THE EFFECTS OF A VARIABLE MODULUS OF ELASTICITY

ON PRESTRESS LOSSES IN BRIDGE GIRDERS

by

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ABSTRACT

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The purpose of this research is to evaluate a method proposed by Franz Dischinger (Dischinger, 1939) to consider the effects of a variable modulus of elasticity (MOE) of concrete over a period of time. Current guidelines suggested by the American Association of State Highways and Transportation Officials (AASHTO) assume a constant value for the MOE and base its calculation on the unit weight of concrete and the twenty eight day compressive strength of concrete. A few methods such as the ACI 209 Model Code (ACI 209, 1982) and CEB-FIP 1990 (CEB-FIP 1990, 1990) provide guidelines for the calculation of a variable modulus of elasticity. Dischinger's method varies the MOE through the use of a concrete creep function. The values of the time dependent MOE for each of the methods have been compared. Specifically this research focused on the effect Dischinger's method has on prestress losses in a prestressed concrete beam. This research also compared the losses between all three methods. The results showed that Dischinger's theory provides for higher values of modulus of elasticity and predicts higher losses. Over a period of time the prestress losses become constant. This is extremely useful since it can be used in conservative beam design.

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CHAPTER 1

INTRODUCTION

1.1 Purpose

The purpose of this research is to evaluate a method proposed by Franz Dischinger (Dischinger, 1939) to consider the effects of a variable modulus of elasticity (MOE) over a period of time. Current guidelines suggested by the American Association of State Highways and Transportation Officials (AASHTO) consider the value of the MOE to remain constant through the life of a structure. AASHTO's calculation of the MOE is based on the unit weight of concrete and the twenty eight day compressive strength of concrete. A few methods such as the ACI 209 Model Code (ACI 209, 1982) and CEB-FIP 1990 (CEB-FIP 1990, 1990) provide guidelines for the calculation of a variable modulus of elasticity. Dischinger's method varies the MOE through the use of the concrete creep function. The values of the time dependent MOE for CEB-FIP, ACI and Dischinger have been compared. Specifically this research focused on the effect Dischinger's method had on prestress losses in a prestressed concrete beam. This work determined whether the method proposed by Dischinger is a viable approach to accurately predict the prestress losses in a beam. Here a simply supported bridge was modeled and designed using PSTRS 14, a software currently used by the Texas Department of Transportation for prestressed beam design.

1.2 AASHTO Methods

The American Association of State Highway and Transportation Officials (AASHTO) regulate highway bridge design in the United States. Currently all bridge design in the state of Texas is performed in accordance with the AASHTO LRFD (Load Resistance Factor Design) 2007 specifications (AASHTO, 2007). In 2007 it became mandatory for bridge design to be based on LRFD methods. Prior to 2007 design was done in ordinance with the AASHTO Standard Specifications which were based primarily on the allowable stress design method. For the purpose of this research, the design of beams was done in accordance with the AASHTO LRFD 2007 specifications. The modulus of elasticity for concrete was taken from the AASHTO LRFD 2007 specifications Eq. 5.4.2.4-1 and is reproduced in Eq. 1.1 below.

$$E_c = 33{,}000 \times w_c^{1.5} \times \sqrt{f'c} \tag{1.1}$$

where:

E_c = modulus of elasticity (ksi)

w_c = unit weight of concrete (kcf)

f'_c = 28 day compressive strength of concrete (ksi)

Eq. 1.1 is valid for concrete with unit weights in the range of 0.090 kcf and 0.155 kcf. The Texas Department of Transportation (TxDOT) considers this equation valid for twenty eight day compressive strengths up to 8.5 ksi. For the purposes of this work, the unit weight of concrete was taken as 0.150 kcf. The modulus of elasticity for prestressing strands was taken as 28,500 ksi per AASHTO LRFD 5.4.4.2. The prestress loss was taken from AASHTO LRFD Eq. 5.9.5.1-1 and is reproduced in Eq. 1.2.

$$\Delta f_{pT} = \Delta f_{pES} + \Delta f_{pSR} + \Delta f_{pCR} + \Delta f_{pR2} \tag{1.2}$$

where:

Δf_{pT} = total loss (ksi)

Δf_{pES} = loss due to elastic shortening (ksi)

Δf_{pSR} = loss due to shrinkage (ksi)

Δf_{pCR} = loss due to creep of concrete (ksi)

Δf_{pR2} = loss due to relaxation of steel after transfer (ksi)

The loss due to elastic shortening of concrete is given by AASHTO Eq. 5.9.5.2.3a-1, and is given in Eq. 1.3 below.

$$\Delta f_{pES} = \frac{E_p}{E_{ci}} \times f_{cgp} \tag{1.3}$$

where:

E_p = modulus of elasticity of prestressing steel (ksi)

E_{ci} = modulus of elasticity of concrete at transfer (ksi)

f_{cgp} = sum of concrete stresses at the center of gravity of prestressing tendons due to the prestressing force at transfer and the self-weight of the member at the sections of maximum moment (ksi)

From Eq. 1.3 it can be seen that the elastic shortening loss is constant after the initial prestress application, however for the purposes of this work, it was considered to vary with time. The prestress loss due to shrinkage is given by AASHTO Eq. 5.9.5.4.2-1 and is given in Eq. 1.4.

$$\Delta f_{pSR} = (17.0 - 0.150 \times H) \tag{1.4}$$

where:

H = average annual ambient relative humidity (%)

The prestress loss due to creep is given by AASHTO Eq. 5.9.5.4.3-1 and is given in Eq. 1.5.

$$\Delta f_{pCR} = 12.0 \times f_{cgp} - 7.0 \times \Delta f_{cdp} \geq 0 \tag{1.5}$$

where:

f_{cgp} = concrete stress at center of gravity of prestressing steel at transfer (ksi)

Δf_{cdp} = change in concrete stress at center of gravity of prestressing steel due to permanent loads, with the exception of the load acting at the time the prestressing force is applied. (ksi)

The prestress loss due to the relaxation in the steel is given by AASHTO Eq. 5.9.5.5.4.4 and varies for low relaxation and stress relieved strands. For the purposes of this work, low relaxation strands were assumed. Equation 1.6 gives the equation for the prestress loss due to relaxation in the prestressed steel at transfer and Eq. 1.7 gives the equation for after transfer.

$$\Delta f_{pR1} = \frac{\log 24t}{40}\left[\frac{f_{pj}}{f_{py}} - 0.55\right] f_{pj} \tag{1.6}$$

where:

t = time estimated in days from stressing to transfer (days)

f_{pj} = initial stress in the tendon at the end of stressing (ksi)

f_{py} = specified yield strength of prestressing steel (ksi)

$$\Delta f_{pR2} = 0.3\left[20 - 0.4\Delta f_{pES} - 0.2(\Delta f_{pSR} + \Delta f_{pCR})\right] \tag{1.7}$$

where:

Δf_{pES} = loss due to elastic shortening (ksi)

Δf_{pSR} = loss due to shrinkage (ksi)

Δf_{pCR} = loss due to creep of concrete (ksi)

It can be seen from Eq. 1.7 that the steel relaxation loss is influenced by the creep loss. Both affect each other in that for creep there is an increase in deformations with time under a constant load. For steel relaxation there is a decrease in stress with time under a constant strain.

1.3 The Dischinger Method

Franz Dischinger (1887-1953) was a well known German civil and structural engineer who was responsible for the development of the modern cable-stayed bridge. He is known for his work in prestressed concrete and in 1939 published a theory called "Elastic and Plastic Distortions of Reinforced Concrete Structural Members and in Particular of Arched Bridges." In his work, Dischinger shows that the modulus of elasticity is a function of time since the creep of concrete is also a function of time. Dischinger applied this equation to moment, shear and stresses. Through laboratory experiments he proposed that the modulus of elasticity be taken as shown in Eq. 1.8.

$$E_{ot} = E_o \left(1 + \psi_t\right) \tag{1.8}$$

where:

E_{ot} = modified modulus of elasticity at time t

E_o = initial modulus of elasticity

ψ_t = creep coefficient and is given by AASHTO Eq. 5.4.2.3.2-1 and is given here in Eq. 1.9.

$$\psi(t,t_i) = 3.5k_c k_f \left(1.58 - \frac{H}{120}\right) t_i^{-0.118} \left(\frac{(t-t_i)^{0.6}}{10+(t-t_i)^{0.6}}\right) \quad (1.9)$$

where:

$$k_f = \frac{1}{0.67 + \frac{f'_c}{9}} \quad (1.10)$$

k_f = factor for the effect of concrete strength

f'c = 28 day compressive strength of concrete (ksi)

H = relative humidity (%)

k_c = factor for the effect of the volume to surface ratio of the component specified in AASHTO Eq. C5.4.2.3.2-1 and is given in Eq. 1.11.

$$k_c = \left[\frac{\frac{t}{26e^{0.36V/S}+t}}{\frac{t}{45+t}}\right]\left[\frac{1.80 + 1.77e^{-0.54(V/S)}}{2.587}\right] \quad (1.11)$$

where:

V/S = volume to surface ratio

t = time (days)

Dischinger's evaluation of the change in MOE with time was based on a creep coefficient determined from laboratory tests. Analysis of his data and work do not provide information as to exactly how he determined the creep coefficient. In order to verify the validity of the use of the AASHTO creep coefficient for this research, comparisons between the AASHTO value and the values shown by Dischinger were made. The comparisons showed that the values calculated by the AASHTO equation for creep closely resembled the values assumed by Dischinger. For the model bridge used for this project, the calculation and result for the values for the creep coefficient and values of MOE are given in chapter 3.

1.4 The ACI 209 Method

The ACI 209 Model Code calculates the value for a time dependent modulus of elasticity by calculating a time dependent value for the twenty eight day compressive strength. ACI 209 Eq. 20.23 gives the value for a time variable compressive strength and is given here in Eq. 1.12.

$$f_c(t) = \frac{t}{\alpha + \beta t} f_{c28} \tag{1.12}$$

where:

$f_c(t)$ = is the compressive strength at time t

t = time in days

f_{c28} = twenty eight day compressive strength

α, β = constants depending on curing and cement type

The values for α and β are given in ACI 209 table 20.2 and are reproduced here in Table 1.1. For this research type I cement and moist cured concrete was assumed. In standard practice when beams are manufactured, the concrete is normally steam cured to allow for quick results. For the purposes of this work, both types of curing, steam and moist, were checked to note the difference in the values for the MOE. The difference in MOE is not significant for either choice. The values using moist cured concrete were slightly higher and hence more conservative. ACI Eq. 20.25 gives the equation for the variable MOE and is given in Eq. 1.13.

