PRESTRESS LOSSES AND THE ESTIMATION OF LONG-TERM DEFLECTIONS AND CAMBER FOR PRESTRESSED CONCRETE BRIDGES

FINAL REPORT

By

HEMA JAYASEELAN
Graduate Research Assistant
and

BRUCE W. RUSSELL, Ph.D., P.E.
Associate Professor

School of Civil Environmental Engineering

Oklahoma State University

August, 2007

TABLE OF CONTENTS

LIST OF FIGURES

LIST OF TABLES

CHAPTER I

1.0 INTRODUCTION

1.1Overview

Prestressed concrete, an ideal combination of concrete and high strength steel, has emerged as an efficient material for modern construction. The construction of prestressed concrete bridges as a standard practice in the United States dates back to 1949 when the Philadelphia Walnut Ave Bridge was constructed. The technical and economical benefits of prestressed concrete permits longer spans and increased girder spacing. Complimenting this, higher performance concrete can feature lower water to cementious materials ratio (w/cm) and the inclusion of supplemental cementious materials that promote a dramatic improvement of concrete quality and durability.

Efficient design of prestress concrete bridges demands an accurate prediction of prestress losses. The prestress losses are defined as the loss of tensile stress in the prestress steel which acts on the concrete component of the prestressed concrete section. In pretensioned concrete, the four major sources of prestress losses are elastic shortening (ES), creep (CR), shrinkage (SH) and relaxation (RE). Additionally, prestress losses are further affected by variations in material properties of the concrete. Numerous research programs have been conducted and a variety of prestress loss prediction methods have been proposed [NCHRP Report 496 by Tadros et al (2003), Huo, Omashi and Tadros

(2001),]. However, accurate determination of prestress losses has always challenged the prestressed concrete industry. Inaccurate predictions of losses result in excessive camber or deflection of prestressed concrete bridges. Excessive camber or deflection can, in turn, adversely affect the service conditions such as: cracking, ride and overall performance of the bridge. Excessive cracking can even reduce the bridge's durability since cracking can be a route for water borne contaminants to deteriorate the concrete and its reinforcements. The primary objective of this research is to review the relevant literature and databases available from prior research programs. This research will also develop design guidelines towards the accurate estimation of prestress losses, restricted to pre-tensioned concrete. We expect to develop equations for losses from the existing AASHTO LRFD time dependent equations for creep, shrinkage and relaxation. A spreadsheet was developed using the above proposed equations which can be used in design of prestressed concrete bridges. Additionally, some recommendations for design to ODOT will also be made.

1.2 Background

The ACI-ASCE Joint Committee 423 (1958) proposed the lump sum prestress loss estimates. These losses included the effects of creep, shrinkage and relaxation, but excluded the frictional and anchorage losses. The further refinement of losses led to the development of the PCI Committee recommendations (1975), the AASHTO-LRFD method (1977) and the ACI-ASCE Committee recommendations (1979). These methods for the calculation of losses failed to acknowledge the variability of material properties of concrete which then led to either overestimation or underestimation of losses [Shenoy and Frantz (1991), Gruel, et.al,(2000), NCHRP Report 496 (2003), Hale and Russell (2006)]

The National Cooperative Highway Research Program [NCHRP Report 496 (October 2003)] investigated the measurement of material properties (elastic modulus, concrete strength, volume to surface ratio and creep coefficient) and their effect on measured prestress losses and deflections. Further new equations for prestress losses were proposed. The experimental research programs performed on prestressed concrete bridge girders by Tadros et al (2001), Miller et al (2000), Pessiki et al (1996), Hale and Russell (1996) verified that the PCI Design Handbook method, ACI 318 and AASHTO-LRFD equations overestimated the prestress losses. However, the issues in camber and deflection were not discussed in detail, but it was concluded that accurate determination of losses was mandatory for the exact prediction of camber/deflection. The relatively new PCI Bridge Design Manual (1997) with a section titled, "Improved Multiplier Method" built on the previous Multiplier method, was found to predict more accurate camber and deflection in prestressed bridge girders.

1.3 Analytical Modeling

The research also developed and programmed spreadsheets which calculated the losses using the following loss prediction methods: PCI, AASHTO-LRFD and the NCHRP-496 detailed method. The spreadsheet calculates prestress losses along with estimating camber and deflection for different stages of the loading of the beam (e.g. at release, storage and handling, erection, cast-in-place slab, sustained loads, live loads and time).

The primary data including the cross-sectional properties and the concrete strength of the girder has to be fed into the sheet.

The spreadsheet also calculates the day to day losses and camber using the AASHTO Time Step method. Further study will be performed by varying the material properties (elastic modulus, creep coefficient) and the design properties (addition of top prestressed steel and mild steel and cross section details) with the intent of minimizing excessive camber in prestressed bridge girders. With the refinement of the model, direct recommendations are made to ODOT and OTA including the design procedure for the accurate estimation of prestress losses, camber and deflection of prestressed bridge girders.

CHAPTER II

2.0 REVIEW OF LITERATURE

2.1 Definitions

2.1.1 Prestress losses

Prestress losses are defined as the reduction in the tensile stress in prestressing tendons. They are categorized as either instantaneous losses or long-term losses. Instantaneous losses include frictional losses, elastic shortening (ES) and seating loss or anchorage slip. Long term losses occur over a period of time. They include losses in prestress due to concrete creep (CR), shrinkage (SH) and relaxation of prestressing strands (RE).

The generalized variation of the stress over time is due to the various losses illustrated in Figure 2.1.

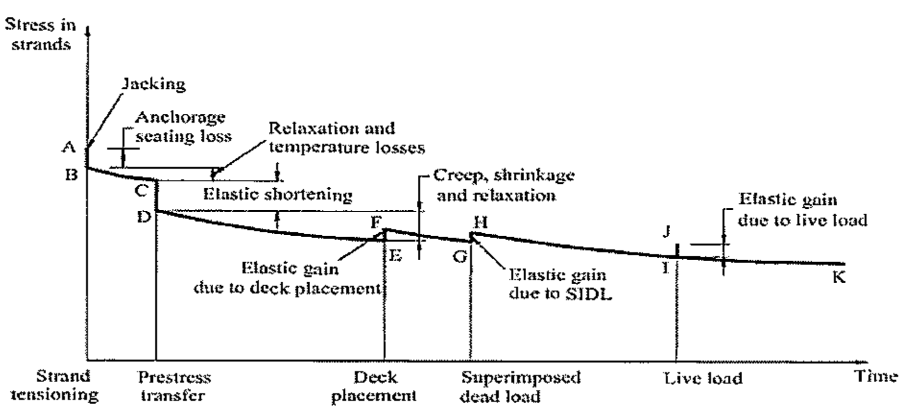

Figure 2-1 Stress versus time in the strands in a pretensioned concrete girder (NCHRP 493 Tadros, 2003)

2.1.2 Elastic Shortening (ES)

Loss due to Elastic Shortening is caused by the instantaneous compression of concrete when the prestress force is transferred to the hardened concrete member. As the concrete shortens it allows the prestressing steel to shorten with it. It is defined as the loss of tensile stress in prestressing steel due to the prestress combined with the stress gain due to the self weight of the member. The Elastic Shortening depends on the modular ratio and the average stress in the concrete at the level of prestressing steel.

2.1.3 Concrete Creep (CR)

The prolonged time-dependent deformation of the concrete under sustained compressive load or stress is called the creep. Concrete creep is subdivided into two parts, basic creep and drying creep. Basic creep is the continual deformation occurring in a sealed specimen subjected to a hydro-equilibrium environment. An unsealed specimen undergoes additional creep (drying creep) due to free exchange of moisture with the environment. This prolonged shortening of the concrete girder results in the loss of prestress. The rate of creep depends on various factors such as: time, magnitude of stress, water-cement ratio, amount and type of cement, ambient relative humidity, and aggregate properties.

2.1.4 Concrete Shrinkage (SH)

The volumetric contraction of concrete specimen due to the loss of free water through evaporation, carbonation or continued cement hydration, in the absence of load is called shrinkage. It is composed of three components: drying shrinkage, autogenous shrinkage and carbonation. The decrease in the volume of concrete due to the diffusion of water into the environment is called the drying shrinkage. Autogenous shrinkage occurs when free water is used in continued hydration of the cement paste after hardening. Carbonation results from the chemical reaction of the carbon dioxide from the atmosphere with hardened cement paste. Again free water is used in the carbonation reaction.

Regardless the source of shrinkage, the change in volume of the concrete causes an overall shortening of the strand length and thus reduces the strand stress resulting in prestress losses.

2.1.5 Relaxation (RE)

Relaxation is the gradual reduction of stress over time subjected under sustained strain. It occurs without the changes in the length of the steel. Relaxation is a property of the prestressing steel and is independent of concrete properties. The most common types of prestressing strands today are Low Relaxation strands which normally have losses that do not exceed 5 ksi.

2.1.6 Camber and Deflection

Camber is the common word for the upward deflection of eccentrically prestressed bridge girders. The amount of camber is governed by the combined action of the prestress force which causes the camber and self weight of the girder to work against the camber. Self weight and other sustained gravity loads can cause downward deflections to exceed the amount of overall camber. The beams and bridges can deflect downward as a result.

Camber and/or deflection are also a function of time dependent concrete creep and prestress loss. Proper estimation of camber or alternatively deflection is essential for an efficient use of longer spans in HPC bridge girders.

The PCI Handbook (PCI 1999) provides equations for deflection and initial and long term cambers. In addition, the new PCI Bridge Design Manual (1997) provides the "Improved Multiplier Method" with intent to predict cambers and deflection. This method, build upon the existing method found in the PCI Handbook, specifically modifies the multipliers. This method also considers the Time to Erection and the creep properties of the concrete.

2.2 Prestress Loss Prediction Methods

Several loss prediction methods have been developed over the years, but simple practical solutions for accurate estimation of prestress loss have proved difficult. The accurate estimation of losses requires more precise knowledge of material properties as well as the interaction between creep, shrinkage of concrete and the relaxation of steel. The current methods for the prediction of losses can be classified according to their approach for the calculation of losses. They are listed according to their descending order of perceived accuracy:

(a) Time-Step methods;

(b) Refined methods;

(c) Lump-Sum methods.

2.2.1 Time-Step methods

These methods fit in to the most detailed group of loss prediction methods based on a step-by-step numerical procedure and they are implemented in specialized computer programs. This method has found its appreciation in multi-stage bridge construction especially in spliced girder and segmental box girder bridges.

The method was developed by dividing the time into intervals whose duration can be increased as the concrete age increases. The stress in the strands at the end of each interval represents difference between the initial conditions at the beginning of that time interval and the calculated prestress losses during that interval. The stresses and deformations at the beginning of each interval equal those at the end of the preceding interval. The research programs performed by Tadros et al (1977), Abdel-Karim (1993) and the PCI-BDM (1997) provide more information on the Time-Step methods.

2.2.2 Refined Methods

In these methods each individual component of prestress losses (elastic shortening and time-dependent losses) is calculated separately. The individual losses are then summed up to obtain the total loss. The difficulty lies in the accurate computation of the interdependency of these individual components. The deck slab of composite sections creep less and shrink more than the prescast girder. This can cause more prestress gain rather prestress loss (Tadros et.al., NCHRP 496 2003)

The accuracy of these methods also depends on the properties of materials, loading and environmental conditions as well as the pertinent structural details. The AASHTO standard Specification method (1993), the AASHTO-LRFD (Refined) method (1998), and the PCI Bridge Design Manual method (1997) use this refined approach.

2.2.3 Lump-Sum methods

Various parametric studies were conducted on the prestress losses of different kinds of prestressed beams under average conditions. The values and trends developed from these studies were utilized in the approximate Lump-Sum methods. Although these methods were useful in the preliminary design, they require reassessment in the final design. The current AASHTO-LRFD Approximate method was developed using this method.

2.3 Background History

Numerous studies have shown that the current models tend to over-predict the long term time dependent losses and hence the camber and deflection in prestress beams (reference). The following literature gives a brief history of the relevant projects that have measured prestress losses and compared with various theoretical models for prestress losses , concrete creep and shrinkage.

Several investigations on concrete creep, shrinkage and prestress losses were performed in different universities in the United States. They have compared the values of creep, shrinkage and prestress losses measured in the laboratory and field with those of the different existing models and methods for the estimation of prestress losses. This section gives a summary of projects that compared the measured and predicted prestress losses.

