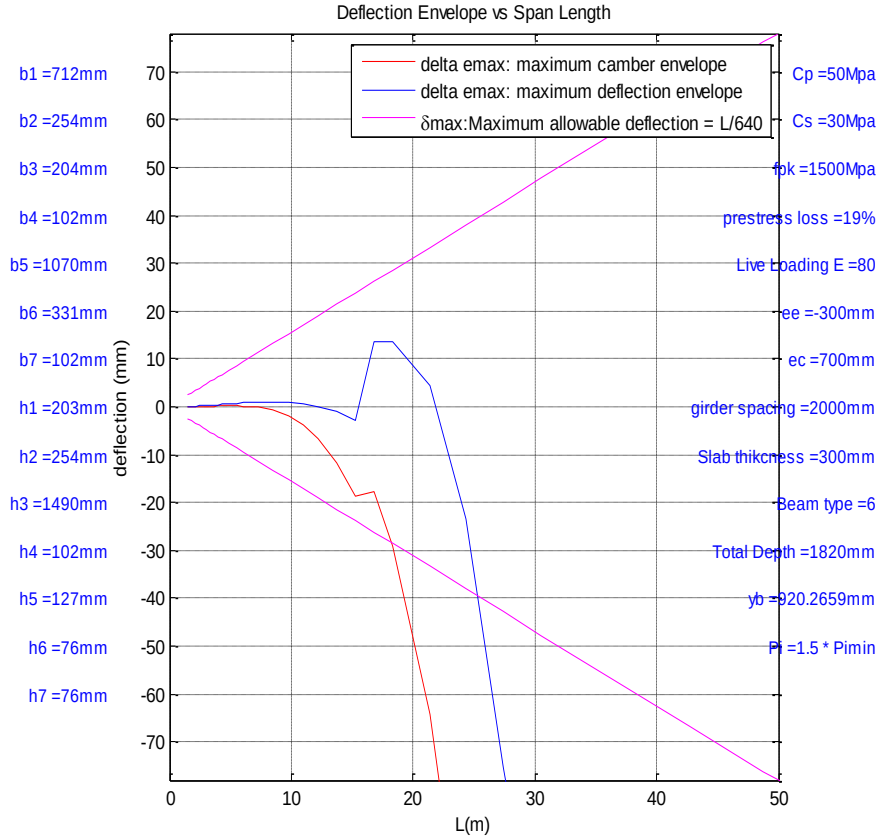

Alloawable eccentricity and Prestressing Force
emax support
emin support
emax mid span
emin mid span
emax for Pmin
Pmin
Pmax
eccentricity (mm)
Prestressing force (MN)
L(m)

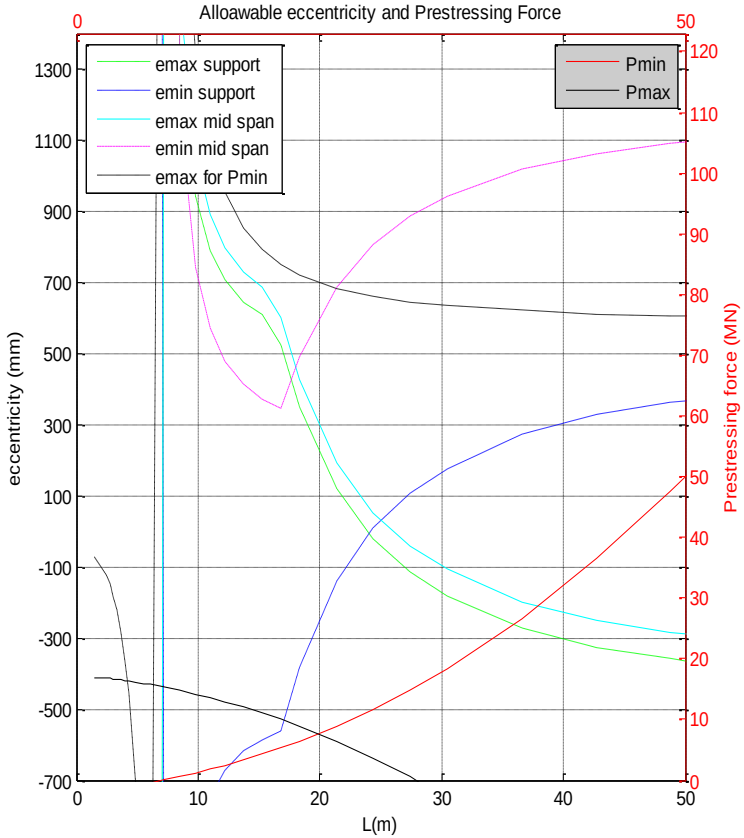

Deflection vs Span Length
δnt: deflection at transfer
δned: Deflection at errection before super imposed dead load
δnes:deflection at errection after superimposed
δnel:Deflection at erection immediatetly after service load
δnfd:final Deflection before super imposed dead load
δnfs:final Deflection after superimposed dead load
δnfl:final long term deflection service load
δmax:Maximum allowable deflection = L/640
deflection (mm)
L(m)
b1 =712mm
b2 =254mm
b3 =204mm
b4 =102mm
b5 =1070mm
b6 =331mm
b7 =102mm
h1 =203mm
h2 =254mm
h3 =1490mm
h4 =102mm
h5 =127mm
h6 =76mm
h7 =76mm
Cp =60Mpa
Cs =40Mpa
fpk =2000Mpa
prestress loss =17%
Live Loading E =60
ee =-300mm
ec =750mm
girder spacing =1200mm
Slab thikcness =200mm
Beam type =6
Total Depth =1820mm
yb =920.2659mm
Pi =1 * Pimin


Deflection Envelope vs Span Length
delta emax: maximum camber envelope
delta emax: maximum deflection envelope
δmax:Maximum allowable deflection = L/640
b1 =712mm
b2 =254mm
b3 =204mm
b4 =102mm
b5 =1070mm
b6 =331mm
b7 =102mm
h1 =203mm
h2 =254mm
h3 =1490mm
h4 =102mm
h5 =127mm
h6 =76mm
h7 =76mm
Cp =60Mpa
Cs =40Mpa
fpk =2000Mpa
prestress loss =17%
Live Loading E =60
ee =-300mm
ec =750mm
girder spacing =1200mm
Slab thikcness =200mm
Beam type =6
Total Depth =1820mm
yb =920.2659mm
Pi =1 * Pimin
deflection (mm)
L(m)