Table 1.1 Coefficients for ACI Model Code

		Cement type		**Curing**	duration
		I	III		t_s [days]
Strength	a	4.0	2.3	moist	
		1.0	0.7	steam	
	β	0.85	0.92	moist	
		0.95	0.98	steam	

$$E_c(t) = 0.043\sqrt{w^3 f_c(t)} \tag{1.13}$$

where:

$E_c(t)$ = modulus of elasticity of concrete at age t days

w = unit weight of concrete

$f_c(t)$ = compressive strength at time t days (given in Eq. 1.12)

1.5 The CEB-FIP Method

The CEB-FIP Model Code 1990 was initially published in 1978 and since then has impacted national codes in many countries. ACI and other known codes have referenced the CEB-FIP in their publications. The code was developed in Europe and has been approved by the Euro-International Committee for Concrete (CEB). It is associated with the Eighth Congress of the International Federation for Prestressing (FIP). The CEB-FIP gives a method to calculate the time dependent modulus of elasticity by CEB-FIP Eq. 2.1-57 and is given here in Eq. 1.14.

$$E_{ci}(t) = \beta_E(t) E_{ci} \tag{1.14}$$

where:

$E_{ci}(t)$ = the modulus of elasticity at an age of t days

E_{ci} = the modulus of elasticity at an age of 28 days

$\beta_E(t)$ = a coefficient which depends on the age of concrete (t days) and is given by CEB-FIP Eq. 2.1-58 and is given here in Eq. 1.15.

$$\beta_E(t) = (\beta_{cc}(t))^{0.5} \tag{1.15}$$

where:

$\beta_{cc}(t)$ = a coefficient which depends on the age of concrete (t days) and is given by CEB-FIP Eq. 2.1-54 and is given here in Eq. 1.16.

$$\beta_{cc}(t) = exp\left\{s\left(1-\left(\frac{28}{t/t_1}\right)^{0.5}\right)\right\} \tag{1.16}$$

where:

t = age of concrete (days)

t_1 = 1 day

s = a coefficient which depends on the type of cement; s = 0.20 for rapid hardening high strength cements (RS), s = 0.25 for normal and rapid hardening cements (N and R) and s = 0.38 for slowly hardening cements (SL).

In this model case normal hardening cement was assumed for the calculation of the variable MOE by CEB-FIP. The CEB-FIP Model Code accounts for maturity of the concrete by allowing the time in days to be adjusted for temperature. In this work, this temperature effect on concrete maturity was not considered. The bridge location for this project (as described in chapter 3) was assumed to be in a stable environment with the temperature range per season to remain fairly constant.

CHAPTER 2

PSTRS14 SOFTWARE

2.1 Overview

The Texas Department of Transportation (TxDOT) developed and maintains the Prestressed Concrete Beam Design and Analysis Program (PSTRS14). PSTRS14 designs and analyzes standard TxDOT I, TxGirder, Box, U, Double-T, Slab, and non standard beams (user defined). These beams can have low-relaxation or stress-relieved strands. The program can design and analyze beams per AASHTO LRFD Specifications, AASHTO Standard Specifications or American Railway Engineering and Maintenance of way Association (AREA) Specifications. PSTRS14 includes a standard beam section library; however, the user can define unique and non-standard shapes and properties of beams. Furthermore PSTRS 14 assigns default values of material properties; however the user may also define material properties of beams, slabs, shear keys, and even non-standard composite regions.

PSTRS14 only analyzes and designs simply supported pretensioned concrete beams with draped or straight seven-wire patterns. Straight strands can be debonded; however, draped strands have to be fully bonded. PSTRS14 can simultaneously solve the required strand pattern (including number of required strands), the release and final required concrete strengths. The software allows for input and results in either English or Metric units.

2.2 History

Prior to the development of PSTRS14, TxDOT developed four programs for the analysis and design of prestressed concrete beams. The first program developed, PSTRS10, was written in the FORTRAN IV language for the IBM 360/XX series computers. PSTRS10 designed standard I-beams and non-standard beams with cross sections that were similar to standard I-beams. However, the design capabilities of PSTRS10 were limited. For example, only stress relieved strands were allowed and the strand pattern could be draped or straight with no debonding. PSTRS12 was similar to PSTRS10 and developed to analyze and check beam designs when inputted with the end and centerline strand pattern and other design parameters. The program would calculate the required concrete strength and moment capacity and compare it to those provided. PSTRS12 did have the option that low-relaxation strands could be used. Two other programs, DBOXSS and DBOXDS, were written to design pretensioned box beams with straight strands and pretensioned box beams with draped strands respectively.

At the time of publication, PSTRS14 was available in version 5.2.3 which was released on November 18, 2010. Version 5.2.3 designs and analyzes beams in accordance with the AASHTO Load and Resistance Factor Design (LRFD) Specifications (3rd Edition, 2004-2006, 4th Edition, 2007-2009, and 5th Edition, 2010), AASHTO Standard Specifications for Highway Bridges (15th Edition, 1994 Interim thru

17th Edition, 2002), or American Railway Engineering and Maintenance-of-Way Association (4th Edition, 2008-2009). For the purposes of this work, version 4.2 was used. This version designs and analyzes prestressed concrete beams based on AASHTO LRFD Specifications (4th Edition, 2007), AASHTO Standard Specifications for Highway Bridges (17th Edition, 2002), AASHTO Standard Specifications for Highway Bridges (15th Edition, 1994 Interim), American Standard Building Code Requirements for Reinforced Concrete (1989) or the American Railway Engineering Association Specifications (1988).

2.3 Standard Beam Design Input and Output

PSTRS14 is a MS-DOS based system in which a text file is inputted with material properties, loading and design considerations for the design of a prestressed concrete beam. The program allows for a maximum of up to 200 beams to be designed or analyzed at one time (or in one input file). PSTRS14 is versatile enough to allow the user to define the specification to which each beam must be designed, select units and specify properties. If values are not entered, the program assumes a set of defaults; however, the user must specify basic information about the beam for the design. This basic information includes the following: beam type (standard name or non-standard), span length (measured center to center of bearing), beam spacing, slab thickness, composite slab width, live load distribution factor, relative humidity, uniform dead load on composite section due to overlay and the uniform dead load on composite section excluding overlay.

PSTRS14 allows the user to select the type of output generated. The user can choose to view the results in either a short format or a long format. The short format is limited to a basic summary of the beam design. The short summary gives the following information: the number of strands and their eccentricity, draped or bonded strands and how much they are draped or bonded, design stresses, ultimate moment required, camber, dead load deflections due to the slab, overlay, other loads and the total deflection. For each beam that is to be designed the long format results include moment, shear, stress, and prestress loss tables. The long format also provides tables listing the beam, specification and strand data and tables listing material properties. PSTRS14 identifies properties taken as default if no value is entered by the user. The output files are created as files with extensions such as *.prn and *.lis. These files can be opened by Microsoft Word or any text file reader.

CHAPTER 3

MODEL BRIDGE DESCRIPTION

3.1 Bridge Location and Type

A model bridge was analyzed to evaluate the differences between Dischinger's modified modulus of elasticity, the ACI 290 Model for a variable MOE, the CEB-FIP model for a variable MOE and AASHTO methods. The bridge was assumed to be a typical bridge crossing located in Dallas, Texas. The bridge was modeled as a simply supported 300'-0" long bridge with three spans of pre-stressed beams. As shown in Fig. 3.1, the bridge consisted of two 90'-0" spans and one 120'-0" span. The 90'-0" spans consisted of type Tx-40 girders and the 120'-0" span of type Tx-54 girders. Type Tx-40 and Tx-54 girders are standard girders used by the Texas Department of Transportation (TxDOT). TxDOT standard sheets showing the detail and properties of these girders are provided in Appendix A.

A 44'-0" roadway without a skew was modeled. A 1'-0" nominal face of rail was assumed, giving an overall width of bridge of 46'-0" as shown in Figure 3.2. Two 12'-0" travel lanes and a 10'-0" shoulder on each side were assumed. The bridge was modeled with an 8" cast in place concrete slab and a 3" haunch over the beams was also assumed. Six beams per span were used at a spacing of 8'-0" with a 3'-0' overhang. The cross section for spans 1 and 3 (Tx-40 Girder) is shown in Figure 3.3. The cross section