2.3.1 Tadros, Al-Omashi, Seguirant and Gallat: NCHRP Report 496, (2003)

A detailed research was conducted by Tadros et al (2003) with an objective to develop design guidelines for estimating prestress losses in high-strength pretensioned concrete bridge girders. The NCHRP report 496 aimed in the development of new guidelines and formulas for prestress losses due to the limitations in the current methods used in the estimation of losses. This research encompassed both experimental and theoretical programs. In the experimental program seven full scale bridges from four different states: Nebraska, New Hampshire, Texas and Washington, were investigated for the effect of material properties and other factors on the prestress losses in pretensioned concrete bridge girders. In addition to this investigation, previously reported test results of 31 pretensioned girders in seven states were listed as shown in Table 2-1. Several variables including the Modulus of Elasticity, creep and shrinkage of concrete were studied. The research demonstrates that both the AASHTO-LRFD and ACI Committee 363 equations tend to underestimate the Modulus of Elasticity of high strength concrete due to their inability to account for the properties and the amount of coarse aggregates in the equation. It was also found that the effects of high strength concrete and the interaction between the precast pretensioned concrete girder and the precast or cast-in-place concrete deck were not accounted in the AASHTO-LRFD specifications.

A thorough study was made on the Time-Step Method, Refined Method and Lump-Sum Methods and their limitations were discussed. A complete study of the various factors affecting the creep and shrinkage of concrete was also done. A detailed method using the age-adjusted effective modulus and an approximate method were proposed for reasonable estimates of prestress losses. Both the methods were found to have good correlation with the experimental test results than the current AASHTO-LRFD methods. Numerical examples were also provided to demonstrate the applicability of the proposed loss prediction methods.

Table 2-1 Measured versus estimated total prestress losses for 31 previously reported pretensioned girders.(Tadros et.al.,NCHRP 2003)

Project		Measured[1]	PCI-BDM	AASHTO-LRFD Specifications		Proposed approx.	Proposed detailed
No.	Reference			Refined	Lump-sum		
1	Greuel et al. (*37*)	37.74	34.16	46.31	32.03	35.81	37.83
2	Pessiki et al. (*38*)	36.46	42.48	47.45	50.15	34.69	33.74
		36.64	42.99	47.64	50.98	36.27	35.56
3	Mossiossian et al. (*39*)	32.54	34.07	45.87	52.05	36.72	35.20
		35.11	34.07	45.87	52.05	36.72	35.20
4	Kebraei et al. (*40*)	17.92	23.68	36.61	38.93	23.92	23.71
		36.77	23.68	36.61	38.93	23.92	23.71
5	Shenoy et al. (*41*)	25.18	37.32	31.66	32.92	32.25	36.67
6	Stanton et al. (*42*)	34.17	25.76	34.72	41.29	26.71	31.62
		34.00	27.52	34.72	41.29	26.71	31.62
		65.62	40.14	63.35	54.31	38.45	39.06
		55.06	40.14	63.35	54.31	38.45	39.06
		69.29	40.14	63.35	54.31	38.45	39.06
7	Seguirant et al. (*43*)	36.11	43.33	50.05	51.25	35.16	41.66
		41.65	44.00	50.28	51.69	37.05	46.63
		35.03	46.06	50.39	53.40	37.91	47.98
8	Gross et al. (*35*)	35.68	37.98	61.76	48.21	38.50	33.91
		30.30	40.24	65.73	49.91	39.50	30.03
		32.51	38.41	60.95	47.59	38.01	34.64
		26.02	34.00	55.57	46.35	35.89	30.52
9	Gross et al. (*35*)	43.69	48.63	92.35	58.42	53.29	43.60
		50.80	48.87	92.60	58.42	53.43	43.85
		43.99	49.29	95.13	57.94	57.10	45.51
		44.68	49.81	95.07	58.20	56.40	44.84
		49.93	41.68	80.53	53.46	49.51	39.25
		50.80	48.90	95.43	59.05	56.46	44.88
		48.46	50.45	96.94	59.11	57.47	46.16
10	Gross et al. (*35*)	28.24	34.18	48.92	47.50	38.81	31.24
		27.95	34.18	48.92	47.50	38.81	31.24
		26.25	34.18	48.92	47.50	38.81	31.24
		23.96	30.64	46.36	47.48	36.76	27.72
Ave. Estimated/Measured Ratio			1.06	1.60	1.37	1.08	1.00

[1]Modified for time infinity.

The experimental program included the material testing and the measurement of material properties of the bridge girders. In the material testing program both laboratory and on-site material testing were performed. Similar concrete specimen cylinders were cast for both the testing methods and concrete properties including: concrete strength, modulus of elasticity, creep and shrinkage were measured. Proposed formulas for the modulus of elasticity, creep and shrinkage of concrete were developed based on the test results. The modulus of elasticity of concrete was determined in accordance with ASTM C469-94. Comparison of test results with that of the prediction formula given by ACI 318, ACI 363 and AASHTO specifications demonstrated that none of them accounted for the effects of aggregate type on the modulus of elasticity of concrete. Therefore, the proposed formula developed based on the test results included two factors K_1 and K_2, where K_1 represented the difference between the national and local average. K_2 represented the desired usage of an upper-bound or lower-bound value in the calculations. These factors give the proposed formula the ability to account for local materials and also for high strength concrete. Similar observations were made for creep and shrinkage specimens. The specimens had a V/S ratio of 1.0 at an ambient relative humidity of 30% to 40% and the strains were measured using Demountable Mechanical Gauges (DEMEC). The ratio of measured to estimated values of creep and shrinkage using the AASHTO-LRFD and ACI 209 were found to be much lower than the desired ratio. The proposed creep and shrinkage formulas developed encompassed various correction factors such as: relative humidity correction factor, volume to surface ratio correction factor, loading age correction factor, concrete strength correction factor and time-development correction factor. An approximate method was also proposed for the estimation of the Long-term prestress losses.

The second part of the experimental program consisted of the testing of the seven full-scale bridge girders of the four different states which represented a range of geographic conditions and construction practices. Measurements of concrete strains and temperatures were taken with the aid of vibrating strain gauges to calculate the loss in prestress force. The measured total prestress losses averaged to 37.3 ksi. The proposed detail method was verified by comparing its predicted results with the measured prestress losses. The proposed modulus of elasticity formula was found to give more accurate results than those obtained by the current AASHTO-LRFD and the ACI 363 formulae. Moreover, the proposed shrinkage formula produced results that averaged 105% of the measured values compared to 174% when using AASHTO-LRFD method and 155% when using the ACI-209 method. The creep formula averaged 98% of the experimental values compared to 161% for AASHTO-LRFD and 179% for using ACI-209. The above methods appear to perform poorly than the NCHRP 496 method which may be due to the fact the NCHRP 496 method compared the results on bridges where the material properties had been accurately measured. Whereas the other methods were based on formulas that were intended for general use. The substantial effect of the differences in the creep coefficients on the long-term prestress loss estimation, previously observed by Hou et al (2001), Mokhtarzadeh and Gross (1996) were reconfirmed through the experimental results. Table 2-2 lists values of measured versus estimated prestress losses using the AASHTO-LRFD refined and Lump-Sum methods, PCI-BDM and the proposed detail and approximate methods. The measured losses from the experimental program were also listed. The overall research and study concluded that the proposed methods were found to have good correlation with the tests results for the estimation of the prestress losses.

Table 2-2 Measured versus Estimated Total Prestress losses (Tadros et.al., NCHRP 2003)

Girder	Measured[1]	PCI-BDM		AASHTO-LRFD Specifications				Proposed approximate method		Proposed detailed method (using estimated properties)		Proposed detailed method (using measured properties)	
				Refined		Lump-sum							
	Loss	Loss	Ratio*	Loss	Ratio*	Loss	Ratio*	Loss	Ratio*	Loss	Ratio*	Loss	Ratio*
Nebraska G1	31.96	36.85	1.15	52.24	1.63	50.29	1.57	40.18	1.26	38.42	1.20	40.68	1.27
Nebraska G2	35.65	38.27	1.07	52.24	1.47	50.29	1.41	40.18	1.13	40.00	1.12	40.68	1.14
New Hampshire G3	43.51	39.84	0.92	54.26	1.25	50.51	1.16	41.50	0.95	41.39	0.95	36.51	0.84
New Hampshire G4	42.33	39.84	0.94	54.26	1.28	50.51	1.19	41.50	0.98	41.39	0.98	36.51	0.86
Texas G7	25.35	32.11	1.27	52.52	2.07	48.83	1.93	34.20	1.35	27.67	1.09	25.46	1.00
Washington G18	42.06	40.33	0.96	66.86	1.59	52.69	1.25	38.07	0.91	35.85	0.85	38.47	0.91
Washington G19	39.98	40.33	1.01	66.86	1.67	52.69	1.32	38.07	0.95	35.85	0.90	38.47	0.96
Average	---	---	1.05	---	1.57	---	1.41	---	1.07	---	1.01	---	1.00
Standard deviation	---	---	0.12	---	0.26	---	0.25	---	0.16	---	0.12	---	0.15

[1] Modified for time infinity.
* Ratio to measured losses.

2.3.2 Huo, Al-Omaishi, and Tadros (2001)

Huo,Al-Omashi and Tadros (2001) studied the time dependent properties of concrete such as: shrinkage, creep, modulus of elasticity of (HPC) and predicted equations and procedures to determine the above mentioned properties. Understanding the major effects of concrete creep and shrinkage and modulus of elasticity of concrete in the determination of prestress losses of high strength bridge girders, this paper throws light on analytical and experimental procedures required for the refinement of ACI 209 equations for the prediction of time dependent properties of high strength concrete. From the research findings of the paper it has been proved that the ACI 209-92 equations tend to over-estimate the actual shrinkage strains and creep coefficients of high strength concrete. Further more the ACI equations for the modulus elasticity of normal concrete cannot be used to predict the modulus of elasticity of high strength concrete.

The experimental procedure involves the selection of three HPC specimens (12SF, 12FAand 8FA). SF and FA denote mixes with silica fume and fly ash respectively. The mixes were prepared and tested using the locally available materials in Nebraska. A total of 22 (100x100x600mm) shrinkage specimens were prepared, placed and cured in the structural laboratory of the University of Nebraska, Omaha. The shrinkage strains were determined by DEMEC (Demountable Mechanical points) which were attached to the surface of the specimens. In parallel to this, 30 similar creep specimens were also cast and cured at the same time. They also had DEMEC points attached to their surface and were monitored for creep deformation. The testing was done in accordance with ASTM C 512-87. The experimental results proved that ACI 209-92 equations for the prediction of shrinkage strains and creep coefficient were much larger than the measured shrinkage strains and creep coefficient. It was also found that the shrinkage strains and creep strains of HPC tend to develop rapidly in the early age of concrete. Moreover, the ACI equations did not account for the strength of concrete. This resulted in the prediction of new equations for shrinkage and creep with strength correction factors.

Similar tests for modulus of elasticity of concrete were also performed on concrete specimens at the ages of 7, 14, 28 and 56 days. The comparisons of the test results with the various codes prove that both the ASHTO and the ACI equations underestimate the modulus of elasticity of the three HPC mixes. Further studies from the test results showed that the modulus of elasticity was affected not only by the compressive strength but also by the type and amount of coarse aggregate present in the mixture. Therefore new equations for modulus of elasticity for HPC mixtures were developed. However it was finally stated that the conduction of trial tests was the preeminent way of determination of the modulus of elasticity of a specified mixture.

2.3.3 Concrete Technology Associates Technical Bulletin, (1973)

The technical bulletin on prestress losses by Concrete Technology Associates highlights the mechanisms involved and the significance of losses in prestressed concrete. Three standard 60' series Washington State pretensioned Bridge girders manufactured by the Concrete Technology Coorporation (CTC) were selected. The test beams were subjected to a series of flexural tests at CTC's Tacoma plant after two years. The beams were reloaded to determine the point of crack opening from which the prestress losses were calculated directly from statics based on the assumption of linearity of stresses and strains. The three bridges exhibited a measured loss of 35.3ksi where as the predicted losses using AASHO interim specifications for prestress losses (1971) and the PCI Design Handbook were found to be 63 ksi and 58 ksi. This paper points out that the CTC test on the bridge girders indicate reasonable values of prestress losses for I-beams whereas the AASHO recommendations and the PCI method tend to be over conservative.

On exploring the reasons behind the overestimation, the following short comings were found. AASHO does not take into account the elastic recovery, role of concrete strength and the time variant strength properties of concrete during the calculation of prestress losses. Previous research made by Ghali et.al. (J.ACI, Dec 1967) has shown a gain of prestress through creep recovery. This paper has also suggested some ways as how the performance of prestress products can be improved. The objectives include the use of high-strength concrete, shrinkage compensation, combined pre-and post-tensioning, stabilized strand, membrane, preloading and initial overstressing. Various methods for the reduction of prestress losses have also been presented in this paper. Based on the test results and the comparison of prestress values of the different methods the CTC have modified the equations of the PCI Design Handbook method for the calculation of prestress losses. The results conclude that the allowance of creep recovery and the high strength concrete in the modified equations allow the calculated prestress losses to be very close to the measured prestress losses.