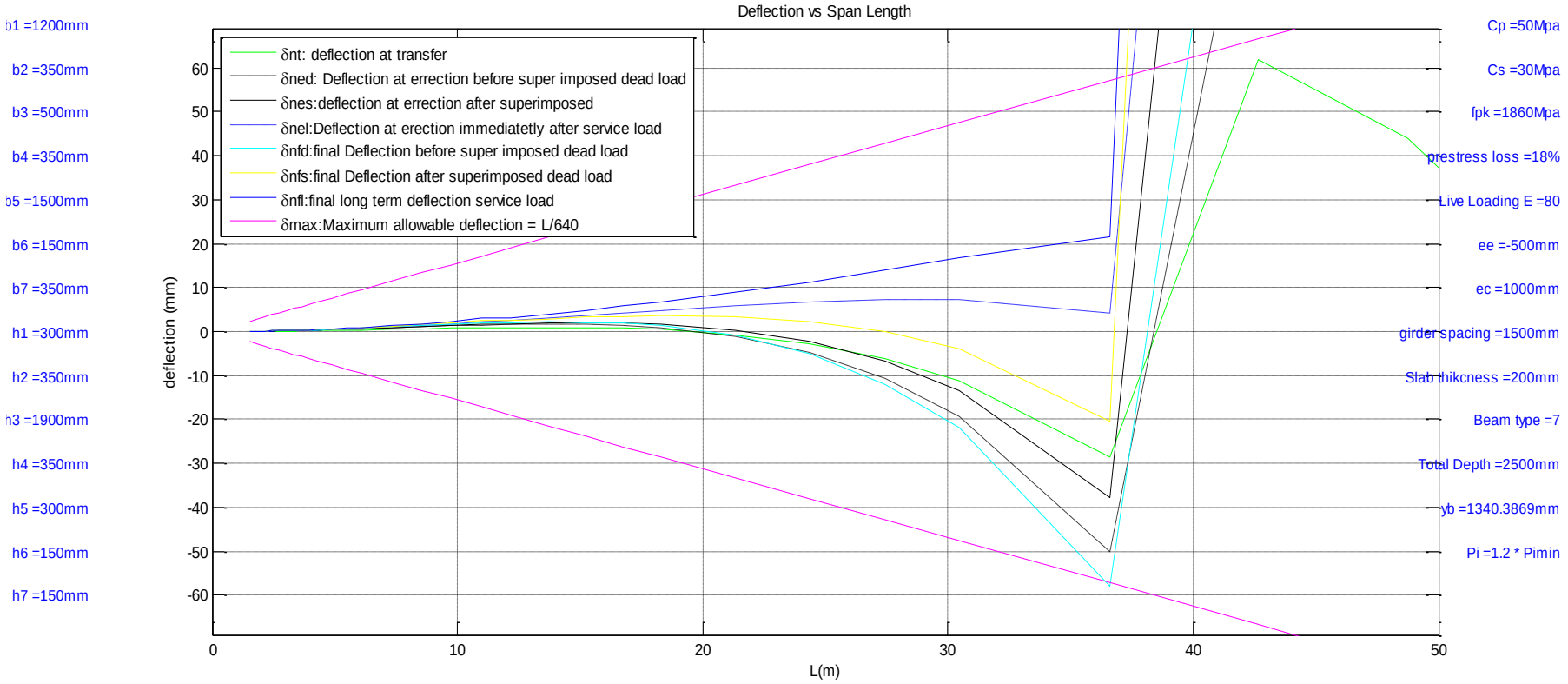

Alloawable eccentricity and Prestressing Force
emax support
emin support
emax mid span
emin mid span
emax for Pmin
Pmin
Pmax
eccentricity (mm)
Prestressing force (MN)
L(m)

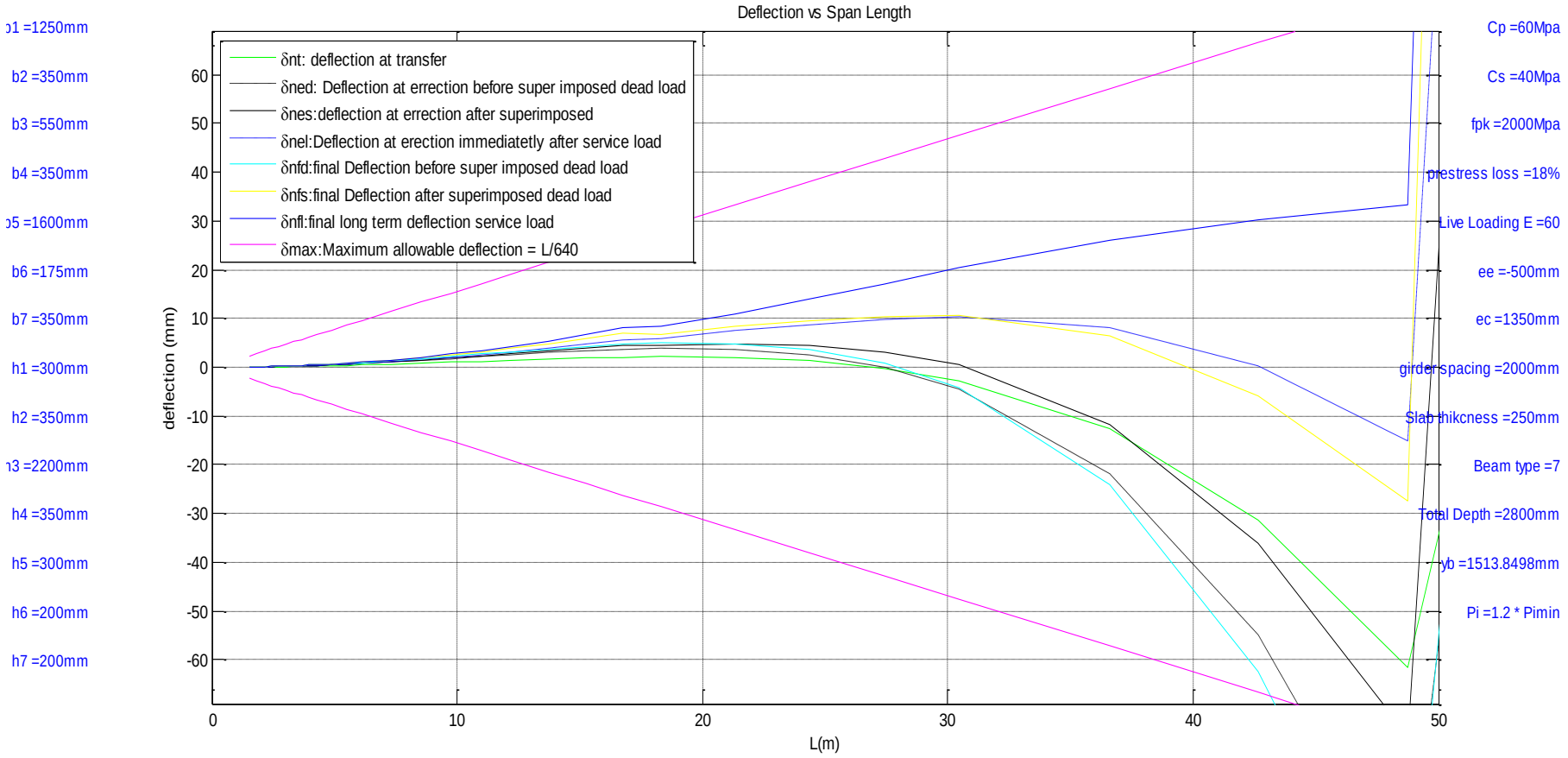

Deflection vs Span Length
δnt: deflection at transfer
δned: Deflection at errection before super imposed dead load
δnes:deflection at errection after superimposed
δnel:Deflection at erection immediatetly after service load
δnfd:final Deflection before super imposed dead load
δnfs:final Deflection after superimposed dead load
δnfl:final long term deflection service load
δmax:Maximum allowable deflection = L/640
deflection (mm)
L(m)
b1 =712mm
b2 =254mm
b3 =204mm
b4 =102mm
b5 =1070mm
b6 =331mm
b7 =102mm
h1 =203mm
h2 =254mm
h3 =1490mm
h4 =102mm
h5 =127mm
h6 =76mm
h7 =76mm
Cp =50Mpa
Cs =30Mpa
fpk =1500Mpa
prestress loss =19%
Live Loading E =80
ee =-300mm
ec =700mm
girder spacing =2000mm
Slab thikcness =300mm
Beam type =6
Total Depth =1820mm
yb =920.2659mm
Pi =1.5 * Pimin