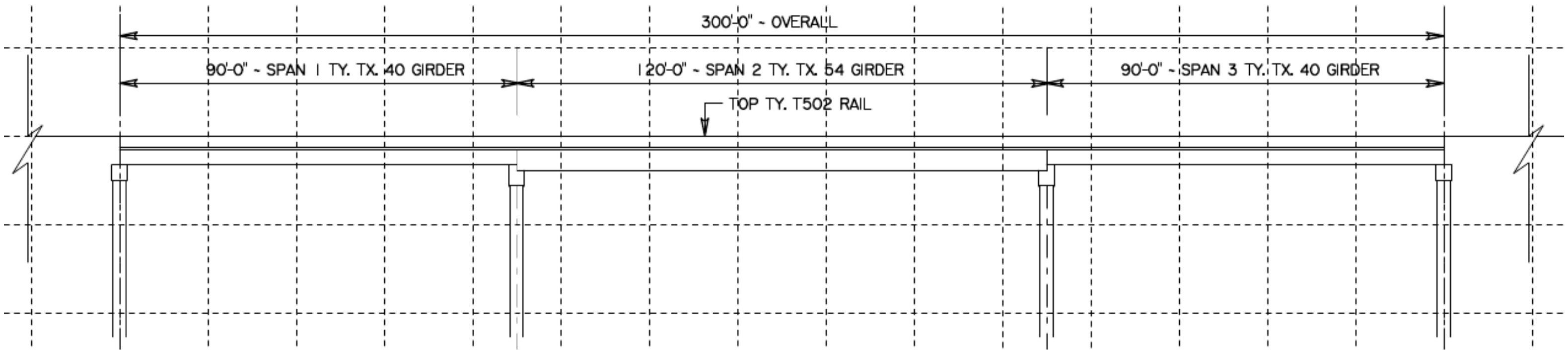

Figure 3.1 Profile View of Model Bridge

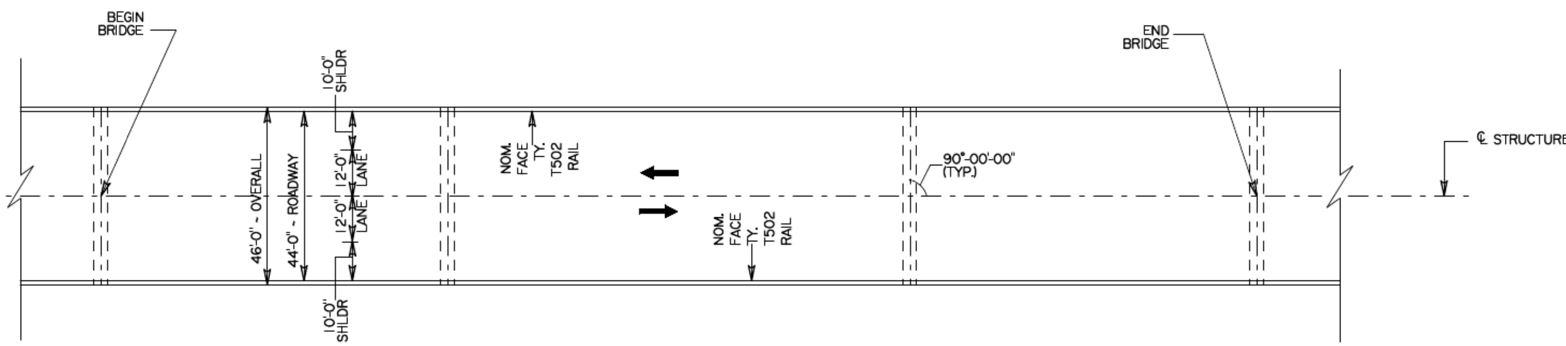

Figure 3.2 Plan View of Model Bridge

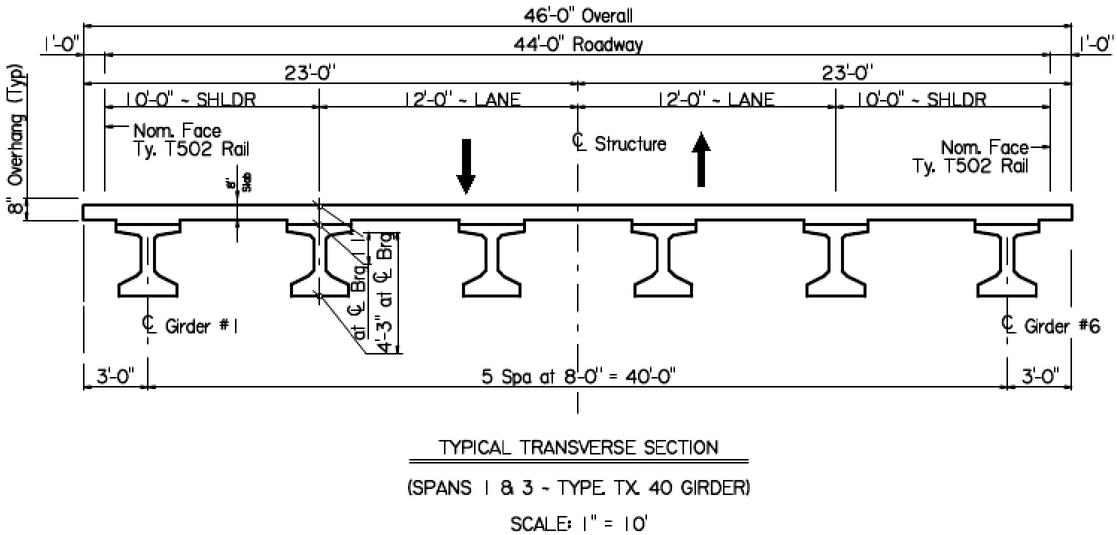

Figure 3.3 Cross Section for Spans 1 and 3.

for span 2 (Tx-54 Girder) is shown in Figure 3.4.

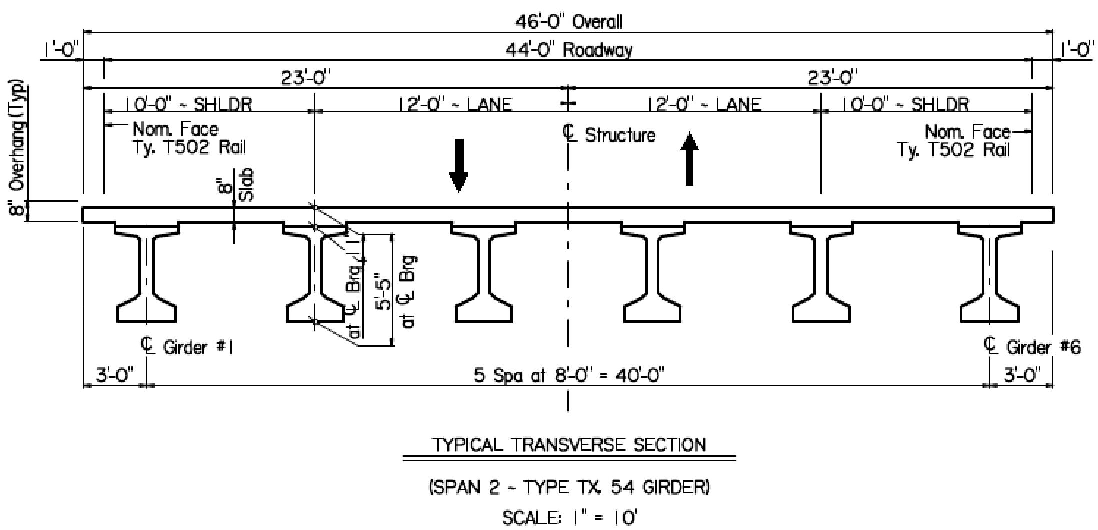

Figure 3.4 Cross Section for Span 2.

The difference in the two sections is the depth of the beam type used. A type T502 traffic rail was modeled for this bridge. The type T502 rail is a standard crash tested TxDOT rail with slots for drainage off the bridge deck. This rail is applicable for design speeds greater than 50 mph. TxDOT standard sheets showing the detail and properties of this rail are provided in Appendix A.

3.2 PSTRS14 Software Input

3.2.1 Beam Data and Material Properties

The standard Tx-40 Girder was input into PSTRS14 in two separate designs. The beam was designed as an exterior and interior girder. The exterior girder was designed separately due to a different tributary beam spacing compared to that of an interior girder. The tributary beam spacing was calculated by Eq. 3.1 (AASHTO, 2007).

$$S_t = \frac{S}{2} + OH \tag{3.1}$$

where:

S_t = effective beam spacing

S = interior centerline to centerline beam spacing

OH = width of overhang of exterior beam

Using the beam spacing, the live load distribution factors for moment and shear were calculated and input into PSTRS14. Equation 3.2 comes from AASHTO 4.6.2.2.2 and calculates the live load distribution factor for moment.

$$LLDF_m = 0.075 + \frac{S}{9.5}^{0.6} \times \frac{S}{L}^{0.2} \times \left(\frac{K_g}{12Lt_s^3}\right)^{0.1} \tag{3.2}$$

where:

$LLDF_m$ = live load distribution factor for moment

S = beam spacing (ft)

L = Length of beam (centerline bearing to centerline bearing) (ft)

t_s = slab thickness (in) and

K_g is defined by equation 3.3.

$$K_g = n(I + Ae_g^2) \tag{3.3}$$

where:

I = moment of inertia of the beam (in^4)

A = area of the beam (in^2)

e_g = distance between center of gravity of beam and deck (in)

and n is defined by Eq. 3.4.

$$n = \frac{E_{beam}}{E_{deck}} \tag{3.4}$$

where:

E_{beam} = modulus of elasticity of the beam

E_{deck} = modulus of elasticity of the deck

For this bridge, the modulus of the beam and deck were assumed to be the same and the value for n was taken as one. The live load distribution factor for shear is defined by AASHTO 4.6.2.2.3 and is given in Eq. 3.5.

$$LLDF_{shear} = 0.2 + \frac{S}{12} - \left(\frac{S}{3.5}\right)^2 \tag{3.5}$$

where:

$LLDF_{shear}$ = live load distribution factor for shear and

S = beam spacing (ft)

Table 3.1 summarizes the beam data for each of the girder types. The value for the percentage of relative humidity was taken as the average humidity encountered in Dallas, Texas in a year per AASHTO recommendations. AASHTO Fig. 5.4.2.3.3-1 gives the annual average ambient relative humidity in percent and is reproduced here in Fig. 3.5. The basic material properties for steel and concrete used for the model bridge are given in Table 3.2. Per TxDOT standards, TxDOT class H concrete was assumed with a minimum twenty eight day compressive strength of 4,000 psi. TxDOT also assumes a constant modulus of elasticity of 5,000 ksi, a unit weight of concrete of 150 pcf and prefers the use of ½” low relaxation prestress strands with a tensile strength of 270 ksi.

Table 3.1 Beam Data for Girders

Beam Data	Exterior Tx-40 Girder	Interior Tx-40 Girder	Exterior Tx-54 Girder	Interior Tx-54 Girder
Centerline to Centerline Bearing Length (ft)	88	88	118	118
Beam Spacing (ft)	7	8	7	8
Slab Thickness (in)	8	8	8	8
Relative Humidity (%)	65	65	65	65
Pre-stress Strand Size (in)	1/2	1/2	1/2	1/2
LLDF Moment	0.585	0.643	0.579	0.636
LLDF Shear	0.743	0.814	0.743	0.814

Figure 3.5 Annual Average Ambient Relative Humidity in Percent

Table 3.2 Material Properties

Modulus of Elasticity for Concrete (ksi)	5,000
Modulus of Elasticity for Pre-Stress Steel (ksi)	28,500
V/S (in)	6.0
α (Table 1.1)	4.0
β (Table 1.1)	0.85
w (pcf)	150
f'c (psi)	4,000
f_{pu} (ksi)	270
f_y (ksi)	60

The beams were re-designed and the prestress losses were compared for the modified modulus of elasticity's per Dischinger's equation, the ACI 209 Method and the CEB-FIP Method. It should be noted that the coefficient of creep calculated for the use in Dischinger's modified modulus of elasticity was assumed to be the same for the Tx-40 and Tx-54 girders since the volume to surface area ratio variation in Eq. 1.11 can be a maximum of 6 inches per AASHTO 5.4.2.3.2. For the Tx-40 girder the volume to surface area was calculated as 15.27 inches, and hence a maximum of 6 inches was used. Since the Tx-54 girder is larger in size, the maximum of 6 inches was used again.

The beams were analyzed for time intervals representing various stages of construction. Table 3.3 gives the time intervals used for the variation of the MOE. The time was measured when from the beam was cast (day 1). Per TxDOT standard specifications the beam must cure a minimum of 10 days. Once a beam is cured and it

Table 3.3 Time Intervals Used for MOE Analysis

t initial (days)	t (days)	Description
1	2	To initial stress transfer
1	10	Beam curing
1	14	Beam placement on jobsite
1	104	Casting of deck on beams
1	134	Casting of railing
1	254	Bridge open to traffic
1	365	1 year later
1	730	2 years later

passes the inspection processes, it typically shipped directly to the jobsite. Hence it was assumed that after curing it is shipped to the jobsite within 4 days. It is difficult to evaluate when construction loads/permanent dead loads (of the deck) are placed on the beam since each type of project and contractor is different. Each project has a different critical path hence the timing of when the dead loads are placed on the beam is difficult to predict. However, for this analysis a time frame of 3 months was assumed until the deck was cast and another 1 month until the rail is placed. This bridge was assumed to be open to traffic 4 months after the rail is cast. In order to further analyze the prestress losses over a period of time, the losses after 1 year, 2 years, 5 years and up to 50 years (assumed life of bridge) were analyzed. In the time interval from 5 years to 50 years, the losses were analyzed every 5 years. Table 3.4 gives the values for the calculated MOEs for the three models at the time intervals shown in table 3.3. These values were inputted into PSTRS 14 and each of the beams was analyzed for prestress losses. Table 3.5

Table 3.4 Calculated MOE Values during Construction Time Intervals

t initial	t (days)	kf	kc	phi (AASHTO)	f'c(t) (ACI Method)	E(ACI)	E (Dischinger)	E (CEB)
1	2	0.8973	0.1493	0.04	1403.51	2959.46	5221.32	3549.23
1	10	0.8973	0.1688	0.15	3200.00	4468.69	5748.71	4596.40
1	14	0.8973	0.1780	0.18	3522.01	4688.14	5922.74	4747.70
1	104	0.8973	0.3268	0.66	4502.16	5300.49	8289.44	5309.93
1	134	0.8973	0.3598	0.77	4546.23	5326.36	8830.34	5351.08
1	254	0.8973	0.4506	1.08	4620.28	5369.57	10396.46	5435.41
1	365	0.8973	0.5018	1.27	4645.98	5384.48	11338.74	5472.94
1	730	0.8973	0.5861	1.60	4675.74	5401.70	13019.96	5528.72

shows the values of the MOE from 5 years to 50 years at an interval of 5 years. Similar

Table 3.5 Calculated MOE Values from 5 to 50 Years

t initial	t (years)	kf	kc	phi (AASHTO)	f'c(t) (ACI Method)	E(ACI)	E (Dischinger)	E (CEB)
1	5	0.8973	0.6590	1.94	4693.78	5412.11	14675.54	5578.69
1	10	0.8973	0.6889	2.09	4699.82	5415.59	15469.81	5604.05
1	15	0.8973	0.6997	2.16	4701.84	5416.76	15791.88	5615.32
1	20	0.8973	0.7053	2.19	4702.85	5417.34	15971.48	5622.05
1	25	0.8973	0.7086	2.22	4703.46	5417.69	16087.84	5626.65
1	30	0.8973	0.7109	2.23	4703.86	5417.92	16170.20	5630.04
1	35	0.8973	0.7126	2.25	4704.15	5418.09	16231.99	5632.68
1	40	0.8973	0.7138	2.26	4704.37	5418.21	16280.31	5634.81
1	45	0.8973	0.7148	2.26	4704.53	5418.31	16319.30	5636.58
1	50	0.8973	0.7155	2.27	4704.67	5418.39	16351.51	5638.07

to the values for the MOE shown in table 3.4, the values in table 3.5 were also input into PSTRS 14 and the prestress losses were determined.