2.3.4 Hansen and Mattock, (1966)

Hansen (1966) and Mattock demonstrated the influence of the size and shape of relatively large sections on the shrinkage and creep of concrete. A suitable parameter known as the volume to surface ratio (V/S) was selected to relate to the size and shape of the member. The significance of creep and shrinkage (especially in prestressed concrete) necessitated the exact prediction of creep and shrinkage in concrete structures. In order to accomplish this, a relationship between shrinkage and creep at any time, and a parameter representing the size and shape of the member called the volume to surface ratio was developed. From the theoretical background research performed by Pickett (1946), it has been confirmed that a relationship between shrinkage and the shape and size of the member can be recognized.

Test specimens were prepared with two different materials: eigen gravel concrete and crushed sandstone. Further comparative studies were performed between two types of specimens, I-section members (11.5 to 46in deep) and concrete cylinders (4 to 24 in dia) with the same volume to surface ratio which were cast in the laboratory. The specimens were stored and cured in fog rooms from the time of casting and were mounted with 10in demountable mechanical strain gauges (DEMEC). The creep specimens were loaded to an applied stress of 25 percent of the compressive strength at 8 days. Strain readings were also measured from the shrinkage specimens simultaneously and the shrinkage and creep strains were recorded. From the measured data, the amount of shrinkage and creep at a given age was found to increase with a decrease in the size of the specimen. The research concluded that the both shrinkage and creep of the concrete (at all ages were) affected by the changes in the shape and size of the member.

2.3.5 Lwin, Khaleghi and Hsieh, (1997)

The main aim of Lwin and Hsieh (1997) was to illustrate the design practice of Washington State Department of Transportation (WSDOT) and the application of the AASHTO LRFD Bridge Design Specifications (1996) in order to predict the time-dependent prestress losses. The added benefits of the High Performance Concrete (HPC) including the durability characteristics and the strength parameters have also been highlighted. To compliment this, a design example for the determination of the time-dependent prestress losses of a WSDOT W74MG prestressed I-girder using Time-Step method and Modified Creep method has been worked out.

The prestress losses including the creep, shrinkage and relaxation of prestressing strands were determined using the AASHTO LRFD (1996) equations. In conjunction, the prestress gain due to the slab casting and differential shrinkage were also determined using the equations proposed by Branson et.al. Four other different methods: the AASHTO LRFD Approximate Lump Sum Estimate method, AASHTO LRFD Refined Estimate method, Time-Step Method and Modified Creep method were also suggested for the determination of time-dependent losses. Comparison of the predicted prestress loss values was performed between the above four methods as shown in Table 2-3. It was observed that the prestress losses computed using the Modified Rate of Creep Method was lower than the ones obtained from other methods. This was because this method takes into account the instantaneous and time-dependent effect of slab casting as well as the transition from non-composite to composite section properties. As mentioned previously, it includes the elastic and creep effect of slab casting as well as the prestress gain obtained from differential shrinkage.

Table 2-3Comparision of Prestress losses in ksi (Reproduced from Lwin et.al.,1997)

	AASHTO-LRFD Approx.Method	AASHTO-LRFD Refined.Method	Time-step Analysis	Modified Rate of Creep Method
Transfer	23.1	23.1	23.1	23.1
Before Slab Cast	-	34.6	33	28.6
After Slab Cast	-	33.7	31.1	25.5
Final	50.5	54.5	47.5	41.1

It was found that the losses obtained using the Time-Step Method were lower than the ones obtained using the AASHTO LRFD refined estimate. This is because the former method uses the effective prestress force than the initial force at transfer. The final deflection of the prestressed girder using the Modified Rate of Creep Method was also found. The paper concludes that the AASHTO LRFD specifications provide a logical and straight forward design of prestressed-I girders. It is also a reasonable method for the prediction of prestress losses due to creep and shrinkage. However it also states that the Modified Rate of Creep method provides more accurate prestress losses.

2.3.6 Shenoy and Frantz (1991)

Two 27-year old precast, prestressed concrete box beams were removed from a deteriorated 54ft span multi-beam bridge (Shenoy and Frantz 1991). The beams were subjected to structural tests and the measured ultimate flexural strength and prestress losses were compared with the predicted values of AASHTO (1989) and ACI-318(1989) code methods. The research program was a combined venture of Connecticut Department of Transportation, the Precast/Prestressed Concrete Institute (PCI) and the Blakeslee Prestress Inc. The main objectives of the study were to determine the effect of 27 years of service on the strength of the beams, the prestress losses and the amount of chloride ion penetration into the concrete. The two beams (beam7 and beam 4) were partially deteriorated due to the penetration of de-icing salts and also due to removal from the bridge.

The material properties of the concrete were determined from the eight cores which were removed from the top flanges of the beams. It was found that the measured values of modulus of elasticity and compressive strength concrete were less than the values predicted using the ACI Committee 363(1984) method. The stresses in the strands were determined both at the University of Connecticut using simple methods (Shenoy and Frantz 1991) and at the Florida Wire and Cable Company using the special strand testing method. The test results agreed very well. However, the value of initial prestress was unknown. Therefore, the final prestress losses were predicted using the PCI general method (PCI-1975) by using different combinations of assumed and measured material properties. The beams were AASHTO-PCI Type BI-36 sections and were subjected to flexural tests. The beams were simply supported on rollers and were permitted for rotation and translation at supports. Strain gauges were attached to the several strands at the mid span to record the strains. The beams were loaded at one third points in stages with the aid of four hydraulic rams. The load, strains, mid span deflection and end rotations of the beams were recorded.

The load deflection curves developed for both the beams proved that they exhibited linear elastic behavior prior to flexural cracking. The measured capacities exceeded the flexural strength values predicted using the AASHTO specifications. The measured rotations also closely followed the rotations predicted by using moment-area principles with a strain compatibility moment curvature section analysis. The prestress force in the beams was determined by observing the reopening of the flexural cracks. The measured prestress losses were much lower than those predicted using the PCI general method. The test results also concluded that the strain compatibility and the moment curvature accurately predicted the beam behavior.

2.3.7 Greuel, et.al, (2000)

An experimental research conducted by Miller et.al. (2000), investigated the static and dynamic response of HPC bridge girder and explored the load transfer between the box girders through experimental mid-depth shear keys. They also experimentally measured the prestress losses in the high strength concrete bridge girders. The Ohio Department of Transportation (ODOT) constructed a (HPC) box girder and similar prototype girders for experimental testing. The prototype girders were tested for both destructive and non-destructive testing and the HPC bridge girder was tested for nondestructive testing. The completed new HPC bridge replaced a 70ft single span, steel stringer bridge with a concrete deck slab located on the US Route 22. Actually, this bridge was designed as a three-span, non-composite, adjacent box girder system using normal strength and 0.5 in diameter prestressing strands. But later it was redesigned as single-span bridge with a span of 115.5 ft and 0.6 in diameter strands. This was successfully performed considering the benefits of the high strength concrete.

Prior to the casting of the girders the researchers at the University of Cincinnati (UC) developed a mix design which produced a concrete with a minimum compressive strength of 10ksi and with a rapid chloride permeability of less than 1000 coulombs. Type III cement with water to cementitious ratio of 0.3 was used and micro silica was added to the mix. Two prototype box girders were casted and tested. It was found that the test results yielded a transfer length between 60D and 80D whereas the AASHTO Standard (1998) and AASHTO-LRFD (1998) specifications use a transfer length of 50D and 60D. It was also found that the AASHTO standard specifications underestimate the value of cracking and ultimate moment. The reason behind this was that the AASHTO prediction equations take into account the modulus of rupture of normal concrete (750 psi) whereas the modulus of rupture of highs strength concrete has a much higher value (1250psi). Prestress losses were determined experimentally from the measurement of the crack opening during loading by the use of clip gauges across the cracks. The measured prestress losses were found to be about 18 percent whereas the AASHTO specifications and PCI methods predicted a loss of 20 and 18 percent.

The actual standard ODOT HPC box girders (numbering 12) were designed according to AASHTO specification and fabricated for a HS-25 loading. A total of 69 internal sensors were embedded during fabrication for the measurement of temperature strains, static and dynamic truckload strains. The girders were erected in two phases and were provided with shear keys at mid-depth to arrest the spalling due to the movement between the girders. Truck load tests were performed on the girders for various positions of loaded trucks across the girder. The vertical deflection of each girder at mid span and quarter points were measured with the aid of wire potentiometer displacement transducers. The test results concluded that the maximum static deflections were only 35 to 50 percent of that allowed by the AASHTO standard specifications and all the girders were working together as a system. The measured dynamic deflections were also found to be lower than the predicted values.

2.3.8 Pessiki et.al, (1996)

Pessiki, Kaczinski and Wescott (1996) performed an experimental research at the Center for Advanced Technology for Large Structural Systems (ATTLSS), Lehigh University.

The objective of the research was to determine the effective prestress force in two full scales prestressed concrete bridge beams in order to evaluate the load rating on these bridges. The two bridge beams were 24 x 60 in prestressed concrete I-beams with a span of 89 ft from the Pennsylvania Department of Transportation (PennDOT). Its overall length was 90 ft 5in with a total of 50, 7/16 in diameter prestressed strands. The beams were removed from 28 years of service on a dual seven-span Shenango River Bridge. They were tested on a 5000 kip capacity Universal Testing Machine at a three point load configuration. Prior to testing the beams were surveyed for a mid span camber of 1.31in and 1.56 in respectively.

The experimental program comprises of three phases (i) Cracking Load Test, (ii) Decompression Load Test and (iii) Ultimate Load Test. The first phase comprised of loading of the specimen to locate and create a series of flexural cracks. A series of strain gauges and strain transducers were used to record the strain distribution throughout the depth of the member. The second phase involved the determination of the decompression load from the Load-Strain curves obtained from the strain gauge and displacement transducer readings which were mounted across the cracks. In the third phase, the beams were loaded to failure and the ultimate strength of the beams were observed. In addition to this, core samples were also obtained from the beams to determine experimentally the compressive strength, modulus of elasticity and splitting tensile strength of the beams. It was found that the cores had an average compressive strength of 8440psi which was 65 percent greater than the design strength and the modulus of elasticity was 11 percent greater than that predicted using the ACI 318-95(1995) equation. From the experimental observation it was found that the beams had remained uncracked during their 28 years of service. With the knowledge of the average decompression load, simple elastic analysis was used to calculate the effective prestress force in each beam. Thus the measured average prestress loss for both the beams were found to be approximately 60 percent of that predicted using the AASHTO Standard (1992) and PennDOT(1993) design specifications. The results of the study also concluded that the use of strain gauges for the determination decompression load produced most reliable and repeatable results.

2.3.9 Hale and Russell (2006)

Hale and Russell observed the effects of increased allowable compressive stresses at release on the performance of precast prestressed concrete bridge girders. A comparative study was also performed between the experimental results of the measured losses and the estimated losses. The losses were estimated using the three most widely accepted methods such as: the AASHTO LRFD refined method(2004), PCI Design Handbook method described by Zia et al (1979) and the detailed method proposed by the National Cooperative Highway Research Program (NCHRP) Report 496(2003). The purpose of their research was to examine the effect of increasing the allowable compressive stress at release from 0.60f'c to 0.70f'c.

Four prestressed I-shaped girders were cast and tested with a compressive release stress ranging from 0.57f'c and 0.82f'c. Two of the beams were air-entrained whereas the other two were not air-entrained. Some strands were debonded to achieve the targeted compressive release strengths. Concrete strains were measured with the aid of a detachable mechanical strain gauge (DEMEC). Prestress losses were measured by multiplying the measured concrete strains by the elastic modulus for the prestressing strands. Three of the four girders exhibited a maximum compressive stress at release greater than the allowable limit 0.6f'c. It was observed that although the measured prestress losses were high due to the increase of the allowable compressive stress at release none of the beams experienced adverse effects and no external signs of distress were visible.

Examination of the losses concluded that the 2004 AASHTO (refined method) and the Zia.et al prediction equations overestimated the measured losses by an average of 18% and 13% respectively where as the detailed method from the NCHRP Report 496 predicted more accurate losses than the former two methods. Table 2-4 shows ratio of the measured to predicted prestress losses using AASHTO (2004), Zia et al (1979) and the NCHRP 496 (2003) methods. The results concluded that the data provides support for increasing the allowable compressive release strength from 0.6f'c to 0.7f'c. It was also concluded that the addition of entrained air had negligible effects on prestress losses. Figure 2-2 graphically depicts the effects of air entrainment and increase in allowable compressive stress at release in the four girders. Table 2-4 shows the comparison between the measured prestress losses and the predicted losses using three design methods AASHTO LRFD, Zia et., al and the NCHRP 496.