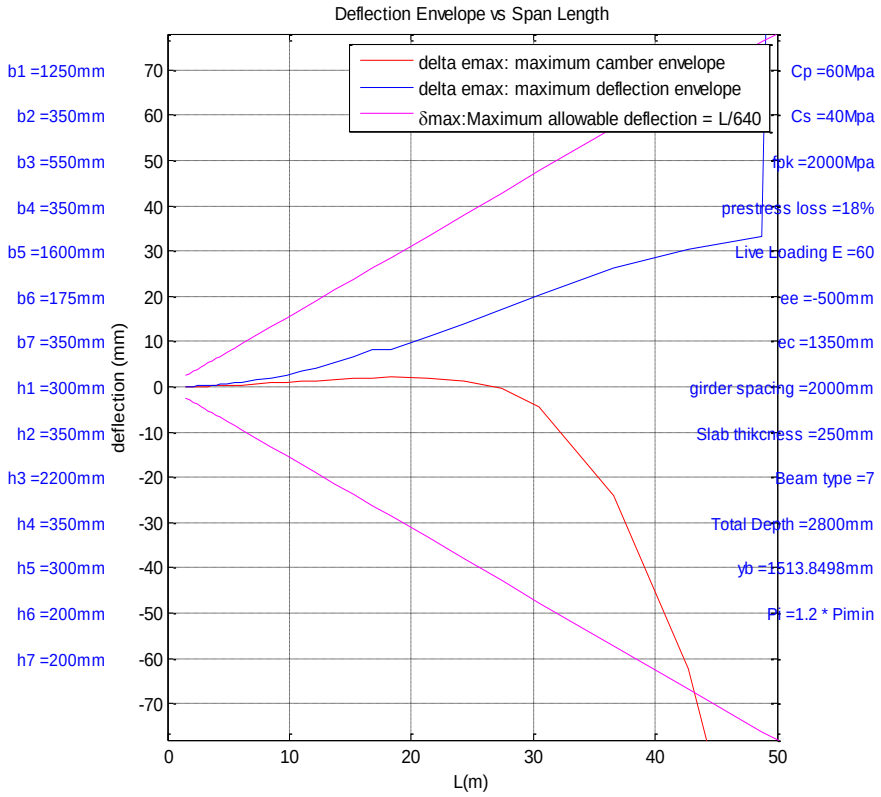

Deflection Envelope vs Span Length
delta emax: maximum camber envelope
delta emax: maximum deflection envelope
δmax:Maximum allowable deflection = L/640
b1 =712mm
b2 =254mm
b3 =204mm
b4 =102mm
b5 =1070mm
b6 =331mm
b7 =102mm
h1 =203mm
h2 =254mm
h3 =1490mm
h4 =102mm
h5 =127mm
h6 =76mm
h7 =76mm
Cp =50Mpa
Cs =30Mpa
fpk =1500Mpa
prestress loss =19%
Live Loading E =80
ee =-300mm
ec =700mm
girder spacing =2000mm
Slab thikcness =300mm
Beam type =6
Total Depth =1820mm
yb =920.2659mm
Pi =1.5 * Pimin
deflection (mm)
L(m)

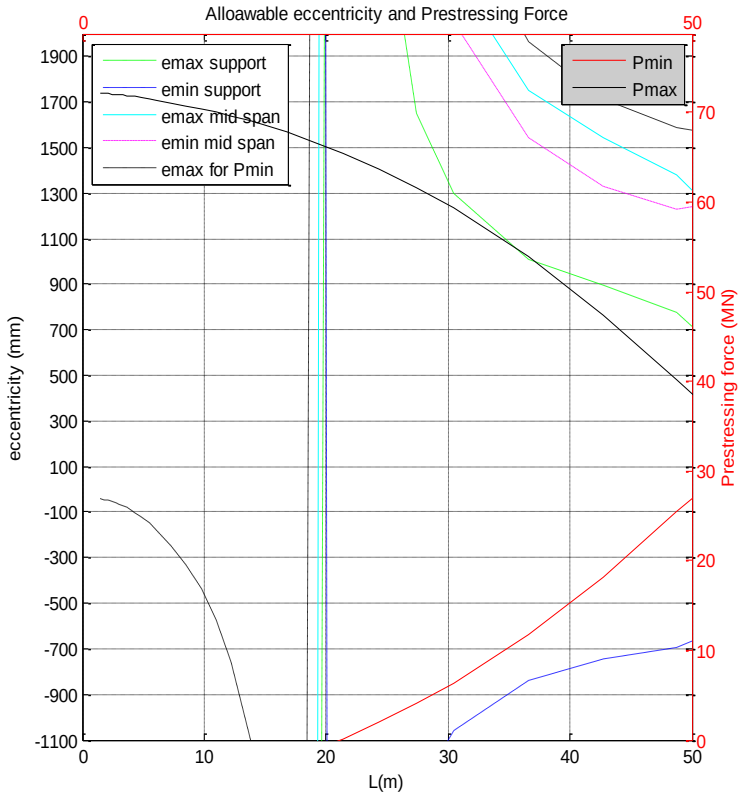

Alloawable eccentricity and Prestressing Force
emax support
emin support
emax mid span
emin mid span
emax for Pmin
Pmin
Pmax
eccentricity (mm)
Prestressing force (MN)
L(m)