3.2.2 Loading

The dead loading due to overlay and railing was inputted into PSTRS 14. All other loads include the dead load of the slab and any live load. This was automatically calculated by PSTRS 14 based on 2007 AASHTO LRFD specifications. For this study a 2 inch overlay thickness was assumed. TxDOT calculates this uniform dead load due to overlay as 0.025 klf. To determine the load per beam, this dead load was multiplied by the beam spacing as shown in Eq. 3.6.

$$DL_{overlay} = DL \times S \tag{3.6}$$

where:

$DL_{overlay}$ = uniform dead load due to overlay per beam

DL = uniform dead load due to overlay (klf)

S = beam spacing (varies for interior and exterior girders) (ft)

For exterior beams the tributary beam spacing was used to calculate the dead load. In the case of this model bridge, there are no sidewalks or temporary railing. The only other dead load acting on the beams was of the Type T502 rail. From the TxDOT bridge railing standard given in Appendix A, the uniform dead load for the railing was given as 0.313 klf. Considering railing on both sides of the structure, this uniform dead load was multiplied by two. To find the dead load per beam, the total uniform dead load from both rails was divided by the number of beams in the span. In the model bridge the number of beams in the 90’-0” span and the 120’-0” span is the same. Hence the dead loads are same for both the Tx-40 and the Tx-54 girders. Table 3.6 gives the values inputted for the superimposed dead loads.

Table 3.6 Uniform Dead Load

Load	Exterior Girder	Interior Girder
Uniform Dead Load Due to Overlay	0.1750	0.2000
Uniform Dead Load Due to Railing	0.1043	0.1043

Since the bridge was designed for 2007 AASHTO LRFD specifications the live load was for HL-93. A default value for the live load impact factor of 1.33 was assumed by the software for this loading. Fig. 3.6 depicts the input file created for the initial design where a constant MOE was assumed. Fig. 3.7 depicts a sample input file (104 days) created using the modified modulus of elasticity as proposed by Dischinger. Appendix B gives the detail output generated by PSTRS 14 (long format) for each of the beams assuming a constant MOE. A sample of the detail output generated by PSTRS 14 (long format) for each of the beams using Dischinger’s modified modulus of elasticity; the ACI 209 and the CEB-FIP Method are given in Appendixes C, D and E respectively.

```
NUM1                                                        BPS
02/16/2009
90' & 120' SPANS,TY. TX40 & TX. 54 BMS , DESIGN BY AASHTO LRFD
$ 90' SPAN
PROB 1
SPEC     5
UNIT     E    E
MAT1      5000.
OUTP    LF
$ EXTERIOR TYPE TX 40 BEAM (SPANS 1 & 3)
STRD  1/2
LLDF      0.743
BEAM 1&3  EXT  TX40 88.0      7.0  8.0  7.0  0.585   65      .1750
.1043
$ INTERIOR TYPE TX 40 BEAM (SPANS 1 & 3)
STRD  1/2
LLDF      0.814
BEAM 1&3  INT  TX40 88.0      8.0  8.0  8.0  0.643   65      .2000
.1043
$ 120' SPAN
PROB 2
SPEC     5
UNIT     E    E
MAT1      5000.
OUTP    LF
$ EXTERIOR TYPE TX 54 BEAM (SPANS 1 & 3)
STRD  1/2
LLDF      0.743
BEAM 2    EXT  TX54 118.0     7.0  8.0  7.0  0.579   65      .1750
.1043
$ INTERIOR TYPE TX 54 BEAM (SPANS 1 & 3)
STRD  1/2
LLDF      0.814
BEAM 2    INT  TX54 118.0     8.0  8.0  8.0  0.636   65      .2000
.1043
```

Figure 3.6 PSTRS 14 Input File for Constant MOE

```
NUM1                                                     BPS
05/1/2011
90' & 120' SPANS,TY. TX40 & TX. 54 BMS , DESIGN AASHTO LRFD
DISCHINGER 104 DAYS
$ 90' SPAN
PROB 1
SPEC     5
UNIT     E    E
MAT1      8289.4
OUTP    LF
$ EXTERIOR TYPE TX 40 BEAM (SPANS 1 & 3)
STRD  1/2
LLDF      0.743
BEAM 1&3  EXT  TX40 88.0      7.0  8.0  7.0  0.585   65     .1750
.1043
$ INTERIOR TYPE TX 40 BEAM (SPANS 1 & 3)
STRD  1/2
LLDF      0.814
BEAM 1&3  INT  TX40 88.0      8.0  8.0  8.0  0.643   65     .2000
.1043
$ 120' SPAN
PROB 2
SPEC     5
UNIT     E    E
MAT1      8289.4
OUTP    LF
$ EXTERIOR TYPE TX 54 BEAM (SPANS 1 & 3)
STRD  1/2
LLDF      0.743
BEAM 2    EXT  TX54 118.0     7.0  8.0  7.0  0.579   65     .1750
.1043
$ INTERIOR TYPE TX 54 BEAM (SPANS 1 & 3)
STRD  1/2
LLDF      0.814
BEAM 2    INT  TX54 118.0     8.0  8.0  8.0  0.636   65     .2000
.1043
```

Figure 3.7 PSTRS 14 Sample Input File for Dischinger Method (104 days)

CHAPTER 4

ANALYSIS AND RESULTS

4.1 Variations in the Modulus of Elasticity

As discussed in chapter 3, the modulus of elasticity was varied with time for the three different methods studied in this work. A plot of table 3.4 showing the variation of the MOE for the Dischinger, ACI 209 and CEB-FIP Methods is given in Fig. 4.1. This

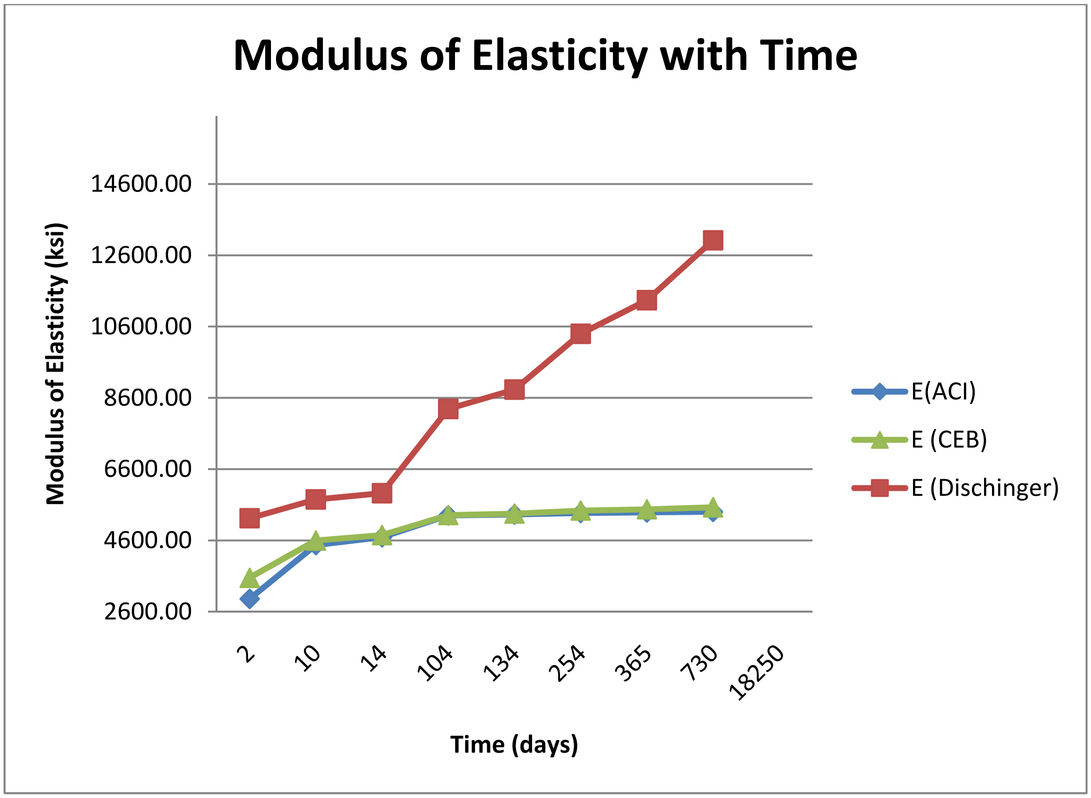

Figure 4.1 Variation of MOE for Construction Time Intervals

figure represents the construction period of the model bridge. It can be seen that the values of MOE for the ACI 209 and the CEB-FIP 1990 Model are in close agreement with one another. The values predicted by ACI and CEB-FIP become constant after the time interval assumed for the casting of the deck (104 days). Furthermore, ACI and the CEB-FIP models reach a maximum MOE of 5,000 ksi, which is what is assumed to be the constant value for MOE by TxDOT. Hence the values of the MOE based upon the CEB-FIP and ACI Methods converge upon the values predicted by AASHTO. Dischinger's proposed method for the evaluation of the MOE shows a significantly higher value compared to ACI and CEB-FIP and continues to increase for the duration of the construction period. This significant difference is attributed to the nature of Dischinger's equation. Dischinger's equation is based on the creep coefficient. Dischinger assumes that the value of the MOE increases by a percentage equal to the creep coefficient. By examination of the equation for k_c, the volume to surface area ratio, it can be seen that it is proportional to time as well. Since the creep coefficient is dependent upon time and proportional to the value of k_c, the Dischinger equation also yields a value of the MOE proportional to this value. Similar to the CEB-FIP and the ACI 209 Methods, the Dischinger Method shows a significant increase in MOE after the casting of the deck. Table 3.5 showing the change in MOE from 5 to 50 years is plotted in Fig. 4.2. It can be seen that all models show that the MOE becomes constant after 10 years of service life of the beam. The values for ACI and CEB-FIP remain constant at approximately 5,000 ksi, while the values predicted by Dischinger increase slightly yet remain fairly close to approximately 16,000 ksi.