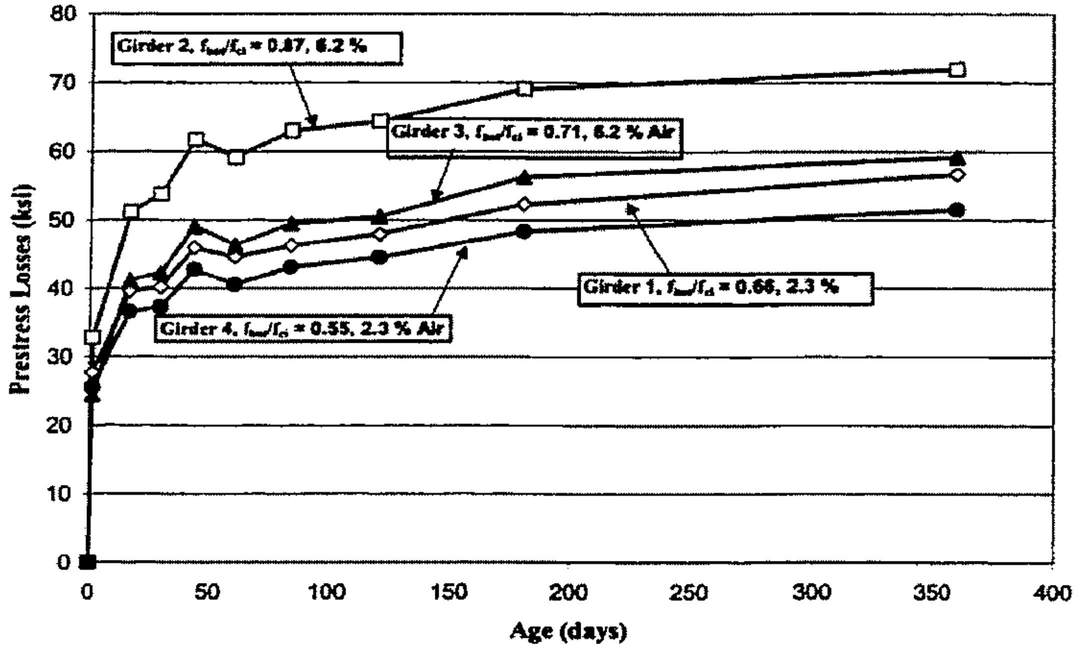

Figure 2-2 Measure Total Losses (Hale 2002)

Table 2-4 Ratio of Measured to Predicted Losses (Hale and Russell , 2006)

Girders	Location	Ratio of Measured to Predicted Losses AASHTO 2004	Zia et al (1979)	NCHRP 496 (2003)
1	Ends	0.72	0.81	1.07
	Center	0.68	0.77	1.01
2	Ends	0.92	0.89	1.04
	Center	0.95	0.92	1.08
3	Ends	0.93	0.94	1.05
	Center	0.92	0.94	1.05
4	Ends	0.73	0.84	1.08
	Center	0.74	0.84	1.09
Average		0.82	0.87	1.06

2.3.10 Waldron (2004)

Waldron emphasized on the advantages of high strength concrete in prestressed bridge girders and performed a thorough investigation on three prestressed concrete bridges (Chickahominy River Bridge(HPLWC), Pinner's point Bridge and Dismal Swamp

Bridge) constructed in Virginia. Studies on long-term prestress losses were performed and comparisons were made between the measured and predicted prestress losses. Several existing methods including the AASHTO Standard (1996), AASHTO LRFD (1998), PCI Bridge Design Manual (PCI-BDM,1997) and NCHRP Report 496 (Tadros et. al.,2003) and other creep and shrinkage models including ACI-209(1992), CEB-FIP (1990), PCI Committee on Prestress Losses (1975), PCI-BDM, B3 (Basant and Baweja,1995 a,b,c), GL2000 (Gardner and Lockman, 2001), AFREM (Le Roy et. al.,1996), AASHTO LRFD, Shams and Khan (2000), and NCHRP Report 496, were used for the comparison of measured and predicted prestress losses.

Based on the comparisons made on the above mentioned methods and specifications a design recommendation was provided for the determination of prestress losses for Virginia's prestressed concrete. In conjunction, creep and shrinkage studies were conducted on three prestressed concrete bridges in Virginia Tech. The girders were instrumented to measure the long-term creep and shrinkage strains associated with the prestress losses. The results of the experimental studies proved that the AASTO Standard and LRFD specifications over predicted the total losses of the three bridges by18% to 98% whereas the NCHRP 496 Refined method under predicted the total losses for the three bridges between 82% and 98%. Finally, it was stated that the PCI-1975 method was the best predictor of prestress losses for the HPLWC girder with an overestimation of measured losses by 17%. The PCI-BDM was the most consistent method for the prediction of total losses, overestimating the measured prestress losses of HPLWC by 18%.

2.3.12 Sridhar et.al, (2005)

Measured prestress losses were compared with those obtained using the recommendations given in the different code practices such as: ACI Committee 209 (2002), PCI General Method for the Computing of Prestress Losses (1975) and the CEB-FIP Modal code(1978). In conjunction, estimated prestress losses of the field data collected on a prestressed concrete highway bridge were also compared with the design codes. Although other different codes of practices (i.e, AASHTO Standard Specifications (1996) and IS-1343code) were recommended for the estimation of prestress losses. The above three methods were used since they predict the prestress losses at a particular time within the lifespan of the structure.

The prestress losses were measured from laboratory tests on two prestressed beams using vibrating strain gauges. Vibrating strain gauges were chosen to measure the strain in the prestressed beams due to their stability and reliability over periods exceeding 15 years as previously stated by Window and Holister (1982). Two embedment type and two surface mounting type vibrating strain gauges were used in the measurement of strains in the two prestressed beams. The measured strains were also corrected for the temperature induced apparent strains in the gauges. The initial prestress was computed from the measured strain values. The beams were monitored under laboratory conditions for a period of 400days. Since no external loads were applied the time dependent strains were calculated based on the creep, shrinkage and relaxation of the prestressing steel. A comparative study of the measured prestress losses with the design codes was done. It was found that the prestress losses predicted using the ACI 209 method matched with the measured losses at the early ages (up to 50 days). However this method was found to underestimate the losses up to 15 to 20% at the later stages. It was observed that the PCI method underestimated the measured prestress losses by about 20 to 30%. The reason was that the PCI method adopted a relative humidity of 70%, while the field relative humidity was not the same throughout the year. Unlike the ACI method the CEB-FIP method does not take into account the parameters for the change in cement content, water content and aggregate ratio. Therefore it was concluded that the CEB-FP underestimated the prestress losses. Table 2-5 and Figures 2-3 and 2-4 represent the comparisons between the estimated prestress losses, using the code estimates, and the field measured losses.

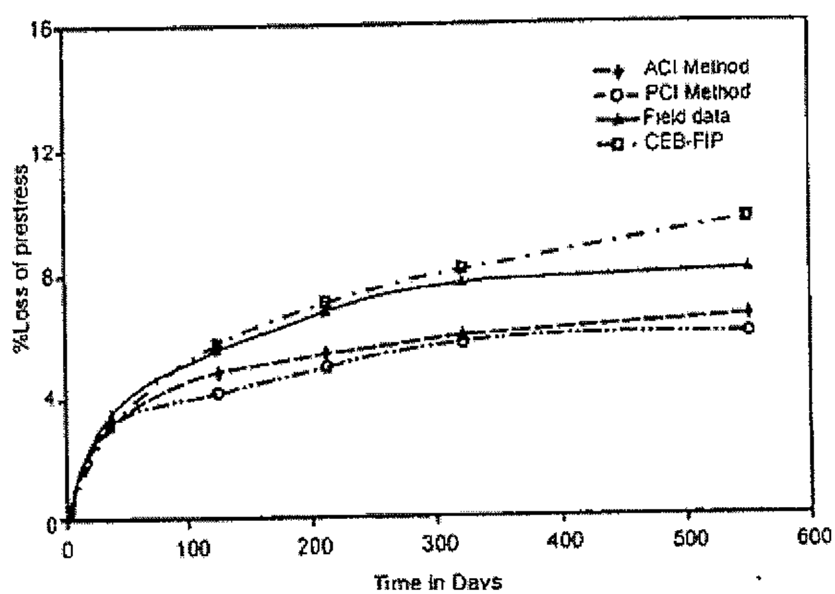

Figure 2-2 Comparision of Field Measure Losses with Design Code Estimates (Sridhar et., al)

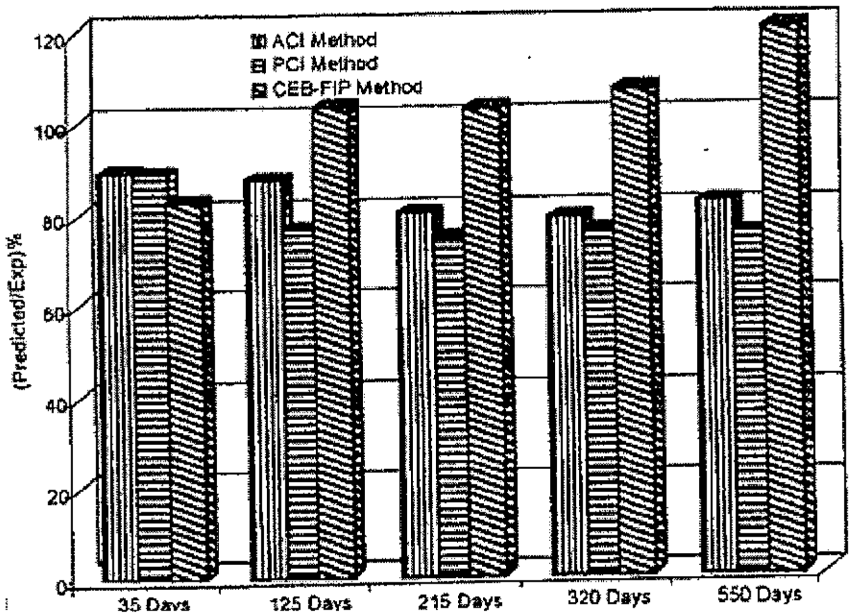

Figure 2-4 Predicted Losses from Design Code Estimates over Field Measured Losses. (Sridhar et.al., 2005)

Table 2-5 Measured and Estimated Percent Age Prestress Losses for B1.(Sridhar et.al., 2005)

No of days	Experimental Losses (Total Losses)	Losses from ACI method				Losses from PCI method				Loss from CEB-FIP method			
		Creep Loss	Shrinkage Loss	Relaxation Loss	Total Loss	Creep Loss	Shrinkage Loss	Relaxation Loss	Total Loss	Creep Loss	Shrinkage Loss	Relaxation Loss	Total Loss
50	4.58	2.31	0.83	0.81	3.95	1.62	0.81	0.81	3.24	2.26	0.83	0.81	3.9
100	6.03	2.78	1.22	0.89	4.89	2.02	1.08	0.89	3.99	3.23	1.3	0.89	5.42
200	7.1	3.2	1.6	0.99	5.79	2.41	1.8	0.97	5.18	4.4	1.81	0.99	7.2
300	7.35	3.41	1.78	1.02	6.21	2.68	2.57	1.02	6.27	5.07	2.1	1.02	8.19
400	7.5	3.55	1.89	1.05	6.49	2.86	2.66	1.05	6.57	5.5	2.29	1.05	8.84

2.3.13 Tadros , Ghali and Dilger, (1975)

Tadros, Ghali and Dilger (1975) proposed a numerical procedure for accurate determination of time dependent losses (creep, shrinkage, and relaxation) which further resulted in the precise determination of deflection. This paper emphasizes more on practical applications without the derivation of equations. Moreover, it takes into account the creep recovery factor and the relaxation reduction factor for the proper estimation of final losses. The three basic factors required for this technique, free shrinkage, creep coefficient and intrinsic relaxation of the prestressing tendon were assumed to be known. These were obtained from the equations derived by Branson et.al.,(1971). The two reduction parameters, μ creep recovery and Ψ relaxation reduction factor, used in the paper were obtained from a step by step numerical method proposed by Tadros et.al., (1974).

It was found that the general equation widely used for the prediction of prestress losses during that period overestimated the losses. Therefore, a new refined formula for the calculation of prestress losses was developed in a step by step design procedure taking into account the reduction parameters μ and Ψ resulting due to creep recovery and reduction in steel relaxation. Furthermore, the axial strain and the curvature of the concrete section were also found based on the time dependent strain at the prestress steel level. Although axial strain and curvature were not used in the calculation of losses, they were significant in the calculation of deflection and camber. The results from the proposed method were found to have good correlation with the existing experimental data. In addition, this paper also gives a procedure to determine the loss or gain in prestress and the deflection caused by superimposed sustained loads.