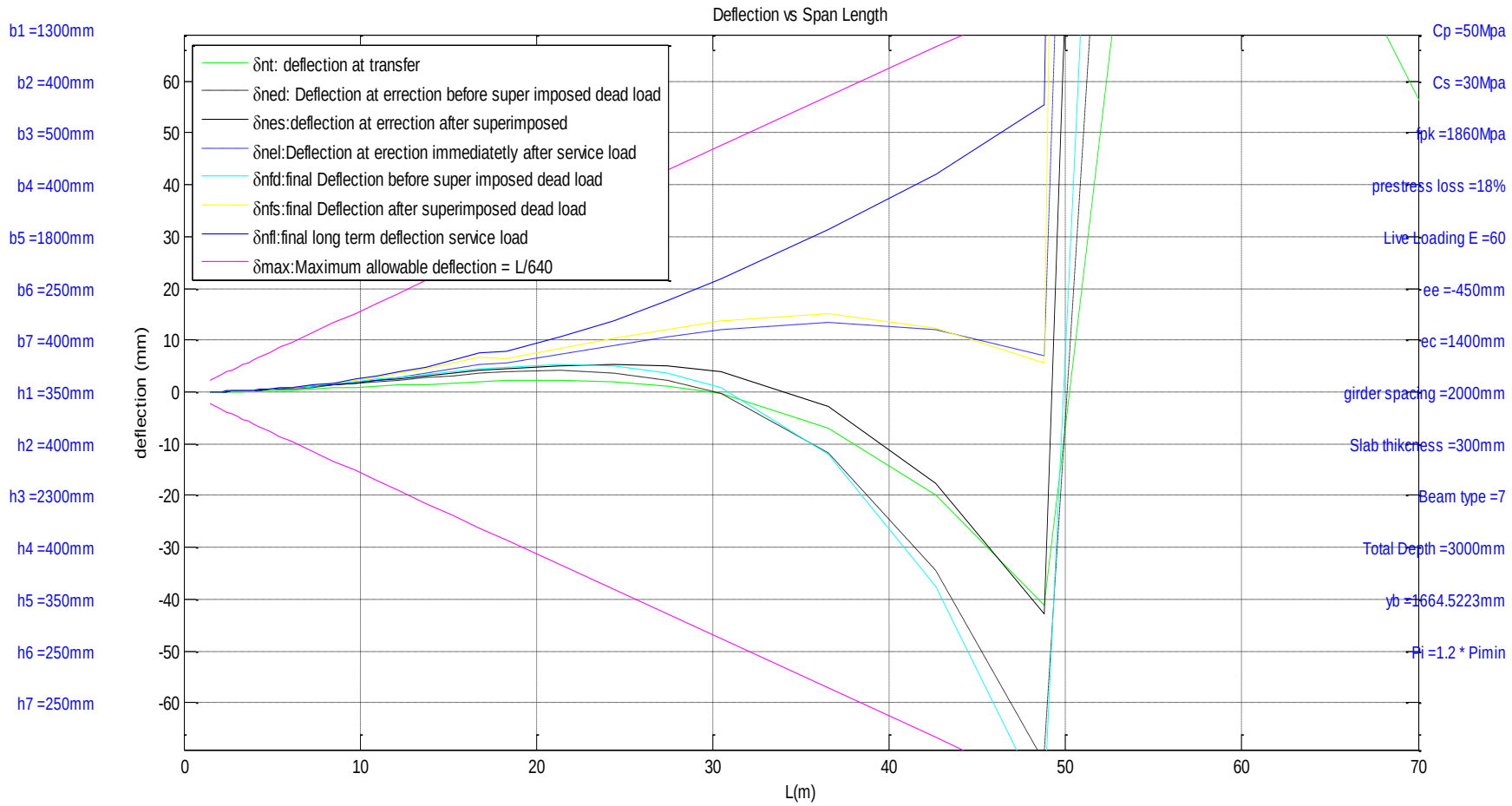
User Defined I beam

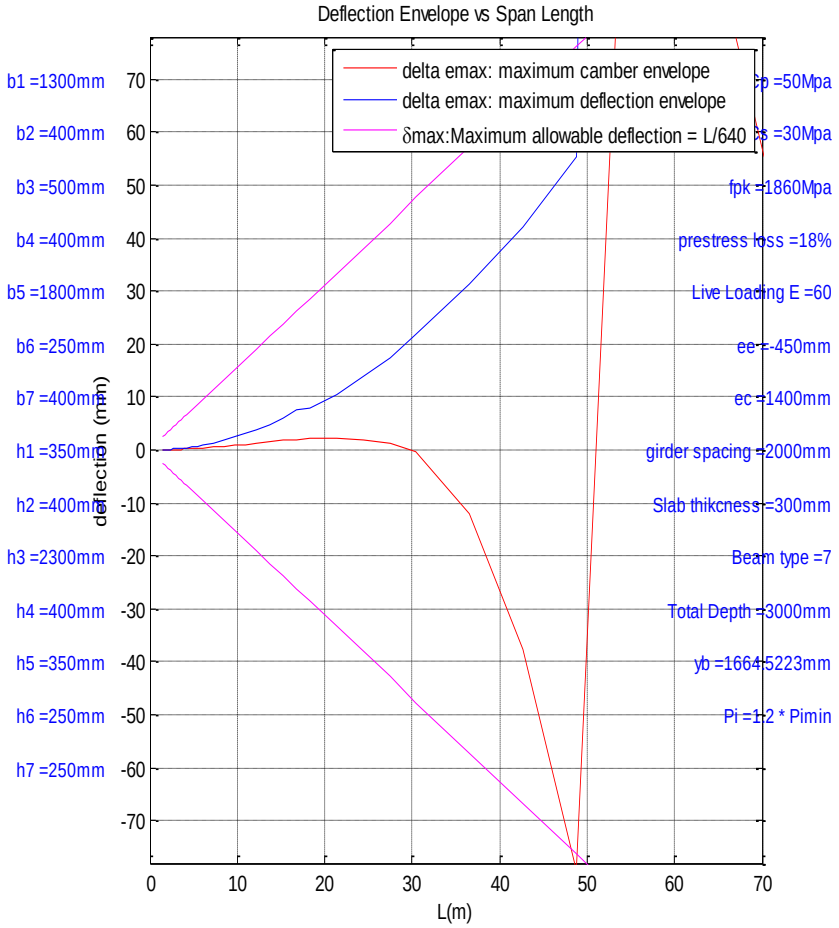


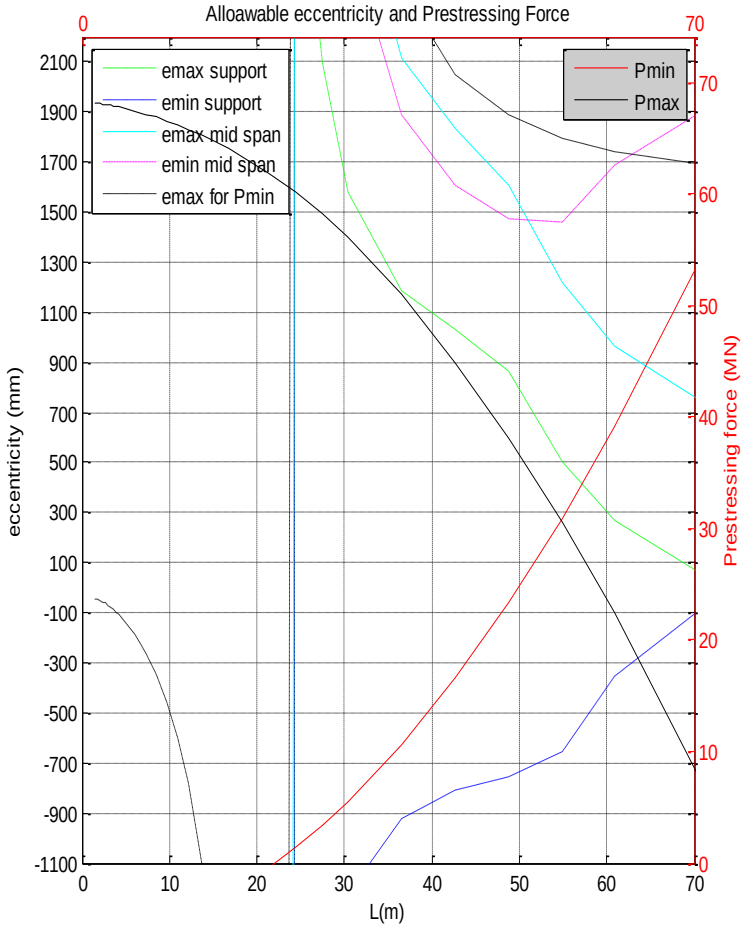

Deflection vs Span Length
δnt: deflection at transfer
δned: Deflection at errection before super imposed dead load
δnes:deflection at errection after superimposed
δnel:Deflection at erection immediatetly after service load
δnfd:final Deflection before super imposed dead load
δnfs:final Deflection after superimposed dead load
δnfl:final long term deflection service load
δmax:Maximum allowable deflection = L/640
deflection (mm)
L(m)
b1 =1250mm
b2 =350mm
b3 =550mm
b4 =350mm
b5 =1600mm
b6 =175mm
b7 =350mm
h1 =300mm
h2 =350mm
h3 =2200mm
h4 =350mm
h5 =300mm
h6 =200mm
h7 =200mm
Cp =60Mpa
Cs =40Mpa
fpk =2000Mpa
prestress loss =18%
Live Loading E =60
ee =-500mm
ec =1350mm
girder spacing =2000mm
Slab thikcness =250mm
Beam type =7
Total Depth =2800mm
yb =1513.8498mm
Pi =1.2 * Pimin