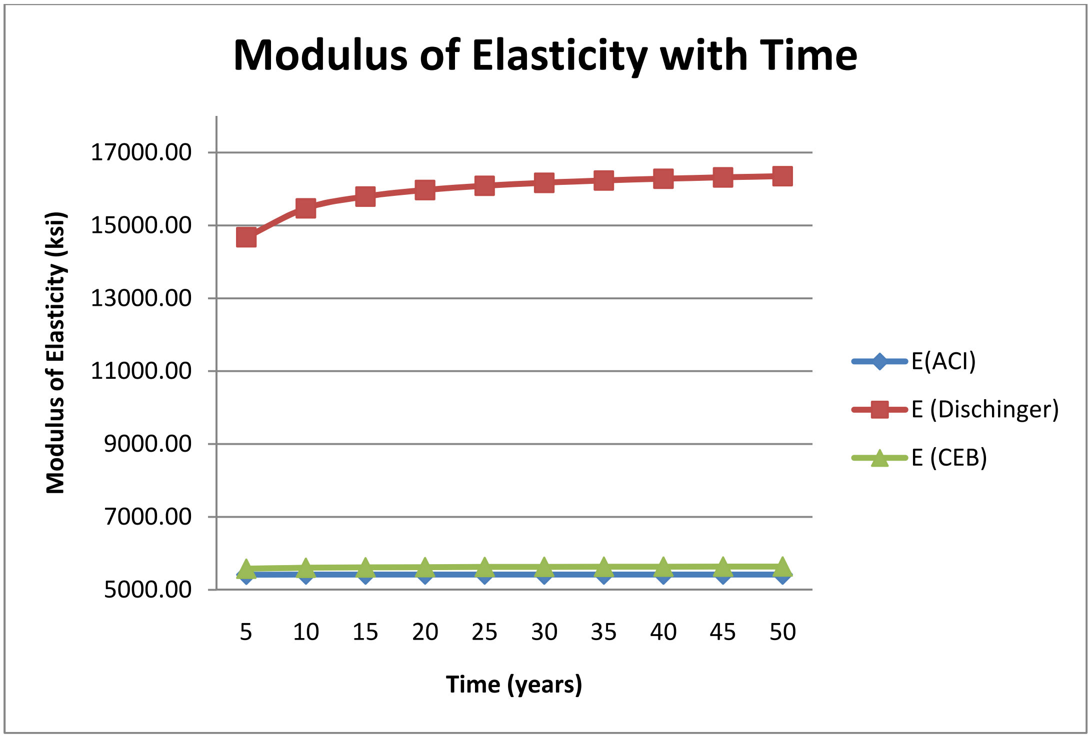

Figure 4.2 Variation of MOE from 5 to 50 Years

4.2 Short-Term Effects of a Variable MOE on Prestress Losses

The design of the Tx-40 and Tx-54 girders using PSTRS 14 for the various methods yielded a beam summary reports which presented the basic calculations derived from the design of the beams. This summary included tables for the strand design, design stresses, dead load deflections and values for the different types of prestress loss. The PSTRS 14 results also provided values for the percentage of losses at release and a final value. In this work the effect of a variable MOE was studied in elastic shortening, creep and steel relaxation losses. Since the relative humidity was taken to be constant the shrinkage losses were not evaluated. Here the effects of a variable MOE during the construction phase for the model bridge were evaluated.

4.2.1 Elastic Shortening Loss

The results obtained for the elastic shortening loss from the run of PSTRS 14 for the time frames given in table 3.3 were plotted by beam type. These results are given in Fig. 4.3 through Fig. 4.6. From Fig. 4.3 and Fig. 4.4, it can be seen that there is an increase in elastic shortening loss at 104 and 730 days (2 years) for the two exterior girders from the Dischinger Method. This can be attributed to the significant increase in the MOE and the number of pre-stressing strands necessary to meet design requirements. From the output of PSTRS 14 it can be seen that the stresses, strands and eccentricity of the strands all increase at these time frames. The increase in the number of strands causes an increase of the concrete fiber stress at the steel center of gravity at transfer, thus causing an increase in the elastic shortening loss. Table 4.1 depicts the change in strands for the Dischinger Method in the construction phase as the MOE changed. The CEB-FIP 1990 and ACI 209 Methods both are both in agreement with each other since the values for MOE are also approximately equal for both methods. The increases in elastic shortening loss from these methods are similar to the jumps from the Dischinger Method. The jumps are noticed when there is a significant increase in MOE which causes an increase in stress. It can be seen that the elastic shortening loss remains constant after 104 days when the value for MOE becomes constant in these two methods. Furthermore it can be seen that the values of the CEB-FIP and ACI models produce elastic shortening losses that are predicted by a constant value of MOE. From figures 4.5 and 4.6 it can be seen that the interior beams produce a jump in loss around 134 days and 254 days with the