2.3.14 Tadros ,Ghali and Meyer, (1985)

Tadros, Ghali and Meyer (1985) proposed a simple method to determine time dependent prestress losses and eventually the time dependent deflection and camber in prestressed beams. They discussed more about the influence of creep, shrinkage of concrete, relaxation of prestressed steel and the presence of non-prestressed steel on time dependent deflection behavior of concrete. Empirical formulae and multipliers were developed for the prediction of time dependent deflections. The proposed method was a simple modification of the existing methods, it rationally accounted for the effects of the cross-sectional area and the location of the non-prestressed steel in the member section and the effects of cracking. The precision of the projected method was verified comparing its values with those of experimental results and other methods including the PCI Design method.

The proposed method included a five step calculation of deflection in prestressed concrete members. Multiplier formula tables were developed for the calculation of immediate and long-term defection of cracked and uncracked prestressed concrete members. The results of this formula were comparable with the results obtained using the PCI Design Hand book (1978) design. A new empirical formula suggested by Naaman(1982), shown in equation was used to calculate the moment of inertia and the curvature of the cracked section.

$$I_{cr} = \left(n_{ps}A_{ps}d_{ps}^2 + n_s A_s d_s^2\right)\left(1 - \sqrt{\rho_{ps}\rho_s}\right) \qquad in^4 \qquad \text{Eq 2-1}$$

Where,

n = modular ratio

A_{ps} = area of prestressed steel, in^2

A_s = area of non-prestressed reinforcement, in^2

d_{ps} = distance from the extreme compression fibre to the centroid of Prestressing steel

d_s = distance from the extreme compression fibre to the centroid of non-prestressed steel

ρ_{ps} = reinforcing ratio of prestressed steel.

ρ_{ps} = reinforcing ratio of non- prestressed steel.

This formula accounts for both prestressed and non-prestressed steel. It also takes into account the increase in the eccentricity due to cracking which when ignored causes overestimation of defection in cracked members. In addition to this, empirical equation for "tension stiffening" was also formulated where the uncracked concrete existing between the cracks contributed of the additional stiffness. Table 2-6 shows the comparison of the influence of prestress level and reinforcement on deflection between the proposed average time-dependent multipliers and PCI Design Hand Book multipliers. From the results it was concluded that the implication of empirical multipliers tend to the increase the time dependent deflection as opposed to the decrease in deflection predicted by the PCI Design Handbook method. It points out that although the presence of additional non-prestressed steel poses to increase the stiffness of the member, they actually tend to reduce the compression force developed in the concrete resulting in large prestress losses and high time-dependent defection. The experimental test data was found to have a close correlation between the observed deflections and those calculated by the proposed method.

Table 2-6 Influence of prestress level and reinforcement on deflection (Ghali et.al., 1985)

Input data:	Beam A*		Beam B		Beam C	
Prestress force just before release P_i Reinforcement For other data, see Example 1	405.0 14 strands		289.2 14 strands		289.2 10 strands plus 2 #8 bars	
Results:	Proposed method	PCI Design Handbook	Proposed method	PCI Design Handbook	Proposed method	PCI Design Handbook
Prestress force just after release, P_{c0}	375.3	364.0	274.6	260.3	269.9	260.3
Deflection at release due to:						
Prestress	−4.64	−4.49	−3.39	−3.15	−3.23	−3.15
Self weight	3.00	3.00	3.00	3.00	3.00	3.00
Total deflection at release	−1.64	−1.41	−0.39	−0.15	−0.23	−0.15
Time-dependent prestress loss, ΔP_c	−71.7	−54.6	−36.8	−39.0	−53.2	−39.0
Final "ultimate" time-dependent deflection due to:						
Prestress	−11.00	−11.00	−8.55	−7.72	−7.35	−5.39
Self weight	8.64	8.10	8.64	8.10	8.64	5.52
Superimposed dead load	1.20	1.44	1.20	1.44	1.20	0.96
Total time-dependent deflection	−1.16	−1.46	1.29	1.82	2.49	1.13
Geometric properties under full load:						
0.4 l Moment of inertia	20985	20985	4180	—	7400	—
0.4 l Prestress force eccentricity	11.78	11.78	16.06	—	15.15	—
Mid-span Moment of inertia	20985	20985	5160	5541	7850	3890
Mid-span Prestress force eccentricity	13.65	13.65	17.62	13.65	17.01	13.65
Live load curvature x 10^6:						
End section	0	0	0	0	0	0
0.4 l section	15.7	15.7	53.8	47.2	40.3	65.3
Midspan section	16.4	16.4	37.9	49.1	34.5	68.0
Live load deflection	1.21	1.21	2.76	3.61	2.73	5.00

*End section eccentricity used is $e = 4.29$ in., which is consistent with the PCI Design Handbook Example 3.2.8, but slightly different from $e = 3.79$ in. in Example 3.4.1.

Note: Forces are given in kips, and dimensions in inches. 1 kip = 4.45 kN; 1 in. = 25.4 mm.

CHAPTER III

3.0 MODELING USING EXCEL SHEET

3.1 Introduction

Individual excel spread sheets were developed and modeled to determine prestress losses and camber using the prevailing equations in AASHTO LRFD , ACI -209, PCI, NCHRP496. Specifically, the AASHTO LRFD prestress loss equations were incorporated with time dependent creep effects to determine the prestress losses, camber and defection. The sheets were developed on a Time Step basis with time varying day to day. The prestress losses and hence camber and deflection were computed daily from the Time Step approach. The sheets were also modulated for the various changes in the design criteria including the addition of fully tensioned prestressing strands at the top of the cross section and non-prestressed mild steel toward the bottom of the cross section (at midspan). Additionally, the modulus of elasticity (Ec) and creep coefficient (Cr) of concrete were varied between 80% 120% of the nominal values.

Table 3-1 displays the different cases for which the sheets have been modulated for the calculation of losses.

Table 3-1 Load cases that represent the variations in the material properties and design properties of the concrete bridge girder.

Case	Number of Top Strands	Number of Mild Steel Bars	% Creep Coefficient	% Elastic Modulus
Base Case	0	0	100%	100%
T 2	2	0	100%	100%
T4	4	0	100%	100%
MS 2.4	0	4	100%	100%
MS 5.0	0	5	100%	100%
CR-120	0	0	120%	100%
CR-80	0	0	80%	100%
E-120	0	0	100%	120%
E-80	0	0	100%	80%

The 'Base Case' represents the case without the inclusion of top prestressing strand and mild steel without on increase in the percentage of creep coefficient and Elastic modulus of concrete. The case T2 and T4 represents the case with addition of two and four top prestressing strands only. Losses predicted using 4 #7 mild steel bars with area of steel as 2.4in^2 was represented as MS 2.4 and losses predicted using 5#9 bars with area of steel of 5.0 in^2 as MS 5.0. Finally, the variation between 80 to 120 percentage of creep coefficient and elastic modulus without the top prestressing strand and mild steel have been indicated as CR-120, CR-80, E-120,E-80 correspondingly.

3.2 Girder Details

An AASHTO Type IV girder was used whose cross section details concrete properties and loading data are given in Table 3-1. The girder span, distance between c/c bearings and the spacing between the girders have been specified in the sheet. The transformed and composite cross section properties of the girder were calculated based on the specified concrete strength, gross section properties, prestressing strand details and the concrete deck details.

Table 3.1 Girder Properties and Prestressing/Non-Prestressing Steel Details

Girder Properties Strand Details	
Girder Type	**AASHTO Type IV Beam**
Span, ft	105
Girder Spacing ,in	96
Girder details	
Girder Height h ,in	54
Gross Cross Section Area Ag,in^2	789
yb,in.	24.73
Gross Moment of Inertia Ig,in^4	260,730
Girder unit weight,k/ft^3	0.15
Prestressing strands	
Number of bottom strands, N	34
Diameter of strands db, in.	0.5
Area of Prestressing strands Ap, in^2	5.2
Eccentricity at mid-span ep, in.	20.85
Strands initial stress fpi, ksi	202.5
Strands modulus of elasticity Es, ksi	28,500
Deck details	
Thickness of deck hf, in	8
Area of Deck Ad, $in.^2$	768
Deck unit weight, k/ft^3	0.15
Non-Prestressing Steel	
Number of bars, n	4#7 or 5#9
Modulus of Elasticity, Em, ksi	29,000

Concrete Properties

The one day and 28 day compressive strengths of concrete were specified as f'ci = 6 ksi and f'c=10 ksi respectively. The above strengths can be varied accordingly. The respective modulus of elasticity of concrete was calculated using the ACI 318-02 equation (ACI 310-02 Section 8.5.1).

$$E_c = 33w^{1.5}(f'c)^{0.5} \qquad \text{psi} \qquad \text{Eq 3-1}$$

where,

w = unit weight of concrete (150pcf);

f'c = concrete compressive strength in ksi.

The AASHTO Time Step method calculated the concrete strength (f'c) using the ACI 209 eq2-1 (ACI Committee 209 R, "Prediction of Creep, Shrinkage, and Temperature Effects in Concrete Structures," ACI 209R-92). The computed concrete strength is varied with time using the Eq 3-2.

$$(f'c)t = \frac{t}{\alpha + \beta t}(f'c)_{28} \qquad \text{ksi} \qquad \text{Eq 3-2}$$

where,

α and β= constants;

t = age of the concrete in days;

$(f'c)_{28}$ = specified 28 day compressive strength of concrete.

Loading Properties

The self weight of the girder was calculated from the beam area and unit weight, the slab weight from the slab unit weight and slab area. The super imposed dead load (SIDL) comes from weight of the parapets and guard rails plus the future wearing course. The SIDLs are equally distributed among the girders.

Prestressing and Non-Prestressing Steel

The prestressing strands are commonly concentrated at the tension flange of the bridge girder cross section. Under certain circumstances the prestressing strands are placed both in tension and compression flanges. Although this arrangement causes the center of gravity of steel (c.g.s) to move near the center of gravity of concrete (c.g.c) resulting in the decrease of lever arm, fully tensioned top strands can help mitigate the tensile stresses, potential tensile cracking, at the top fibers near the ends of the beams. [Linn and Burns (2004)]. The analyses with the addition of two and four top prestressing strands have been represented as cases T2 and T4 respectively. The spreadsheet is constructed to accommodate different strand patterns of varying diameter and number of strands. 270 Grade Low relaxation strands with an elastic modulus (Es) of 28,500ksi have been used in the girder for the prediction of prestress losses and mid span deflection.

Non-prestressed mild steel is not usually placed longitudinally through a prestressed beam. Where employed, the mild steel usually serves to boost moment capacity for the strength limit state.(Linn and Burns 2004) However, mild steel also provides benefit to prestressed sections by reducing losses due to elastic shortening and creep, it can also be useful for helping to control long-term camber and/or deflections. Mild steel bars of yield strength 50ksi and modulus of elasticity of 29000ksi was used. The excel sheet was modeled to accommodate or different arrangement of non-prestressing steel represented as cases MS 2.4 and MS 5.0 respectively.

3.3 Determination of Prestress losses using Recommended Equations

3.3.1 PCI Method [Zia et al (1979)]

ACI-ASCE Committee 423, by Zia, Preston et. al., (1979) developed equations for prestress losses with the intent to obtain reasonable values of losses for both pretensioned and post tensioned bridge girders with bonded and unbonded tendons. The equations were proposed for practical design application under normal design conditions.

This thesis program is limited to prestress losses for prestensioned members with bonded tendons.

Loss due to elastic shortening (ES):

$$ES = K_{es} \frac{E_s}{E_{ci}} f_{cir} \qquad \text{(ksi)} \qquad \text{Eq 3-3}$$

where,

K_{es} = 1.0 for pretensioned members;

K_{cir} = 0.9 for pretensioned members;

E_s = modulus of elasticity of prestressing tendons, ksi;

E_{ci} = modulus of elasticity of concrete at the time of prestressing, ksi;

f_{cir} = net compressive stress in concrete at the center of gravity of tendons immediately after the prestress has been applied to the concrete, ksi;

K_{cr} = stress in concrete at the center of gravity of the tendons due to prestressing force immediately prior to release;

f_g = stress in concrete at the center of gravity of tendons due to weight of structure at time prestress is applied.

Loss due to creep of concrete (CR):

$$CR = K_{cr} \frac{E_s}{E_{ci}} (f_{cir} - f_{cds}) \qquad \text{(ksi)} \qquad \text{Eq 3-4}$$

where,

K_{cr} = 2.0 for pretensioned members;

E_{ci} = modulus of elasticity of concrete at 28 days;

f_{cds} = stress in concrete at the center of gravity of tendons due to all superimposed permanent dead loads that are applied to the member after it has been prestressed, ksi.