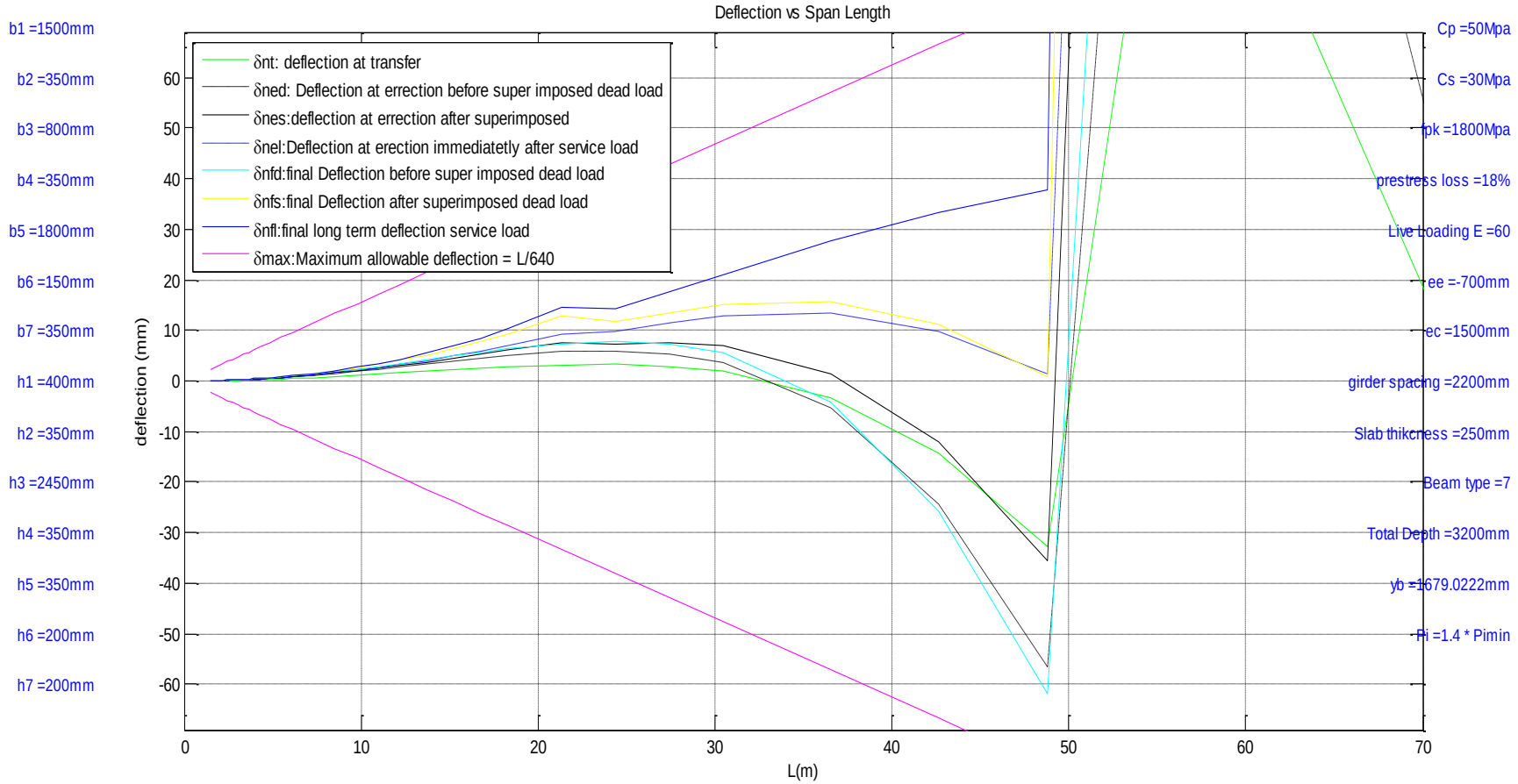

Deflection Envelope vs Span Length
delta emax: maximum camber envelope
delta emax: maximum deflection envelope
δmax:Maximum allowable deflection = L/640
b1 =1250mm
b2 =350mm
b3 =550mm
b4 =350mm
b5 =1600mm
b6 =175mm
b7 =350mm
h1 =300mm
h2 =350mm
h3 =2200mm
h4 =350mm
h5 =300mm
h6 =200mm
h7 =200mm
Cp =60Mpa
Cs =40Mpa
fpk =2000Mpa
prestress loss =18%
Live Loading E =60
ee =-500mm
ec =1350mm
girder spacing =2000mm
Slab thikcness =250mm
Beam type =7
Total Depth =2800mm
yb =1513.8498mm
Pi =1.2 * Pimin
deflection (mm)
L(m)

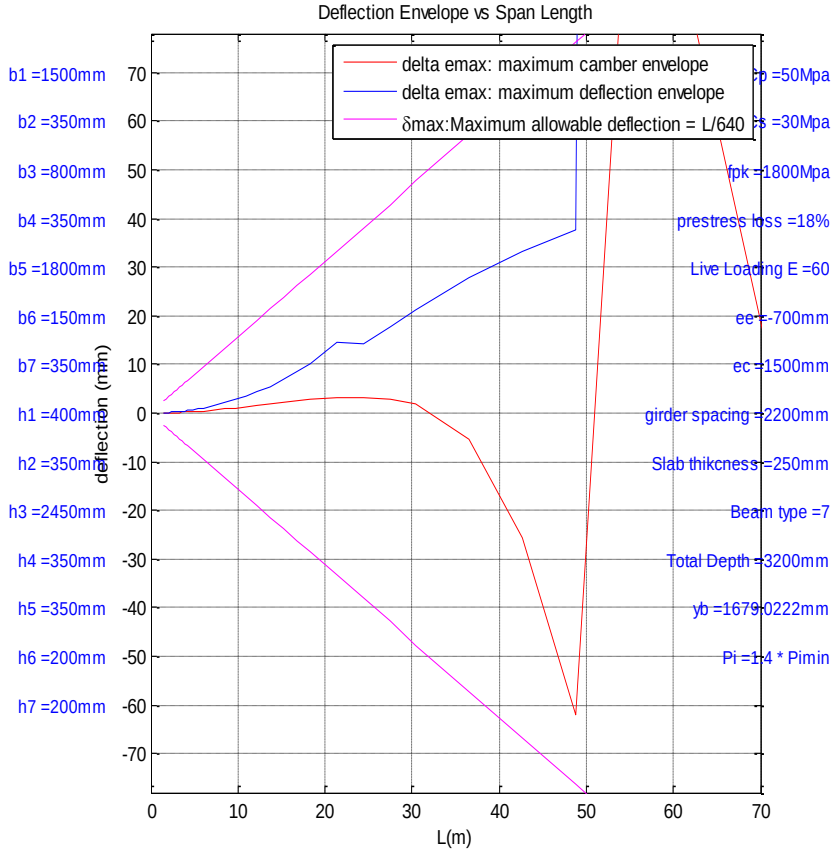

Alloawable eccentricity and Prestressing Force
emax support
emin support
emax mid span
emin mid span
emax for Pmin
Pmin
Pmax
eccentricity (mm)
Prestressing force (MN)
L(m)

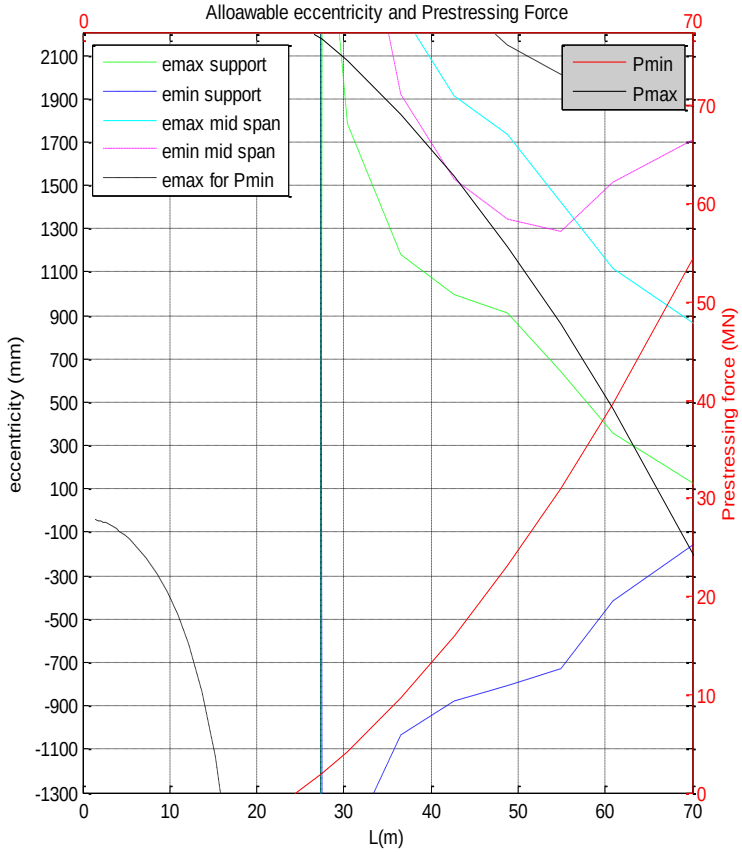

Deflection vs Span Length
δnt: deflection at transfer
δned: Deflection at erection before super imposed dead load
δnes:deflection at erection after superimposed
δnel:Deflection at erection immediatetly after service load
δnfd:final Deflection before super imposed dead load
δnfs:final Deflection after superimposed dead load
δnfl:final long term deflection service load
δmax:Maximum allowable deflection = L/640
deflection (mm)
L(m)
b1 =1300mm
b2 =400mm
b3 =500mm
b4 =400mm
b5 =1800mm
b6 =250mm
b7 =400mm
h1 =350mm
h2 =400mm
h3 =2300mm
h4 =400mm
h5 =350mm
h6 =250mm
h7 =250mm
Cp =50Mpa
Cs =30Mpa
fpk =1860Mpa
prestress loss =18%
Live Loading E =60
ee =-450mm
ec =1400mm
girder spacing =2000mm
Slab thikness =300mm
Beam type =7
Total Depth =3000mm
yb =1664.5223mm
Pi =1.2 * Pimin