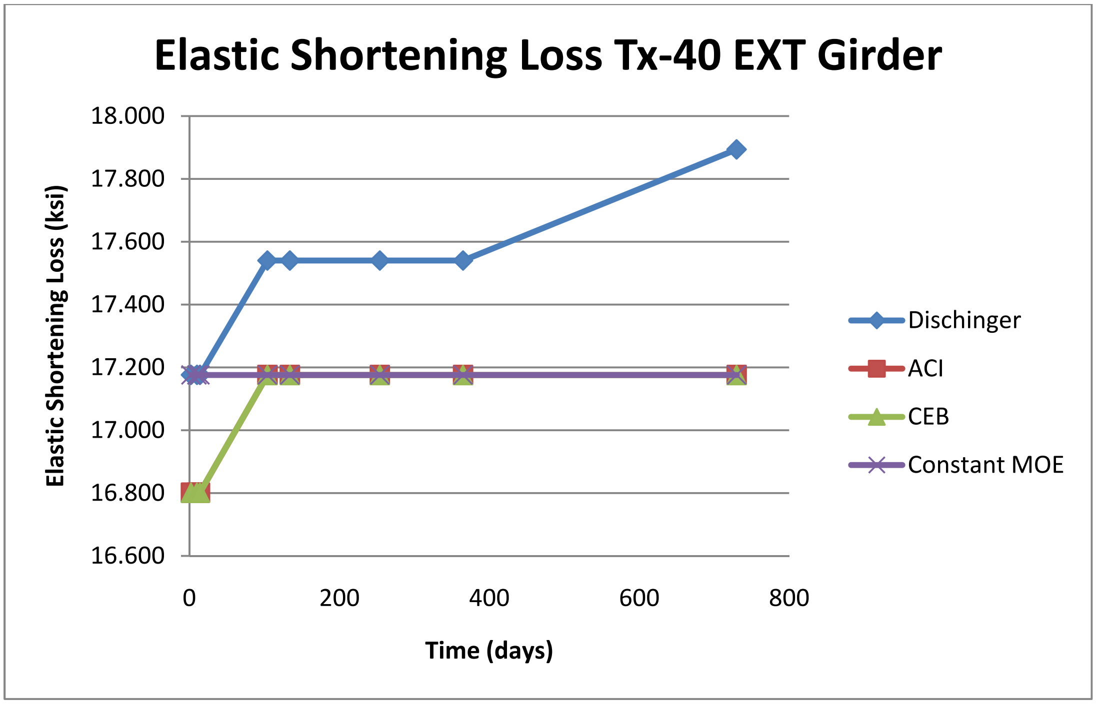

Figure 4.3 Elastic Shortening Loss over Time for Tx-40 Exterior Girder

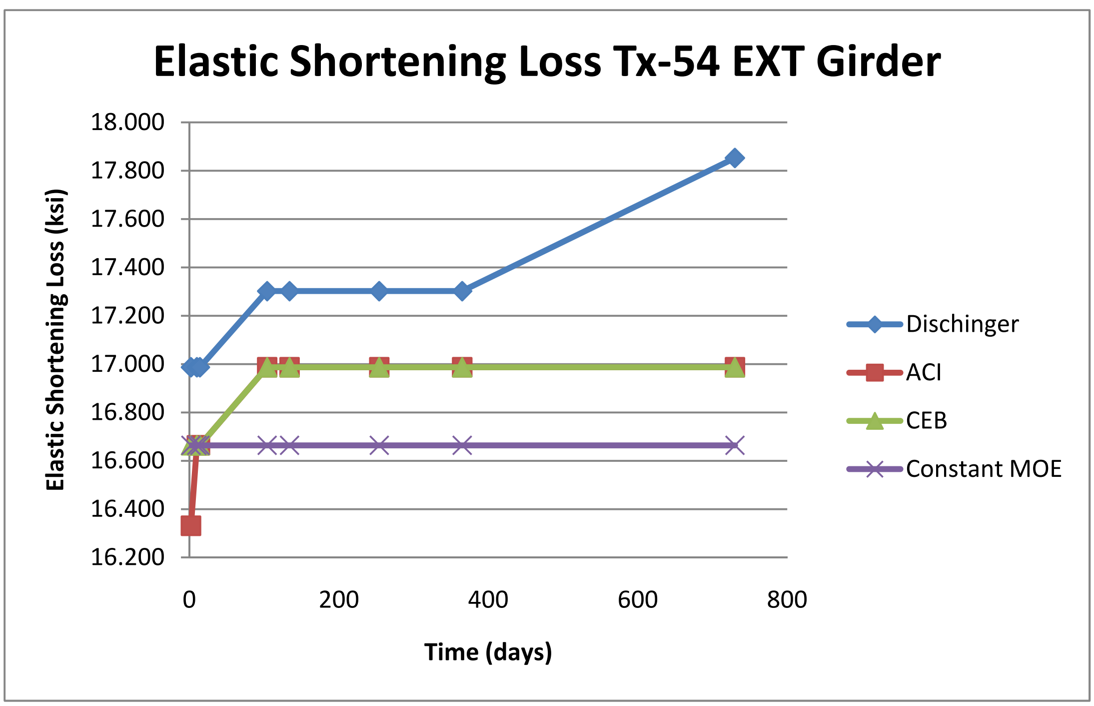

Figure 4.4 Elastic Shortening Loss over Time for Tx-54 Exterior Girder

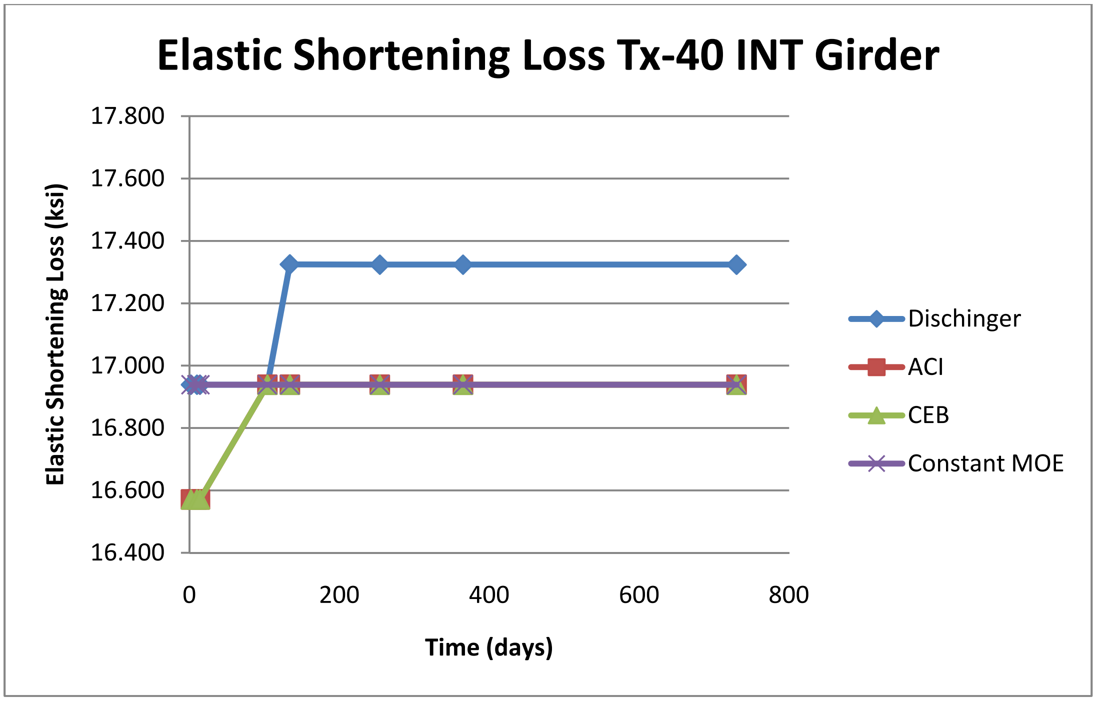

Figure 4.5 Elastic Shortening Loss over Time for Tx-40 Interior Girder

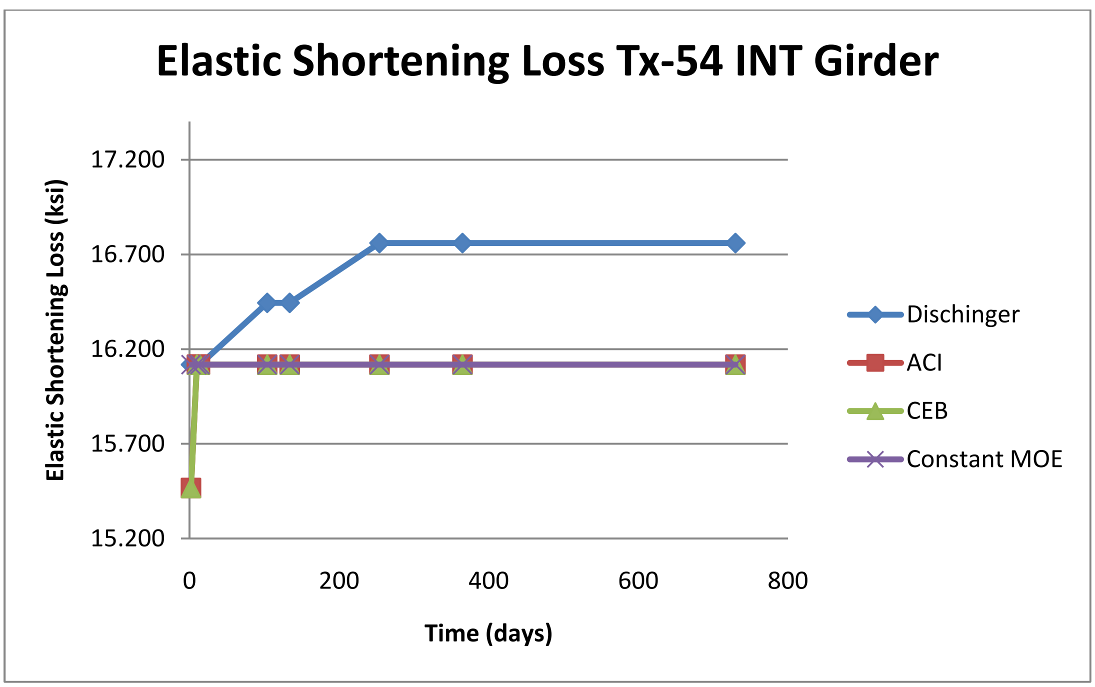

Figure 4.6 Elastic Shortening Loss over Time for Tx-54 Interior Girder

Table 4.1 Short Term Change in Strand Design for Dischinger Method

Beam Type	No. Strands Total	Eccentricity at centerline (in)	Eccentricity at end (in)	No. Draped Strands	Yb of Top Draped Strands	Time (days)
Tx-40 Exterior	44	13.33	8.24	8	36.5	2
Tx-40 Interior	44	13.33	8.24	8	36.5	
Tx-54 Exterior	60	17.61	10.94	10	50.5	
Tx-54 Interior	56	17.94	10.8	10	50.5	
Tx-40 Exterior	46	13.17	8.3	8	36.5	104
Tx-40 Interior	44	13.33	8.24	8	36.5	
Tx-54 Exterior	62	17.46	11.01	10	50.5	
Tx-54 Interior	56	17.94	10.8	10	50.5	
Tx-40 Exterior	46	13.17	8.3	8	36.5	134
Tx-40 Interior	46	13.17	8.3	8	36.5	
Tx-54 Exterior	62	17.46	11.01	10	50.5	
Tx-54 Interior	58	17.77	10.87	10	50.5	
Tx-40 Exterior	46	13.17	8.3	8	36.5	254
Tx-40 Interior	46	13.17	8.3	8	36.5	
Tx-54 Exterior	62	17.46	11.01	10	50.5	
Tx-54 Interior	60	17.61	10.94	10	50.5	
Tx-40 Exterior	48	13.02	8.35	8	36.5	730
Tx-40 Interior	46	13.17	8.3	8	36.5	
Tx-54 Exterior	64	17.26	10.14	12	50.5	
Tx-54 Interior	60	17.61	10.94	10	50.5	

Dischinger Method. The longer Tx-54 beam shows the jump at 254 days while the shorter Tx-40 beams show the jump in elastic shortening loss at 104 days. Again from the detail beam design report it was found that the stresses, strands and eccentricity of the strands increase in these time periods for the two types of beams. Contrary to what was seen with the exterior beams, the interior beams are more stable for elastic shortening losses. The ACI 209 and CEB-FIP Models show that the value of elastic shortening loss again remains constant for a constant MOE. Appendixes C, D and E provide the beam summary reports with strand and stress changes for these significant time periods for the Dischinger, ACI 209 and CEB-FIP 1990 Models respectively.

4.2.2 Creep Loss

The results obtained for the loss in creep from the run of PSTRS 14 for the time frames given in table 3.3 were also plotted by beam type. These results are given in Fig. 4.7 through Fig. 4.10. It can clearly be seen that the creep loss behavior with MOE and time is exactly similar to the behavior of elastic shortening loss. Hence the figures when compared by beam type are the same shape however the magnitude of the loss is different. This similarity is due to the fact that the creep loss is also dependent upon the concrete stress at center of gravity of the prestressing steel at transfer per Eq. 1.5. As described in section 4.2.1, the number of strands necessary to satisfy design requirements increases, causing an increase in this concrete stress. Similar behavior of interior and exterior beams was observed as was observed with the elastic shortening loss.