In this equation, the creep loss of concrete is directly proportional to the net compressive strain in the concrete. This equation essentially adopts a creep coefficient of 2.0.

Loss due to shrinkage of concrete (SH):

$$SH = 8.2 \times 10^{-6} E_s \left(1 - 0.06 \frac{V}{S}\right)(100 - RH) \qquad \text{(ksi)} \qquad \text{Eq 3-5}$$

Where,

K_{sh} = 1.0 for prestensioned members;

$\frac{V}{S}$ = Volume to Surface ratio of the member, in;

RH = ambient relative humidity, %.

The equation was developed on an approximate value of basic ultimate shrinkage strain(ε_{sh}) for concrete of -550 x 10^{-6} and an ambient relative humidity correction factor of $1.5 - 0.0015RH$.

Loss due to relaxation of tendons (RE):

$$RE = [K_{re} - J(SH + CR + ES)]C \qquad \text{(ksi)} \qquad \text{Eq 3-6}$$

The values of K_{re} and J depend on the stress level and the material characteristic of the tendon as shown in Table 3-2.

Table 3.2 Vales of K_{re} and J (Zia et.al., 1979)

Type of Tendon	Kre	J
Grade 270 stress-relieved strand or wire	20,000	0.15
Grade 270 low-relaxation strand or wire	5,000	0.04
Grade 145 or 160 stress-relieved bar	6,000	0.05

The value of C is taken as unity for low-relaxation strands and stress-relieved strands initially stressed to 0.75 f_{pu} and 0.70 f_{pu} respectively, where f_{pu} is the ultimate strength of the prestressing tendon. The total losses (TL) are calculated by summing up the losses due to the individual components.

$$TL = ES + CR + SH + RE \qquad \text{(ksi)} \qquad \text{Eq 3-7}$$

The effective prestress at the level of the prestressing strands is calculated as below

$$f_{se} = f_{pi} - TL$$

where,

f_{pi} is the initial prestress at the time of loading.

These equations are applicable to prestressed concrete members with an extreme fiber compressive stress in the precompressed tensile zone under the full dead load conditions ranging from 350 psi to 1750 psi. This method is limited for maximum values of prestress losses as shown in Table 3-3.

Table 3-3 Maximum Prestress losses Recommended by ACI Committee

Type of Strand	Maximun Loss(psi)	
	Normal concrete	Lightweight Concrete
Stress-relieved strand	50,000	55,000
Low-relaxation strand	40,000	45,000

3.3.2 AASHTO-LRFD Refined Losses

("AASHTO-LRFD Bridge Design Specifications," Second Edition, Washington, DC (1998).)

In lieu of more detailed analysis, prestress losses in members constructed and prestressed in single stage, relative to the stress immediately before transfer, may be computed as the sum of individual loss components.

$\Delta f_{pT} = \Delta f_{pES} + \Delta f_{pSR} + \Delta f_{pCR} + \Delta f_{pR2}$ (ksi) Eq 3-8

where,

Δf_{pT} = total loss of prestress, 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.

This method is based on the following assumptions

a) Spans not greater then 250 feet

b) Normal density concrete

c) Strength in excess of 3.5 ksi at the time of prestress.

The loss due to elastic shortening in pretensioned members shall be taken as:

$$\Delta f_{pES} = \frac{E_p}{E_{ci}} f_{cgp} \qquad \text{(ksi)} \qquad \text{Eq 3-9}$$

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.

Loss due to creep of concrete may be taken as:

$$\Delta f_{pCR} = 12.0\, f_{cgp} - 7.0 \Delta f_{cdp} \geq 0 \qquad \text{(ksi)} \qquad \text{Eq 3-10}$$

where,

f_{cgp} = concrete stress at the center gravity of pretressing steel at transfer, ksi;

Δf_{cdp} = change in concrete stress at the center of gravity of prestressing steel due to permanent loads, with the exception of the self weight of the beam Values of

Δf_{cdp} should be calculated at the same section or at sections for which f_{cgp} is calculated, ksi.

The first term (f_{cgp}) in Eq 3-10 is based on a creep coefficient of approximately 1.7and a modular ratio of 7. The Eq 3-10 assumes a creep coefficient of zero for later permanent loads.

The loss due to shrinkage of concrete may be taken as:

$$\Delta f_{pSR} = (17.0 - 0.150H) \qquad \text{(ksi)} \qquad \text{Eq 3-11}$$

where,

H = the average annual ambient relative humidity (%)

The Eq-3-11 is roughly based on an ultimate concrete shrinkage strain of approximately -0.00042 and a modulus of elasticity of approximately 28,000ksi for prestressing strands.

The total relaxation at any time after transfer shall be taken as the sum of losses that take place at transfer and after transfer. At transfer for low-relaxation strand, initially stresses in excess of 0.50fpu

$$\Delta f_{pR1} = \frac{\log(24.0t)}{40.0}\left[\frac{f_{pj}}{f_{py}} - 0.55\right] f_{pj} \qquad \text{(ksi)} \qquad \text{Eq 3-12}$$

where,

t = estimated time in days from prestressing to transfer (days);

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

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

After transfer for low-relaxation strands may be taken as:

$$\Delta f_{pR2} = 6.0 - 0.12\Delta f_{pES} - 0.05(\Delta f_{pSR} + \Delta f_{pCR}) \qquad \text{(ksi)} \qquad \text{Eq 3-13}$$

3.3.3 AASHTO-LRFD Approximate Lump Sum Estimates of Time Dependent Losses.

("AASHTO-LRFD Bridge Design Specifications," Second Edition, Washington, DC (1998).)

Table 3-4 specifies the approximate lump sum estimates of time-dependent losses resulting from creep, shrinkage of concrete, relaxation of steel in prestressed, partially pretressed members. The losses due to elastic shortening should be added to the time-dependent losses to obtain the total losses.

The satisfaction of the following conditions is required:

a) pretensioned members are stressed after attaining a compressive strength of 3.5 fci,

b) members are made from normal weight concrete,

c) the concrete is either steam or moist-cured,

d) prestressing is by bars or strands with normal and low relaxation properties, and

e) the exposure and temperature conditions are average. More accurate estimates shall be used for unusual exposure.

This method reflects the values and trends obtained from a computerized time-step analysis of a large number of bridges and building members designed for a common range of variables a specified below:

a) ultimate concrete creep coefficient ranging from 1.6 to 2.4;

b) ultimate concrete shrinkage coefficient ranging from 0.0004 to 0.0006(in/in);

c) ambient relative humidity ranging from 40 to 100 percent;

d) moist-curing or steam curing of concrete, and

e) a partial prestressing ratio ranging from 0.2 to 1.0.

where,

$$PPR = A_{ps}f_{py} / \left(A_{ps}f_{py} + A_s f_y\right) \quad \text{ksi} \qquad \text{Eq 3-13}$$

PPR = partial prestressing ratio;

A_s = area of non-prestressed tension reinforcement,in^2;

A_{ps} = area of prestressed steel, in^2;

f_y = specified yield strength of reinforcing bars, ksi;

f_{py} = yield strength of prestressing steel, ksi.

Table 3-4 AASHTO-LRFD Lump-Sum Estimate of time dependent losses, ksi. ("AASHTO-LRFD Bridge Design Specifications," Second Edition, Washington, DC (1998).)

Type of Beam Section	Level	For wire and strands with fpu=235,250 or 270 ksi	For Bars with fpu=145 or 160ksi
Rectangular Beams, Solid Slab	Upper	29+4 PPR	19+6 PPR
	Bound		
	Average	26+4 PPR	
Box Girder	Upper	21+4 PPR	15
	Bound		
	Average	19+4 PPR	
I-Grirder	Average	33[1-0.15(fc-6)/6]+6PPR	19+6 PPR
Single T,Double T, Hollow Core and Voided Slab	Upper	39[1-0.15(fc-6)/6]+6PPR	31[1-0.15(fc-6)/6]+6PPR
	Bound		
	Average	33[1-0.15(fc-6)/6]+6PPR	

3.3.4 NCHRP 496 Detailed Prestress Losses [Tadros et al (2003)]

The detailed losses procedure proposed by the National Cooperative Highway Research Program (NCHRP) Report 496, by Tadros, et al(2003) make use of the aging coefficient approach for the computation of losses between the transfer and casting of the decks. The method covers the composite action between the precast concrete girders and cast-in-place deck slab. The prestress losses are computed in four stages:

a) Instantaneous prestress loss due to elastic shortening at transfer, Δf_{pES}

b) Long-term prestress losses due to shrinkage of concrete, $(\Delta_{pSR})_{id}$, and creep of concrete, $(\Delta f_{pCR})_{id}$ and relaxation of prestressing strands, $(\Delta f_{pR2})_{id}$, between the time of transfer and jus t before deck placement.

c) Instantaneous prestress gain due to the placement of deck weight and SIDL, Δf_{pED}.

d) Long-term prestress losses between the time of deck placement and the final service life of the structure, due to shrinkage of the girder, $(\Delta f_{pSD})_{df}$ creep of the girder, $(\Delta f_{pCD1} + \Delta f_{pCD2})_{df}$, relaxation of prestressing strands, $(\Delta f_{pR3})_{df}$, and shrinkage of deck concrete, $(\Delta f_{pSS})_{df}$.

The total prestress losses in the prestensioned bridge girder is given by:

$$\Delta f_{pT} = \Delta f_{pES} + \left(\Delta f_{pSR} + \Delta f_{pCR} + \Delta f_{pR2}\right)_{id} - \Delta f_{pED} + \left(\Delta f_{pSD} + \Delta f_{pCD1} + \Delta f_{pCD2} + \Delta f_{pR3} - \Delta f_{pSS}\right)_{df} \quad \text{ksi} \qquad \text{Eq 3-14}$$

The loss due to elastic shortening is given by:

$$\Delta f_{pES} = n_i f_{cgp} \quad \text{ksi} \qquad \text{Eq 3-15}$$

where,

n_i = E_p/E_{ci};

f_{cgp} = concrete stress at the center of gravity of the prestressing force;

$$= P_i\left(\frac{1}{A_{ti}} + \frac{e_{pti}^2}{I_{ti}}\right) - \frac{M_g e_{pti}}{I_{ti}} \quad \text{ksi} \qquad \text{Eq 3-16}$$

P_i = initial prestressing force just before release, kips;

A_{ti} = initial transformed area of cross section at release, in^2;

I_{ti} = initial transformed moment of inertia, in^4;

e_{pti} = initial eccentricity of strands with respect to the transformed cross section, in;

Mg = maximum moment due to self weight of the girder, kip-in.

The long term prestress losses due to shrinkage, creep, and relaxation (for the time between the transfer and placement of deck slab) are determined in three different stages. The net section properties of the non-composite section are used for the calculation of the prestress losses.

The prestress loss due to shrinkage is given by:

$$\Delta f_{pSR} = \varepsilon_{bid} E_p K_{id} \qquad \text{ksi} \qquad \text{Eq 3-17}$$

where,

ε_{bid} = concrete shrinkage strain of the girder between transfer and deck placement;

K_{id} = transformed section age-adjusted effective modulus of elasticity factor, for adjustment between time of transfer and deck placement;

$$= \frac{1}{1 + n_i \rho_n \alpha_n \left(1 + \chi \psi_{bif}\right)} \qquad \text{Eq 3-18}$$

ψ_{bif} = girder creep coefficient between the transfer and final;

χ = aging coefficient that accounts for the variability of concrete stress with time, and may be considered constant for all concrete members at age 1 to 3 days. (0.7 average);

ρ_n = tensile reinforcement ratio for the initial section;

= A_{ps}/A_n

A_{ps} = area of the prestressing strands, in^2;

A_n = area of net concrete section, in^2;

α_n = factor for initial net section properties;

$$= \left(1 + \frac{A_n e_{pn}^2}{I_n}\right) \qquad \text{Eq 3-19}$$

e_{pn} = eccentricity of the prestressing strands with respect to net concrete section, in;

I_n = moment of inertia of the net concrete section, in^4.

The loss due to concrete creep is given by:

$$\Delta f_{pCR} = n_i f_{cgp} \psi_{bid} K_{id} \qquad \text{(ksi)} \qquad \text{Eq 3-20}$$

where,

ψ_{bid} = girder creep coefficient between the transfer and deck placement.

Prestress loss due to relaxation of prestressing strands is given by:

$$\Delta f_{pR2} = \phi_i L_i K_{id} \qquad \text{(ksi)} \qquad \text{Eq 3-21}$$

where,

ϕ_i = reduction factor that reflects he steady decrease in strand prestressing due to creep and shrinkage of concrete;

$$= 1 - \frac{3(\Delta f_{pSR} + \Delta f_{pCR})}{f_{po}} \qquad \text{(ksi)} \qquad \text{Eq 3-22}$$

$$L_i = \left[\log\left(\frac{24t_d + 1}{24t_i + 1}\right) \Big/ 45\right] [f_{po}/f_{py} - 0.55] f_{po} \qquad \text{(ksi)} \qquad \text{Eq 3-23}$$

= intrinsic relaxation loss between transfer and placement of deck slab;

f_{po} = stress in the prestressing strands just after release, ksi;

t_d = age of the concrete after the placement of deck, days;

t_i = age of the concrete at the time of transfer, days.