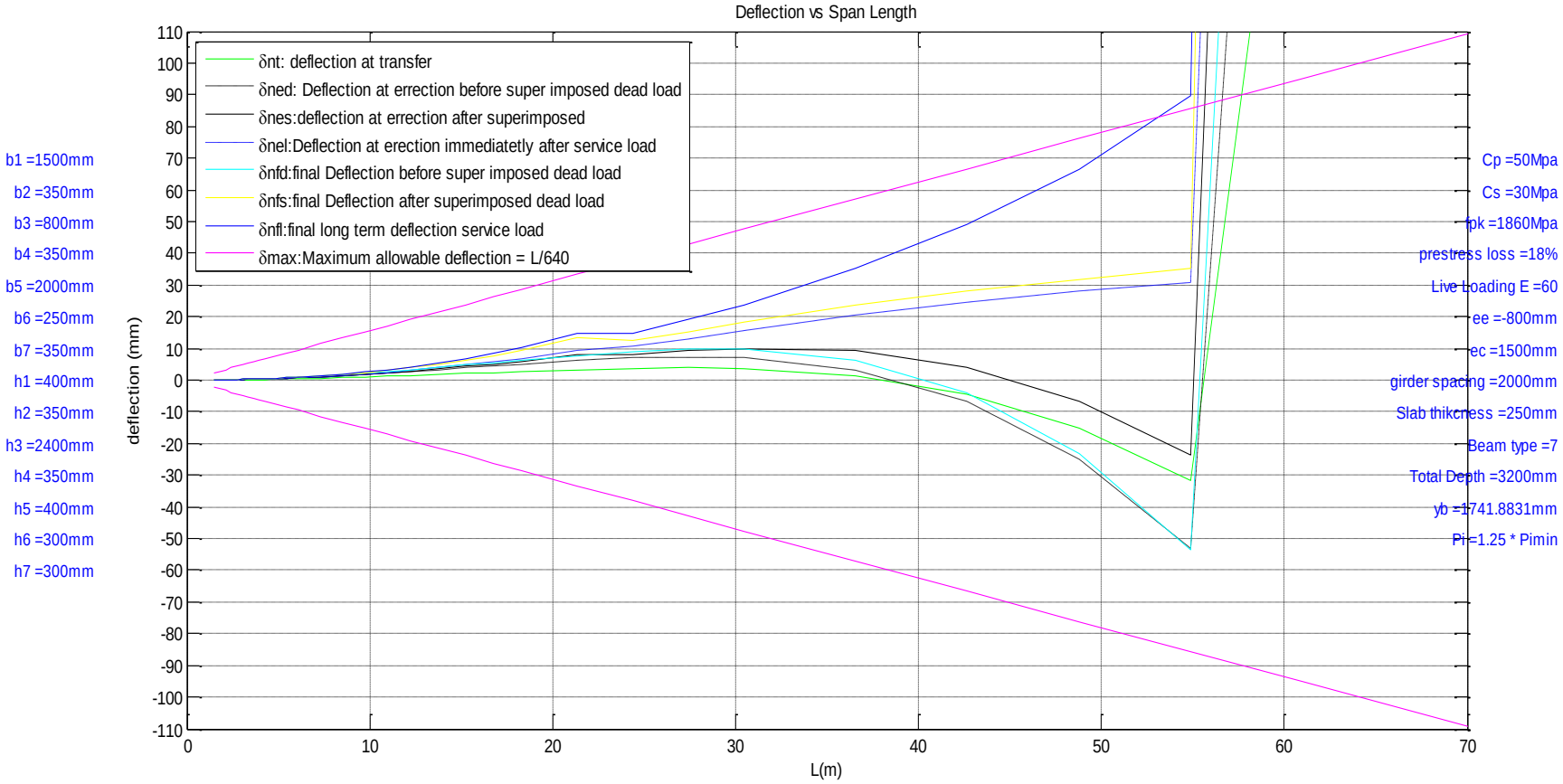

Deflection Envelope vs Span Length
delta emax: maximum camber envelope
delta emax: maximum deflection envelope
δmax:Maximum allowable deflection = L/640
b1 =1300mm
b2 =400mm
b3 =500mm
b4 =400mm
b5 =1800mm
b6 =250mm
b7 =400mm
h1 =350mm
h2 =400mm
h3 =2300mm
h4 =400mm
h5 =350mm
h6 =250mm
h7 =250mm
fb =50Mpa
fs =30Mpa
fpk =1860Mpa
prestress loss =18%
Live Loading E =60
ee =-450mm
ec =1400mm
girder spacing =2000mm
Slab thikcness =300mm
Beam type =7
Total Depth =3000mm
yb =1664.5223mm
Pi =1.2 * Pimin
deflection (mm)
L(m)


Alloawable eccentricity and Prestressing Force
emax support
emin support
emax mid span
emin mid span
emax for Pmin
Pmin
Pmax
eccentricity (mm)
Prestressing force (MN)
L(m)


Deflection vs Span Length
δnt: deflection at transfer
δned: Deflection at errection before super imposed dead load
δnes:deflection at errection after superimposed
δnel:Deflection at erection immediatetly after service load
δnfd:final Deflection before super imposed dead load
δnfs:final Deflection after superimposed dead load
δnfl:final long term deflection service load
δmax:Maximum allowable deflection = L/640
deflection (mm)
L(m)
b1 =1500mm
b2 =350mm
b3 =800mm
b4 =350mm
b5 =1800mm
b6 =150mm
b7 =350mm
h1 =400mm
h2 =350mm
h3 =2450mm
h4 =350mm
h5 =350mm
h6 =200mm
h7 =200mm
Cp =50Mpa
Cs =30Mpa
fpk =1800Mpa
prestress loss =18%
Live Loading E =60
ee =-700mm
ec =1500mm
girder spacing =2200mm
Slab thikcness =250mm
Beam type =7
Total Depth =3200mm
yb =1679.0222mm
Pi =1.4 * Pimin


Deflection Envelope vs Span Length
delta emax: maximum camber envelope
delta emax: maximum deflection envelope
δmax:Maximum allowable deflection = L/640
b1 =1500mm
b2 =350mm
b3 =800mm
b4 =350mm
b5 =1800mm
b6 =150mm
b7 =350mm
h1 =400mm
h2 =350mm
h3 =2450mm
h4 =350mm
h5 =350mm
h6 =200mm
h7 =200mm
fpk =1800Mpa
prestress loss =18%
Live Loading E =60
ec =1500mm
girder spacing =2200mm
Slab thikcness =250mm
Beam type =7
Total Depth =3200mm
Pi =1.4 * Pimin
deflection (mm)
L(m)


Alloawable eccentricity and Prestressing Force
emax support
emin support
emax mid span
emin mid span
emax for Pmin
Pmin
Pmax
eccentricity (mm)
Prestressing force (MN)
L(m)