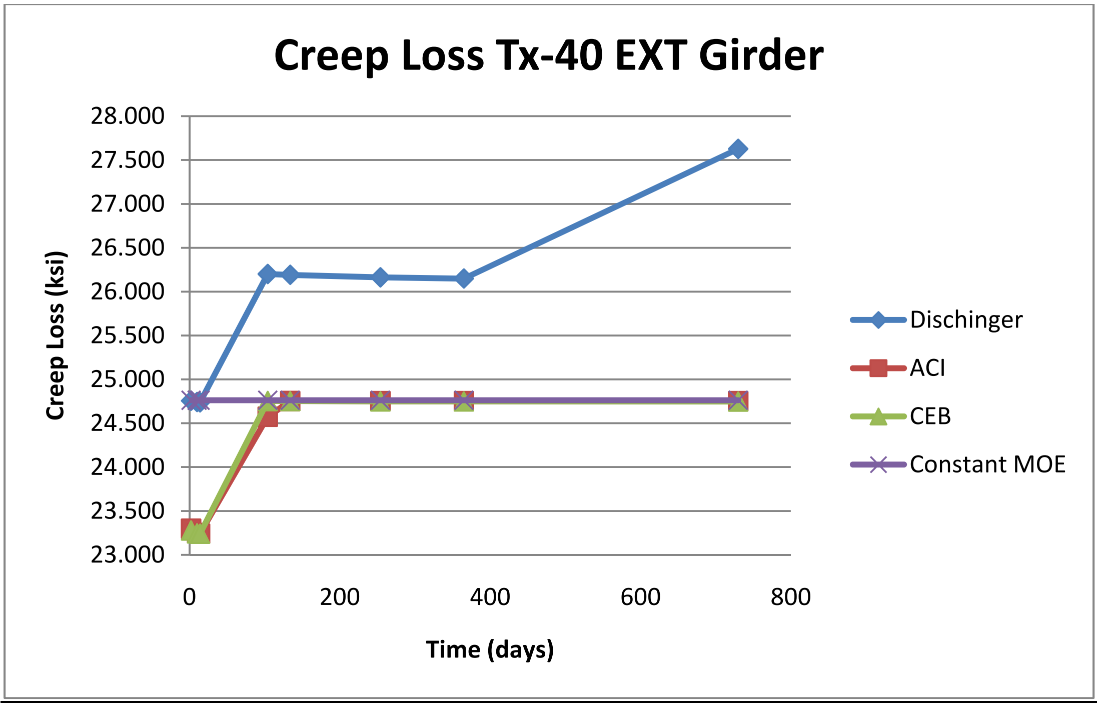

Figure 4.7 Creep Loss over Time for Tx-40 Exterior Girder

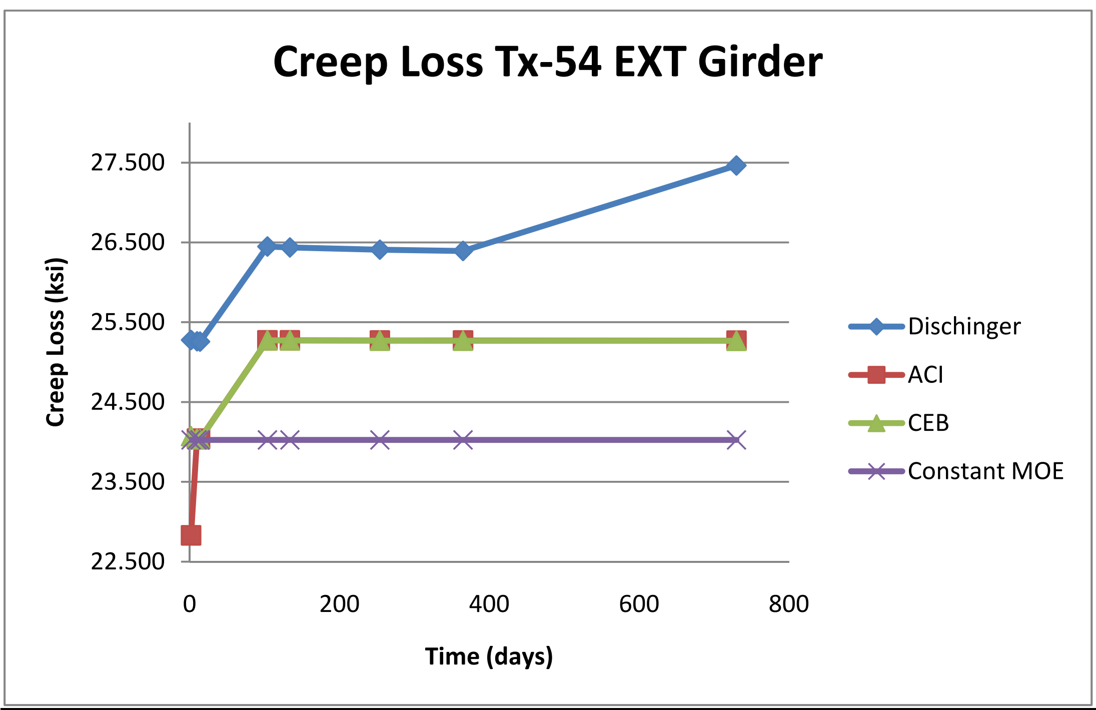

Figure 4.8 Creep Loss over Time for Tx-54 Exterior Girder

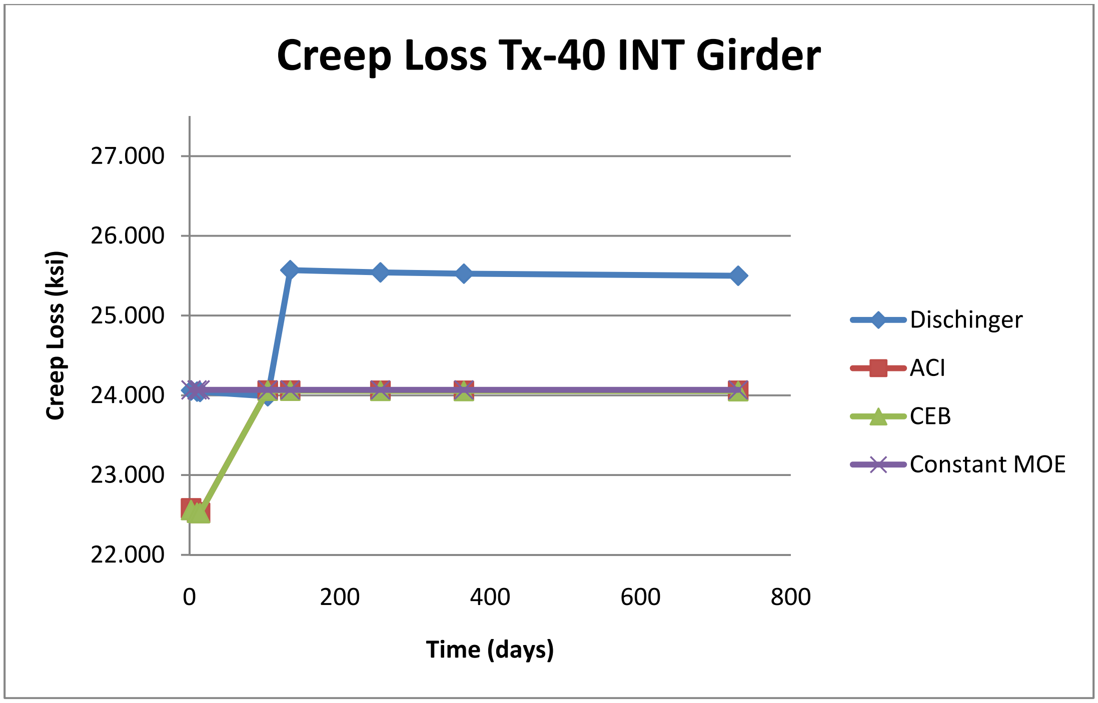

Figure 4.9 Creep Loss over Time for Tx-40 Interior Girder

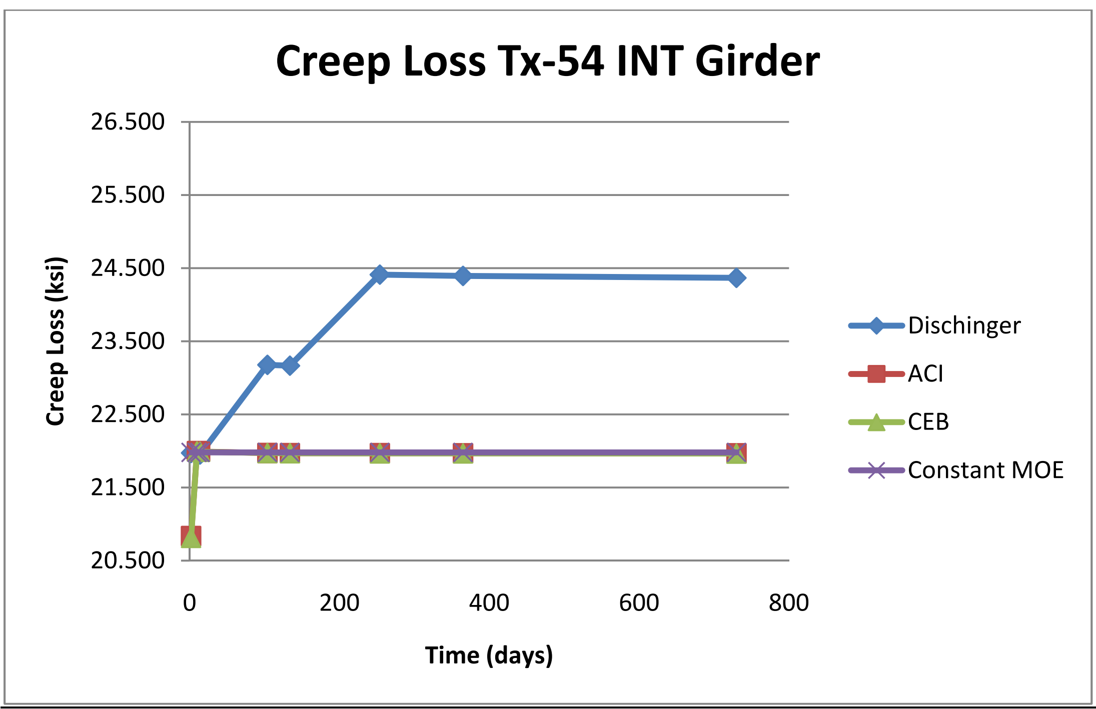

Figure 4.10 Creep Loss over Time for Tx-54 Interior Girder

4.2.3 Steel Relaxation Loss

The results obtained for the loss in steel relaxation after transfer from the run of PSTRS 14 for the time frames given in table 3.3 were also plotted by beam type. These results are given in Fig. 4.11 through Fig. 4.14. It can clearly be seen that the graphs for steel relaxation loss depict a behavior that is opposite to the creep and elastic shortening losses. The graphs for steel relaxation are mirror images to the creep and elastic shortening losses. It can be seen that the losses due to steel relaxation decrease over a period of time and then become constant. From Eq. 1.7 it is evident that the steel relaxation loss is proportional to the elastic shortening, shrinkage and creep losses. Since the elastic shortening and creep losses increase the steel relaxation will decrease. Hence it is also noted that the steel relaxation loss becomes constant after a period of time.

4.3 Long-Term Effects of a Variable MOE on Prestress Losses

The prestress losses calculated by the PSTRS 14 software were also analyzed for a period of 5 to 50 years at 5 year intervals. Once again, the Dischinger Method predicts losses higher than the CEB-FIP 1990 and ACI 209 Models. However it is noticeable that with the Dischinger Method, the values for elastic shortening creep and steel relaxation losses all reach a maximum at 5 years and then remain constant for the life of the structure. With the exception of the interior Tx-54 girder, the other three types of girders exhibit this kind of behavior. The Tx-54 interior girder attains its maximum value of losses at 10 years. This girder is the longest and highest load carrying girder. It shows an increase in the number of prestressing strands necessary at 10 years, therefore this result