Since the relaxation loss of the low-relaxation strands is very small ranging from 1.5 ksi to 4.0 ksi, a constant value of 2.4ksi is assumed. This value is equally split between the two time stages, transfer to the deck placement and deck placement to final. The deck weight on the non-composite section and the superimposed dead loads on the composite section cause instantaneous elastic prestress gain. The method states that proper accounting of elastic losses produces more refined results regardless of the use of transformed or gross section properties. Separate calculations of elastic losses at transfer and gain increments at the different loading stages are required when gross section properties are used. The explicit calculation of this prestress gain is not necessary provided the stress analysis using the transformed cross section properties is automatically takes into account for the change in stress in the steel component.

The long term prestress losses between the time of the deck placement and the final stage are determined in five stages: the shrinkage, creep of concrete, relaxation of prestressing strands, concrete deck shrinkage between the time of deck placement and the final service life of the structure. These long term losses are calculated using composite cross section properties of the girder.

The loss due to shrinkage of the concrete is given by:

$$\Delta f_{pSD} = \varepsilon_{bdf} E_p K_{df} \qquad \text{(ksi)} \qquad \text{Eq 3-24}$$

where,

ε_{bdf} = concrete shrinkage strain of the girder between deck placement and final;

K_{df} = the transformed cross section factor based on the age-adjusted effective modulus of elasticity of concrete. It is used to adjust the small gain in steel stress stress resulting from the continuous interaction between concrete

and

steel components of the member, between the time of deck placement and the final time;

$$= \frac{1}{1 + n_i \rho_{nc} \alpha_{nc} \left(1 + \chi \psi_{bif}\right)} \qquad \text{Eq 3-25}$$

ρ_{nc} = tensile reinforcement ratio for the net composite section;

= A_{ps}/A_{nc}

A_{nc} = net area of the composite cross section, in^2;

α_{nc} = factor for the net composite section properties;

$$= \left(1 + \frac{A_{nc} e_{pnc}^2}{I_{nc}}\right) \qquad \text{Eq 3-26}$$

e_{pnc} = eccentricity of the prestressing strands with respect to net composite concrete section at service, in;

I_{nc} = moment of inertia of the net composite concrete section, in^4.

The prestress loss due to the creep of the composite girder cross section caused by initial prestressing and self-weight is given by:

$$\Delta f_{pCD1} = n_i f_{cgp} \left(\psi_{bif} - \psi_{bid}\right) K_{df} \qquad \text{(ksi)} \qquad \text{Eq 3-27}$$

The prestress loss due to creep of the composite section caused by deck weight and superimposed dead loads is given by:

$$\Delta f_{pCD2} = n\Delta f_{cdp}\psi_{bdf}K_{df} \qquad \text{(ksi)} \qquad \text{Eq 3-28}$$

n $= E_p/E_c$;

E_c = modulus of elasticity of concrete at the time of the placement of the deck superimposed dead loads, ksi;

Δf_{cdp} = change in concrete stress at the center of prestressing strands due to long-term losses between transfer and deck placement caused by the deck weight on non-composite section and superimposed dead load on composite section;

Ψ_{bdf} = ultimate creep coefficient at the time of the placement of deck.

The relaxation loss due to prestressing steel between the time of deck placement and final time can either be calculated in a similar manner or a constant value of 2.4ksi may be assumed for thc total loss due to steel relaxation.

The prestress gain due to shrinkage of deck in the composite section is given by:

$$\Delta f_{pSS} = n\Delta f_{cdf}K_{df}\left(1+\chi\psi_{bdf}\right) \qquad \text{(ksi)} \qquad \text{Eq 3-29}$$

where,

Δf_{cdf} = change in concrete stress at the centroid of prestressing strands due the shrinkage of deck concrete, ksi;

$$= \left[\frac{\varepsilon_{ddf}A_d E_{cd}}{A_{nc}\left(1+\chi\psi_{ddf}\right)}\right] - \left[\frac{\varepsilon_{ddf}A_d E_{cd}e_{dc}e_{pnc}}{I_{nc}\left(1+\chi\psi_{ddf}\right)}\right] \qquad \text{(ksi)} \qquad \text{Eq 3-30}$$

ε_{ddf} = shrinkage strain of the deck concrete between placement and the final time, in/in;

e_{dc} = deck eccentricity with respect to transformed composite section at the time of application of superimposed dead loads, in (always taken as negative);

ψ_{ddf} = deck creep coefficient at the time of deck loading.

3.3.5 NCHRP 496 Approximate Prestress Losses [Tadros et al (2003)]

The approximate method was proposed based on a parametric study of prestressed concrete bridges. The total long term prestress losses is given by:

$$\Delta f_{pLT} = 10.0\left(\frac{f_{pi}A_{ps}}{A_g}\right)\gamma_h\gamma_{st} + 12\gamma_h\gamma_{st} + 2.5 \qquad \text{(ksi)} \qquad \text{Eq 3-31}$$

where,

γ_h = humidity correction factor;

$$= 1.7 - 0.01H \qquad \text{Eq 3-32}$$

γ_{st} = concrete strength correction factor;

$$= \frac{5}{1 + f'_{ci}} \qquad \text{Eq 3-33}$$

f_{pi} = initial prestressing stress in steel, ksi;

A_{ps} = area of prestressing steel, in^2;

A_g = gross cross sectional area of girder cross section, in^2.

3.4 Equations Using Time Dependent Creep Effects

This method initially follows a similar procedure as the Time Step prestress loss method. The time was divided into intervals where the duration of each time interval is made larger as the age of concrete increases. The prestress losses and the stresses in the concrete at the end of each interval were calculated. The calculated stresses include the elastic stresses due to prestress, gravity loads and sustained loads along with the time dependent effects due to creep of concrete. After slab casting, the whole composite section of concrete is treated as an elastic material. The elastic stresses and the stresses due to creep strain in concrete were individually calculated. The proposed AASHTO LRFD Time Step method was developed based on the prevailing AASHTO LRFD time dependent coefficients for the determination of prestress losses and deflection in prestressed concrete bridges.

3.4.1 AASHTO LRFD Time Step method

This method makes use of the AASHTO LRFD time dependent correction factors and the equations for the prediction of losses. Prestress losses and the camber and deflection of the girder have been determined on a day to day basis from a time period of t = 1 day to t = 10,000 days.

3.4.1.1 Creep Strain and Creep Coefficient

The sustained and the gravity loads at the initial stages of loading result in the elastic strain. Creep strain is the additional strain that has developed with time due to creep of concrete. The ratio of the creep strain at time t to the above elastic strain at initial loaded time t_i is defined as the creep coefficient. ($\psi_{t,ti}$)

Creep coefficient $\psi_{t,ti}$ = creep strain / elastic strain

The creep coefficient is calculated using the AASHTO LRFD equation 5.4.2.3.2-1 (AASHTO LRFD Manual)

$$\psi_{(t,ti)} = 3.5\, K_c K_f \left(1.58 - \frac{H}{120}\right) t_i^{-0.118} \left(\frac{(t-ti)^{0.6}}{10.0 + (t-ti)^{0.6}}\right) \qquad \text{Eq 3-34}$$

where,

K_c = factor for the effect of volume to surface ratio;

$$= \left[\frac{\dfrac{t}{26\, e^{0.36(V/S)} + t}}{\dfrac{t}{45 + t}}\right]\left[\frac{1.80 + 1.77 e^{-0.54(V/S)}}{2.587}\right] \qquad \text{Eq 3-35}$$

K_f = factor for the effect of concrete strength;

$$= \frac{1}{0.67 + \left(\dfrac{f'_c}{9}\right)} \qquad \text{Eq 3-36}$$

H = relative humidity in percent.

We can see here that the creep coefficient is an increasing function of time. The creep coefficient $\Psi_{(91\text{-}90)}$ after slab cast is also calculated at 91 days, i.e. after slab casting.

3.4.1.2 Effective modulus or Reduced Modulus of Elasticity

The effective or the reduced modulus of elasticity combines the effect due to elastic strain and creep of concrete as an elastic deformation on concrete section. At a constant loading the elastic plus creep strain is calculated as ($1 + \psi_{t,ti}$) times the elastic strain. The effective modulus of elasticity is calculated using the formula:

E_{eff} = [1/Eci + [ψ(t,t-1) - ψ(t-1)]/E(t) + [ψ(t+2,t) - ψ(t+1,t)]/E(t+2) + …]ksi Eq 3-37.

where,

Eci = Modulus of Elasticity of Concrete

Ψ(t) = Creep coefficient at time t

The effective modulus is reduced due to creep effects in the beam and the transformed cross section properties are further calculated based on this reduced effective modulus. Thus the above equation is used for the time dependent analysis due to the effects of all the loads such as: initial prestress, self weight, deck weight, superimposed dead loads and other live loads. The modular ratio n is calculated as given below:

$n = Es/E_{eff}$. Eq 3-38

where, Es = modulus of elasticity of prestressing strands;

E_{eff} = Effective modulus of elasticity of concrete.

The effect due to time dependent creep has been incorporated by using the modular ratio n in the calculation of transformed cross section properties.

3.4.1.3 Shrinkage strain

$$\varepsilon_{sh} = -k_s k_h \left(\frac{t}{35.0+t}\right) 0.51 * 10^{-3} \qquad \text{Eq 3-39}$$

Where k_s is the size factor given by:

$$k_s = \left[\frac{\dfrac{t}{26\theta^{0.36(V/S)}+t}}{\dfrac{t}{45+t}}\right]\left[\frac{1064-94(V/S)}{923}\right] \qquad \text{Eq 3-40}$$

k_h = humidity factor specified in table 1 (AASHTO-LRFD manual)

The relative humidy (RH) , volume to surface ratio(V/S) of the girder and are specified in the Spread sheet.

3.4.1.4 Prestress Loss Equations

The prestress losses were found using the creep and shrinkage strains for AASHTO LRFD given in section 3.4.1.2.

Elastic Shortening

The Elastic Shortening ES is calculated as given below:

$$ES = n * f_{cir} \qquad \text{Eq 3-41}$$

where,

n = modulus of elasticity of concrete;

f_{cir} = net compressive stress in concrete at the center of gravity of tendons immediately after the prestress has been applied to the concrete, ksi.

The f_{cir} is calculated in two stages, from initial to slab casting and after slab casting. The calculation of f_{cir} for the loading stage after slab casting, utilizes the composite transformed cross section properties of concrete calculated with the use of effective modulus. The Elastic shortering loss at the day of slab cast, (ie at 90 days) is reduced due to the creep effect in the beam.

Creep loss

The creep loss is a function of various time dependent factors including the volume to surface ratio, relative humidity and age of concrete at the time of loading. Incremental creep strains are computed daily using the formula:

$$\Delta\varepsilon_{cr}(t) = [f_{cir}(t-1)][\psi(t,1)] - [\psi(t-1,1)] \qquad \text{Eq 3-42}$$

where,

$\Delta\varepsilon_{cr}(t)$ = incremental ceep strain at time t;

$\Psi_{(t,ti)}$ = creep coefficient calculated using Eq 3-34.

The creep loss is calculated as given below:

$$CR_{(t)} = CR_{(t-1)} + \Delta\varepsilon_{cr(t)} * 0.0285 \qquad \text{Eq 3-43}$$

Shrinkage Loss

The shrinkage of concrete also depends on the volume to surface ratio and relative humidity, but is independent of the of the loading and is caused primarily due to shrinkage of cement paste. The shrinkage loss is calculated as given below:

$$SH = E_s * \epsilon_{sh} \qquad \text{Eq 3-44}$$

Where E_s = modulus of elasticity of prestressing steel;

ϵ_{sh} = shrinkage strain is calculated as given by Eq 3-39.

Relaxation Loss

Low relaxation strands are most widely used in prestressed girders. The relaxation loss is based of the formula in the AASHTO LRFD equation:

$$RE = \frac{\log(24.0 * t)}{40}\left\{ {f_{pi}}/{f_{py}} - 0.55 \right\} f_{pj} \qquad \text{Eq 3-44}$$

Where, f_{pi} = initial prestress at transfer

f_{py} = specified yield strength of prestressing steel

Total Loss TL = ES +CR + SH + CR +RE.

The total loss is calculated by summing up the losses for each day and the prestress, fse and the corresponding prestress force Fse is calculated for each stage of loading.