Deflection vs Span Length
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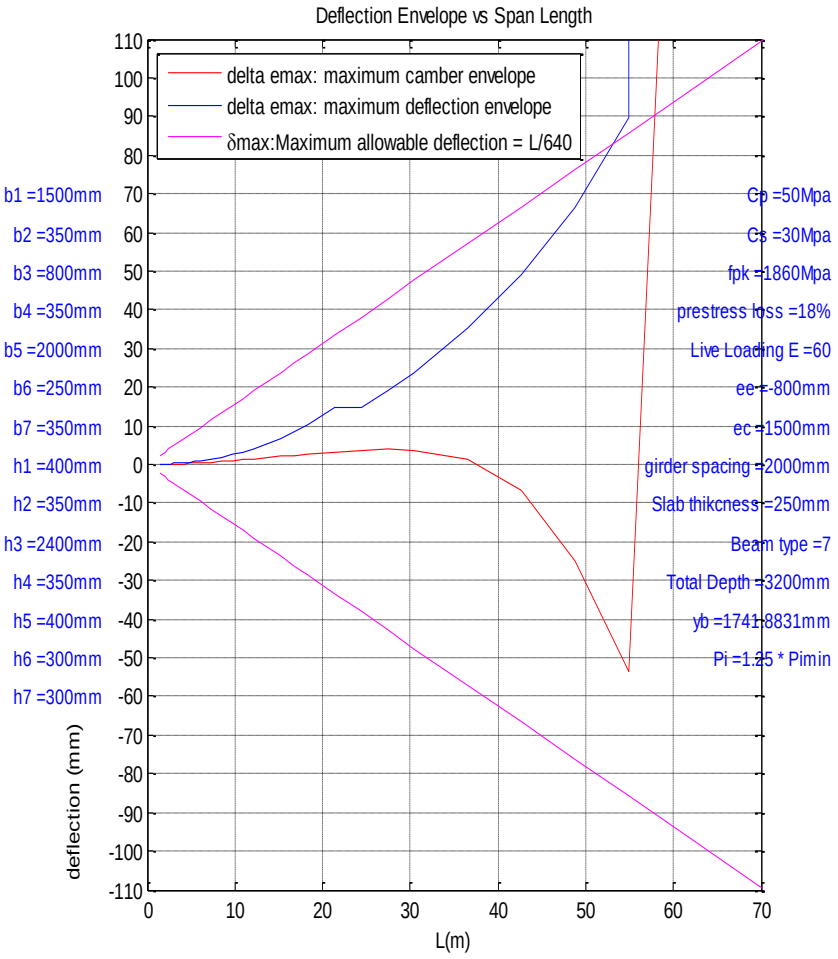
δnfl:final long term deflection service load
δmax:Maximum allowable deflection = L/640
deflection (mm)
L(m)
b1 =1500mm
b2 =350mm
b3 =800mm
b4 =350mm
b5 =2000mm
b6 =250mm
b7 =350mm
h1 =400mm
h2 =350mm
h3 =2400mm
h4 =350mm
h5 =400mm
h6 =300mm
h7 =300mm
Cp =50Mpa
Cs =30Mpa
fpk =1860Mpa
prestress loss =18%
Live Loading E =60
ee =-800mm
ec =1500mm
girder spacing =2000mm
Slab thikcness =250mm
Beam type =7
Total Depth =3200mm
yb =1741.8831mm
Pi =1.25 * Pimin


Deflection Envelope vs Span Length
delta emax: maximum camber envelope
delta emax: maximum deflection envelope
δmax:Maximum allowable deflection = L/640
b1 =1500mm
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h1 =400mm
h2 =350mm
h3 =2400mm
h4 =350mm
h5 =400mm
h6 =300mm
h7 =300mm
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fpk =1860Mpa
prestress loss =18%
Live Loading E =60
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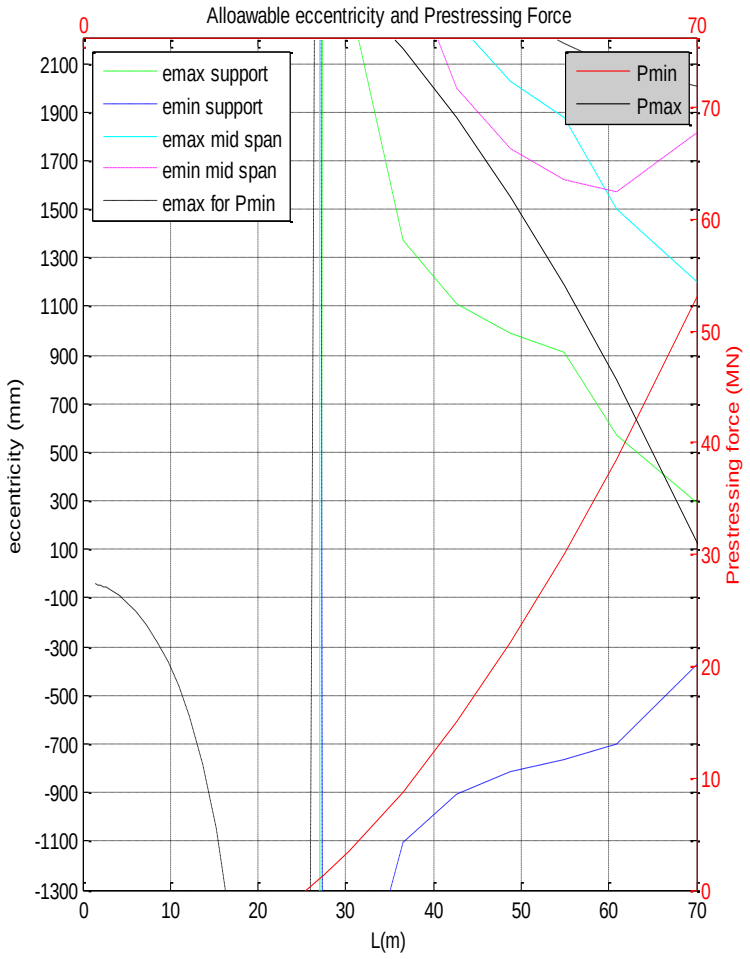
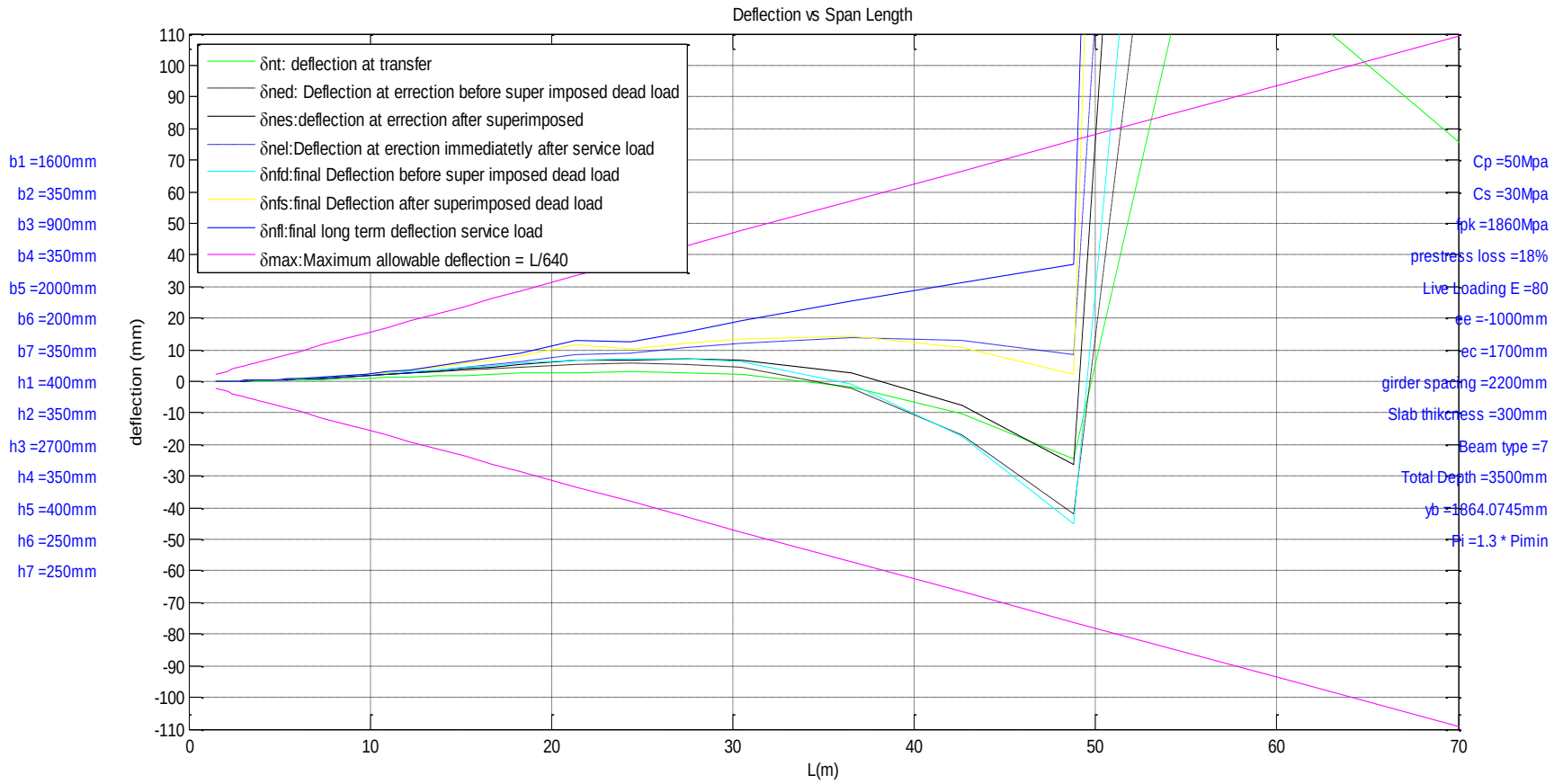
girder spacing =2000mm
Slab thikcness =250mm
Beam type =7
Total Depth =3200mm
yb =1741.8831mm
Pi =1.25 * Pimin
deflection (mm)
L(m)