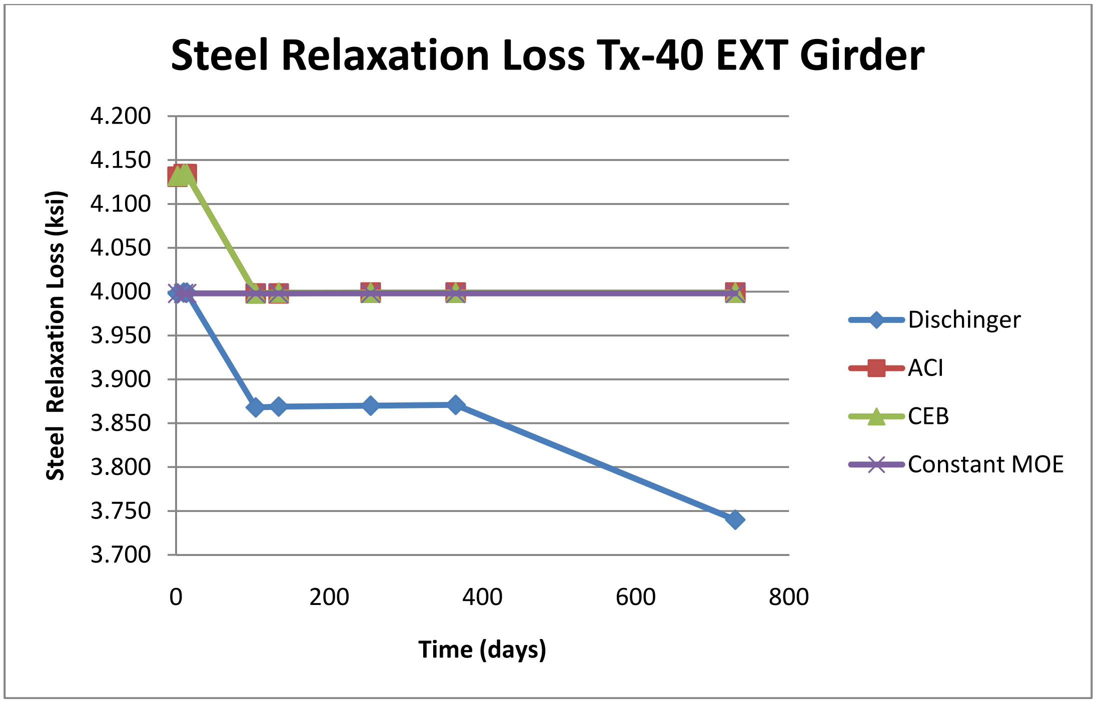

Figure 4.11 Steel Relaxation Loss over Time for Tx-40 Exterior Girder

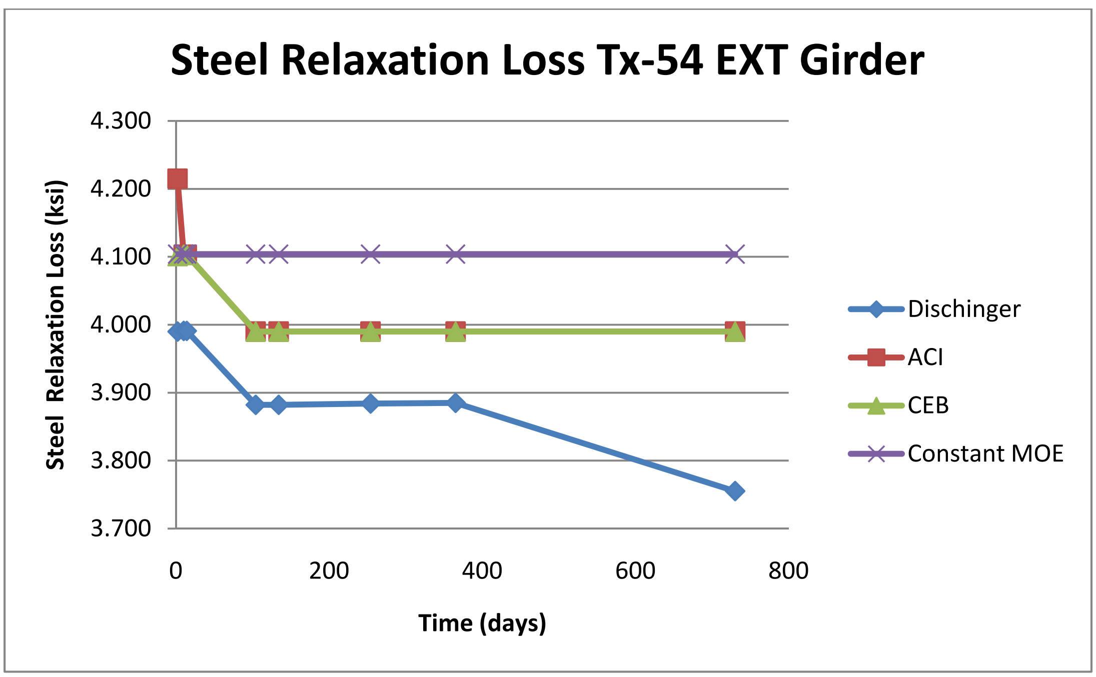

Figure 4.12 Steel Relaxation Loss over Time for Tx-54 Exterior Girder

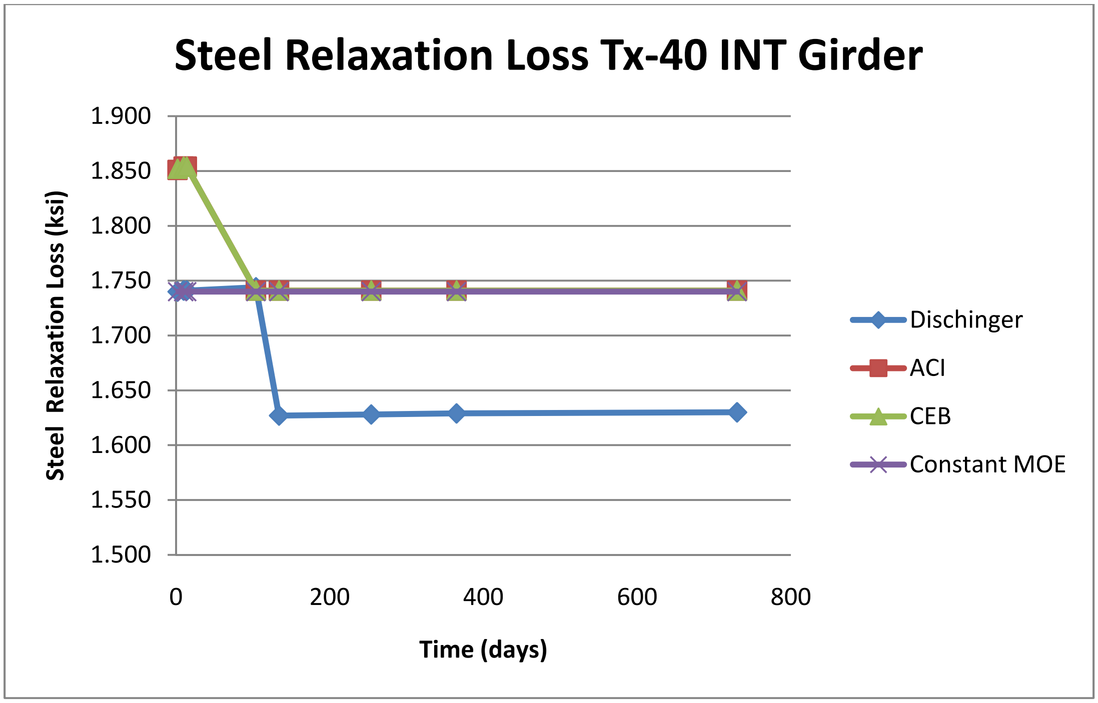

Figure 4.13 Steel Relaxation Loss over Time for Tx-40 Interior Girder

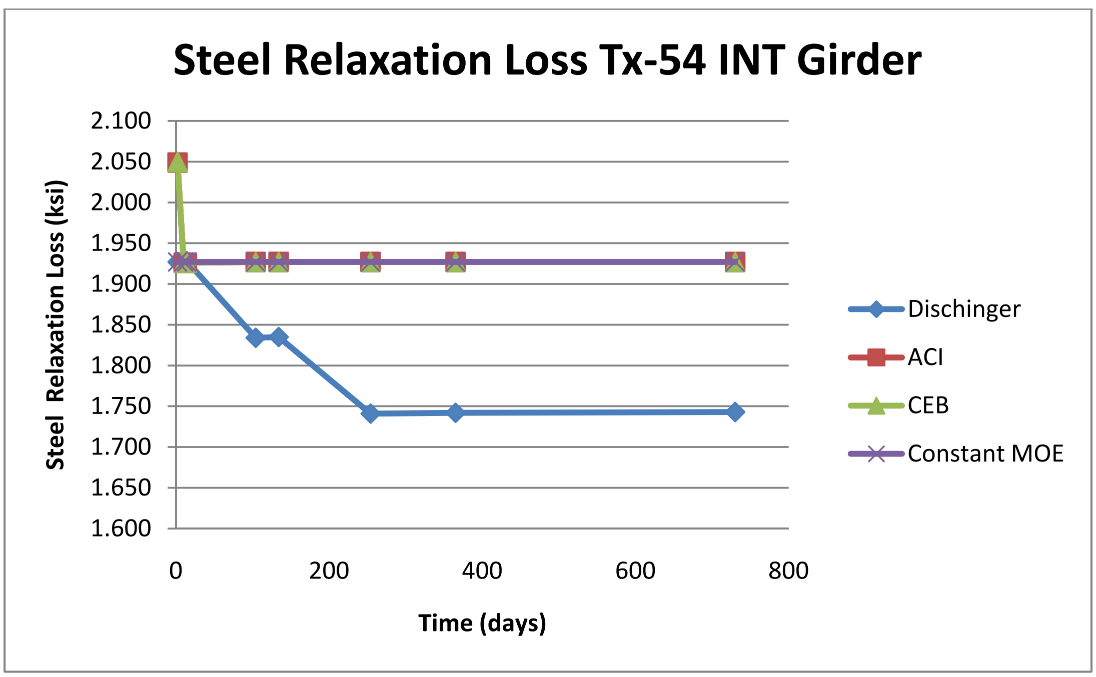

Figure 4.14 Steel Relaxation Loss over Time for Tx-54 Interior Girder

is expected. Table 4.2 gives the change in strands at 5 and 10 years with the Dischinger Method.

Table 4.2 Long Term Change in Strand Design for Dischinger Method

Beam Type	No. Strands Total	Eccentricty at centerline (in)	Eccentricty at end (in)	No. Draped Strands	Yb of Top Draped Strands	Time (years)
Tx-40 Exterior	48	13.02	8.35	8	36.5	5
Tx-40 Interior	48	13.02	8.35	8	36.5	
Tx-54 Exterior	64	17.26	10.14	12	50.5	
Tx-54 Interior	60	17.61	10.94	10	50.5	
Tx-40 Exterior	48	13.02	8.35	8	36.5	10
Tx-40 Interior	48	13.02	8.35	8	36.5	
Tx-54 Exterior	64	17.26	10.14	12	50.5	
Tx-54 Interior	62	17.46	11.01	10	50.5	

The CEB-FIP and ACI Models both show constant losses in this time interval. Appendix C provides beam summary reports for the 5 year and 10 year periods for the Dischinger Model.

4.4 Conclusion

1. The application of Dischinger's Method shows that the prestress losses due to elastic shortening creep and steel relaxation significantly increases with time and MOE.

2. Elastic shortening and creep losses are similar to one another as the modulus of elasticity increases.

3. Predicted steel relaxation losses are opposite in nature to the elastic shortening and creep losses as the modulus of elasticity increases.

4. All prestress losses become constant after 5 to 10 years.

5. The value of the modulus of elasticity becomes constant after 5 to 10 years.

6. When Dischinger's Method is compared to current existing models for a variable MOE it predicts higher values for the variable MOE and prestress losses.

7. Existing models predict a constant value of MOE and prestress losses after the deck is assumed to be cast (period of 104 days).

8. Dischinger's Method can be said to increase the value of the MOE by a percentage calculated as the creep coefficient. The creep coefficient is the percent increase in MOE.

9. The use of Dischinger's Method provides a more conservative beam design and a higher amount of prestress loss can be accounted for.

10. Dischinger's method provides a simple and viable approach to the calculation of a variable modulus of elasticity. The overall simplicity of the application of the method is attractive in nature and can be used by anyone wishing to compare results.

4.5 Future Scope

The Dischinger Method can be studied further in many different ways. The Dischinger method can be further studied for its effect continuous beams and prestress losses in continuous beams. It's affect on deflections can also be studied. Furthermore the effect of Dischinger's method can be applied to differing types of concrete used (such as fiber-reinforced, high early strength, etc.) and the effects of varying the strength of concrete can also be considered. The effects of a variable humidity can be considered as well to observe the prestress losses and the variable MOE in adverse weather conditions and locations. Temperature effects affecting the maturity of concrete can also be evaluated and the actual age of concrete can be modified. Dischinger's method can also be applied to post-tensioned structures.

APPENDIX A

TxDOT STANDARD DRAWINGS FOR BRIDGE GIRDERS AND RAILING

SEE SUPPLEMENTAL FILE

APPENDIX B

PSTRS 14 OUTPUT FOR CONSTANT MOE

SEE SUPPLEMENTAL FILE

APPENDIX C

PSTRS 14 OUTPUT FOR DISCHINGER METHOD

SEE SUPPLEMENTAL FILE

APPENDIX D

PSTRS 14 OUTPUT FOR ACI 209 MODEL

SEE SUPPLEMENTAL FILE

APPENDIX E

PSTRS 14 OUTPUT FOR CEB-FIP 1990 MODEL

SEE SUPPLEMENTAL FILE

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Brahama Singh attained her Bachelor of Science Degree in Civil Engineering and in Engineering Physics with a minor in Mathematics from Texas Tech University, Lubbock, Texas in 2004. She is a licensed Professional Engineer in the state of Texas since 2009. Brahama began her professional career at the Texas Department of Transportation Dallas District office in the District Bridge Design Section. She worked there for three years and then changed paths and began working in the field of Roadway Engineering for two years. Since the fall of 2009 to the present she is working in the field of Construction Engineering as a Project Engineer. She plans to continue her professional career in Construction Engineering in the near future and hopes to attain high career goals.