Alloawable eccentricity and Prestressing Force
emax support
emin support
emax mid span
emin mid span
emax for Pmin
Pmin
Pmax
eccentricity (mm)
Prestressing force (MN)
L(m)


Deflection vs Span Length
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δmax:Maximum allowable deflection = L/640
deflection (mm)
L(m)
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b6 =200mm
b7 =350mm
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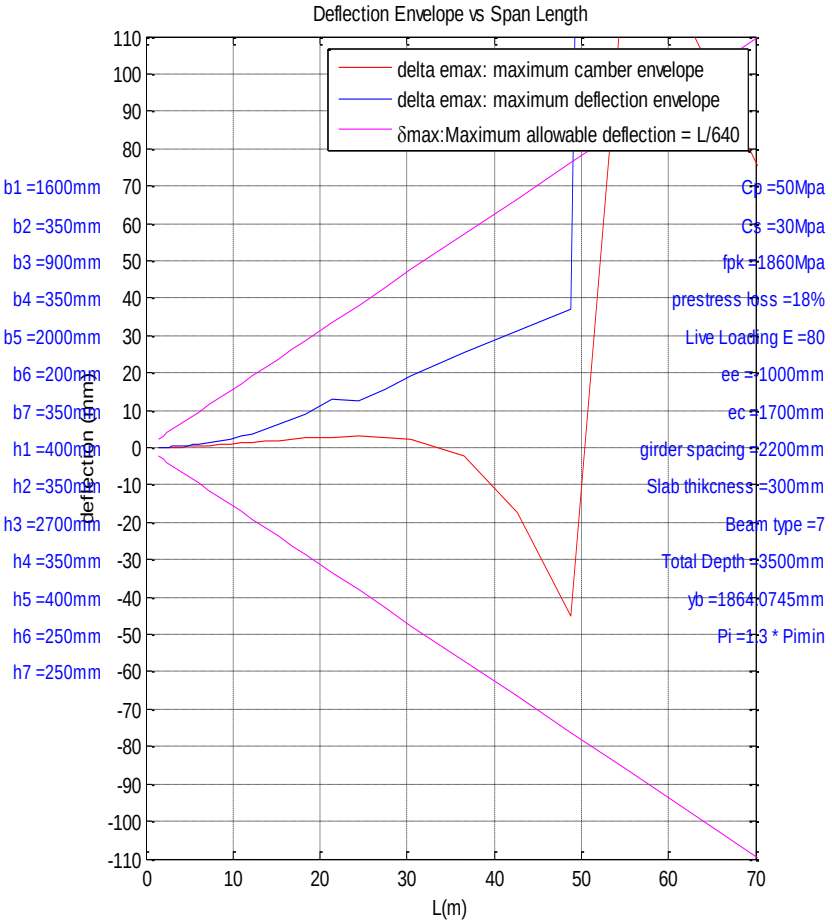
h2 =350mm
h3 =2700mm
h4 =350mm
h5 =400mm
h6 =250mm
h7 =250mm
Cp =50Mpa
Cs =30Mpa
fpk =1860Mpa
prestress loss =18%
Live Loading E =80
ee =-1000mm
ec =1700mm
girder spacing =2200mm
Slab thikcness =300mm
Beam type =7
Total Depth =3500mm
yb =1864.0745mm
Pi =1.3 * Pimin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δmax:Maximum allowable deflection = L/640
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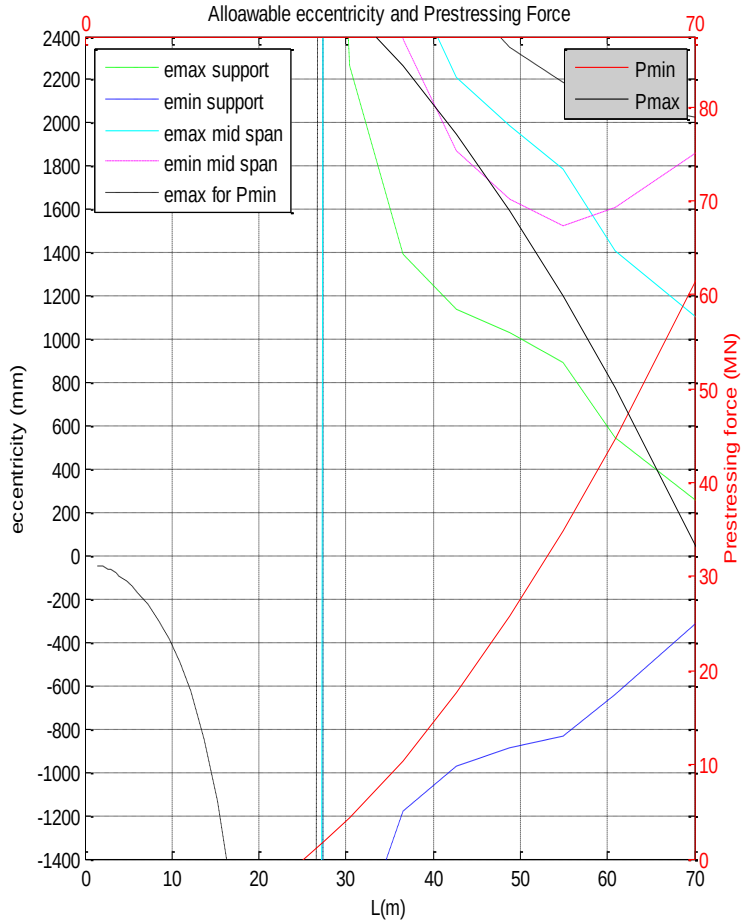
fpk =1860Mpa
prestress loss =18%
Live Loading E =80
ee =1000mm
ec =1700mm
girder spacing =2200mm
Slab thikcness =300mm
Beam type =7
Total Depth =3500mm
yb =1864.0745mm
Pi =1.3 * Pimin
deflection (mm)
L(m)


Alloawable eccentricity and Prestressing Force
emax support
emin support
emax mid span
emin mid span
emax for Pmin
Pmin
Pmax
eccentricity (mm)
Prestressing force (MN)
L(m)