3.4.1.5 Concrete stresses and strains:

The top fiber stress, (f_t) and bottom fiber stresses (f_b) have been calculated for each day of loading. The corresponding strains $\epsilon_t = f_t/E_{eff}$, $\epsilon_b = f_b/E_{eff}$ are also calculated for the respective stresses. The models presume that the slab is cast after 90 days and after 90 days the slab is assumed to be cast on a beam transformed to an elastic concrete. The composite transformed cross section properties of concrete are calculated after 90 days of slab cast.

After slab cast the additional strain due to creep is calculated by applying a resultant force on the composite cross section of the beam. The resultant force is calculated from the change in stress at time (t) due to creep in the beam. The final concrete strains at t= 10,000 days includes: the strain due to the prestressing force, gravity loads, slab wt , super imposed live load (S.D.L) , live loads and the additional strain due to time dependent creep.

3.4.1.6 Camber/ Deflection

The camber and deflection of the beam due to prestress and gravity loads were directly computed from the curvature due to concrete strains using moment area method. After slab cast the camber due to the additional creep strain is also included in the final deflection.

The camber was also calculated using the PCI Design Handbook method (as shown in Table 3-5) in which simple span multipliers were used for the calculation of long term deflection and camber. The camber/deflection calculated using the above method was compared with the deflection values arrived when using the proposed method.

Table 3-5 PCI Design Handbook suggested simple span multipliers for the estimation of long-term camber and deflection for typical presstressed members

Loading stage	Without Composite Topping	With Composite Topping
At Errection:		
(1) Deflection (downward) component-apply to the elastic deflection due to the member weight at release of prestress	1.85	1.85
(2) Camber (upward) component-apply to the elastic camber due to prestress at the time of release of prestress	1.80	1.80
Final:		
(3) Deflection (downward) component-apply to the elastic deflection due to the member weight at release of prestress	2.70	2.40
(4) Camber (upward) component-apply to the elastic camber due to prestress at the time of release of prestress	2.45	2.22
(5) Deflection (downward)-apply to the elastic deflection due to superimposed dead load only	3.00	3.00
(6) Deflection (downward)-apply to the elastic deflection caused by the composite topping	—	2.30

CHAPTER IV

4.0 PRESENTATION OF RESULTS

4.1 Tabulated results for Prestress loss and Camber/Deflection

The following section presents tabulated values for prestress losses and deflection that were developed using each of the prestress loss prediction methods as explained in chapter three. The tables have been generated for different cases at two loading stages, stage prior to slab casting (ie 90 days) and the stage after 10 years of service. The tables compare values for 9 different cases which have been explained in Table 3-1.

Table 4-1 displays the values for prestress losses for a loading stage after 10 years, generated using the PCI (Zia. et. al) method. The camber and deflection was also calculated using the PCI Design Handbook method. Tables 4-2 lists the values for prestress losses after 10 years calculated using the AASHTO LRFD refined method. Tables 4-3 and 4-4 summarize the values for prestress losses for the loading stages, prior to slab casting and after 10 years, calculated using the NCHRP 496 detailed method. The losses calculated using the proposed AASHTO Time Step method has been presented in Tables 4-5 and 4-6.

Table 4-1 Prestress losses at mid span and mid span deflection after 10 years using PCI Design Handbook method

Case	Number of Top Strands	Number of Mild Steel Bars	% Creep Coefficient	% Elastic Modulus	ES	CR	SH	RE	TL	Camber/Deflection
					(ksi)	(ksi)	(ksi)	(ksi)	(ksi)	(in)
Base Case	0	0	100%	100%	11.47	5.80	6.41	4.05	27.74	-0.95
T 2	2	0	100%	100%	10.81	6.14	6.41	4.07	27.43	-0.46
T4	4	0	100%	100%	10.41	6.71	6.41	4.06	27.59	0.01
MS 2.4	0	4	100%	100%	11.23	4.21	6.95	4.10	26.49	-0.73
MS 5.0	0	5	100%	100%	11.00	4.01	6.95	4.12	26.09	-0.70
CR-120	0	0	120%	100%	11.47	6.96	6.41	4.01	28.85	-0.95
CR-80	0	0	80%	100%	11.47	4.64	6.41	4.10	26.62	-0.95
E-120	0	0	100%	120%	9.70	4.87	6.41	4.16	25.14	-0.79
E-80	0	0	100%	80%	14.04	7.15	6.41	3.90	31.50	-1.15

Table 4-2 Prestress losses at mid span after 10 years using AASHTO LRFD refined equations.

Case	Number of Top Strands	Number of Mild Steel Bars	% Creep Coefficient	% Elastic Modulus	ES	CR	SH	RE	TL
					ksi	ksi	ksi	ksi	ksi
Base Case	0	0	100%	100%	11.47	13.77	7.25	3.57	36.06
T 2	2	0	100%	100%	10.81	13.48	7.25	3.67	35.21
T4	4	0	100%	100%	10.41	13.57	7.25	3.71	34.94
MS 2.4	0	4	100%	100%	11.23	13.41	7.25	3.62	35.51
MS 5.0	0	5	100%	100%	11.00	13.07	7.25	3.66	34.98
CR-120	0	0	120%	100%	11.47	18.31	7.25	3.35	40.37
CR-80	0	0	80%	100%	11.47	9.23	7.25	3.80	31.75
E-120	0	0	100%	120%	9.70	13.94	7.25	3.78	34.66
E-80	0	0	100%	80%	14.04	13.51	7.25	3.28	38.08

Table 4-3 Prestress losses at mid span prior to slab casting; at 90 days using NCHRP 496 detailed equations.

Case	Number of Top Strands	Number of Mild Steel Bars	% Creep Coefficient	% Elastic Modulus	ES	CR	SH	RE	TL
					(ksi)	(ksi)	(ksi)	(ksi)	(ksi)
Base Case	0	0	100%	100%	11.47	9.55	6.17	1.20	28.38
T 2	2	0	100%	100%	10.81	9.12	6.25	1.20	27.38
T4	4	0	100%	100%	10.41	8.87	6.31	1.20	26.79
MS 2.4	0	4	100%	100%	11.23	9.34	6.17	1.20	27.94
MS 5.0	0	5	100%	100%	11.00	9.15	6.17	1.20	27.52
CR-120	0	0	120%	100%	11.47	11.28	6.08	1.20	30.03
CR-80	0	0	80%	100%	11.47	7.76	6.26	1.20	26.69
E-120	0	0	100%	120%	9.70	8.28	6.33	1.20	25.51
E-80	0	0	100%	80%	14.04	11.25	5.94	1.20	32.42

Table 4-4 Prestress losses at mid span after 10 years using NCHRP 496 detailed equations.

Case	Number of Top Strands	Number of Mild Steel Bars	% Creep Coefficient	% Elastic modulus	ES (ksi)	CR (ksi)	SH (ksi)	RE (ksi)	TL (ksi)
Base Case	0	0	100%	100%	5.37	10.58	14.97	2.40	33.32
T 2	2	0	100%	100%	5.41	9.76	14.89	2.40	32.47
T4	4	0	100%	100%	5.62	9.28	14.81	2.40	32.11
MS 2.4	0	4	100%	100%	5.19	10.31	14.97	2.40	32.87
MS 5.0	0	5	100%	100%	5.02	10.05	14.97	2.40	32.43
CR-120	0	0	120%	100%	5.37	12.40	14.52	2.40	34.69
CR-80	0	0	80%	100%	5.37	8.66	15.49	2.40	31.92
E-120	0	0	100%	120%	4.54	9.31	14.85	2.40	31.09
E-80	0	0	100%	80%	6.57	12.20	14.97	2.40	36.13

Table 4-5 Prestress losses at mid span and mid span deflection prior to slab casting, at 90 days using AASHTO Time Step equations.

Case	Number of Top Strands	Number of Mild Steel Bars	% Creep Coefficient	% Elastic Modulus	ES	CR	SH	RE	TL	Camber/Deflection
					(ksi)	(ksi)	(ksi)	(ksi)	(ksi)	(in)
Base Case	0	0	100%	100%	11.34	6.14	6.08	4.78	28.35	-2.87
T 2	2	0	100%	100%	10.69	5.80	6.08	4.78	27.36	-2.56
T4	4	0	100%	100%	10.30	5.60	6.08	4.78	26.77	-2.26
MS 2.4	0	4	100%	100%	11.29	6.04	6.08	4.78	28.19	-2.83
MS 5.0	0	5	100%	100%	11.06	5.94	6.08	4.78	27.87	-2.81
CR-120	0	0	120%	100%	11.34	7.34	6.08	4.78	29.54	-3.01
CR-80	0	0	80%	100%	11.34	4.93	6.08	4.78	27.14	-2.72
E-120	0	0	100%	120%	9.59	5.21	6.08	4.78	25.66	-2.47
E-80	0	0	100%	80%	13.88	7.48	6.08	4.78	32.22	-3.41
CR-80+T4	4	0	80%	100%	10.30	4.50	6.08	4.78	25.66	-2.14
CR-80+MS 5.0	0	5	80%	100%	11.06	4.77	6.08	4.78	26.70	-2.66
E-120+T4	4	0	100%	120%	8.69	4.74	6.08	4.78	24.30	-1.95
E-120+MS 5.0	0	5	100%	120%	9.43	5.07	6.08	4.78	25.36	-2.43

Table 4-6 Prestress losses at mid span and mid span deflection after 10 years using AASHTO Time Step equations.

Case	Number of Top Strands	Number of Mild Steel Bars	% Creep Coefficient	% Elastic modulus	ES	CR	SH	RE	TL	Camber/Deflection
					(ksi)	(ksi)	(ksi)	(ksi)	(ksi)	(in)
Base Case	0	0	100%	100%	6.36	8.68	12.93	7.72	35.69	-1.32
T 2	2	0	100%	100%	6.33	8.32	12.93	7.72	35.30	-0.85
T4	4	0	100%	100%	6.47	8.14	12.93	7.72	35.26	-0.41
MS 2.4	0	4	100%	100%	6.42	8.40	12.93	7.72	35.47	-1.19
MS 5.0	0	5	100%	100%	6.28	8.13	12.93	7.72	35.06	-1.09
CR-120	0	0	120%	100%	6.36	10.39	12.93	7.72	37.40	-1.39
CR-80	0	0	80%	100%	6.36	6.97	12.93	7.72	33.98	-1.23
E-120	0	0	100%	120%	5.39	7.30	12.93	7.72	33.34	-1.16
E-80	0	0	100%	80%	7.75	10.70	12.93	7.72	39.10	-1.51
CR-80+T4	4	0	80%	100%	6.47	6.53	12.93	7.72	33.65	-0.41
CR-80+MS 5.0	0	5	80%	100%	6.27	6.55	12.93	7.72	33.47	-1.04
E-120+T4	4	0	100%	120%	5.47	6.84	12.93	7.72	32.96	-0.38
E-120+MS 5.0	0	5	100%	120%	5.37	6.91	12.93	7.72	32.93	-0.99

CHAPTER V

5.0 Discussion of results

5.1Introduction

This chapter displays the effects on prestress losses and camber / deflection by varying the material properties and design properties of the prestress concrete bridge beams. The material properties include the concrete creep and modulus of elasticity where as the variation in design properties represent the addition of more top prestressing strands and mild steel in prestress concrete bridge beams. Comparisons have also been made between the values of prestress losses (with respect to base case) predicted using the proposed AASHTO Time Step method , the AASHTO LRFD refined method, NCHRP 496 detailed method and the PCI Design Handbook method.

5.2 Discussion of results for AASHTO Time Step Method

The prestress losses predicted using the proposed AASHTO Time Step equations for the different cases are shown in Tables 4-5 and 4-6. Charts have been developed to compare the values for the different cases for the two loading stages. The effects of the variation of the concrete material properties and the design properties are discussed in the following sections.

5.2.1 Effects of addition of Top Prestressing Strand

The effects of the addition of top prestressing in the prestress concrete bridges were analyzed by adding two and four top strands to the beam cross section. The Figures 5-1 and 5-2 show the charts for prestress losses at mid span and mid span deflection over time developed using the proposed AASHTO Time Step equations. The addition of the top strands reduces the eccentricity of the prestressing force and thereby causes reduction in prestress loss when compared to the base case with no top prestressing strands.

The prestress loss values from Tables 4-5 and 4-6 show that the addition of four top strands lowered the long term creep losses by approximately 6.2% We can also observe that although the addition of four top prestressing strands reduce losses by 1.2%, they cause a significant reduction in the long term camber. Figure 5-2 shows that the addition of top prestressing strands for the case T4 lowers the long time camber by approximately 69% from the base case. The results clearly indicate that the addition of top strands in prestressed concrete beams cause a significant decrease in initial and long term camber.

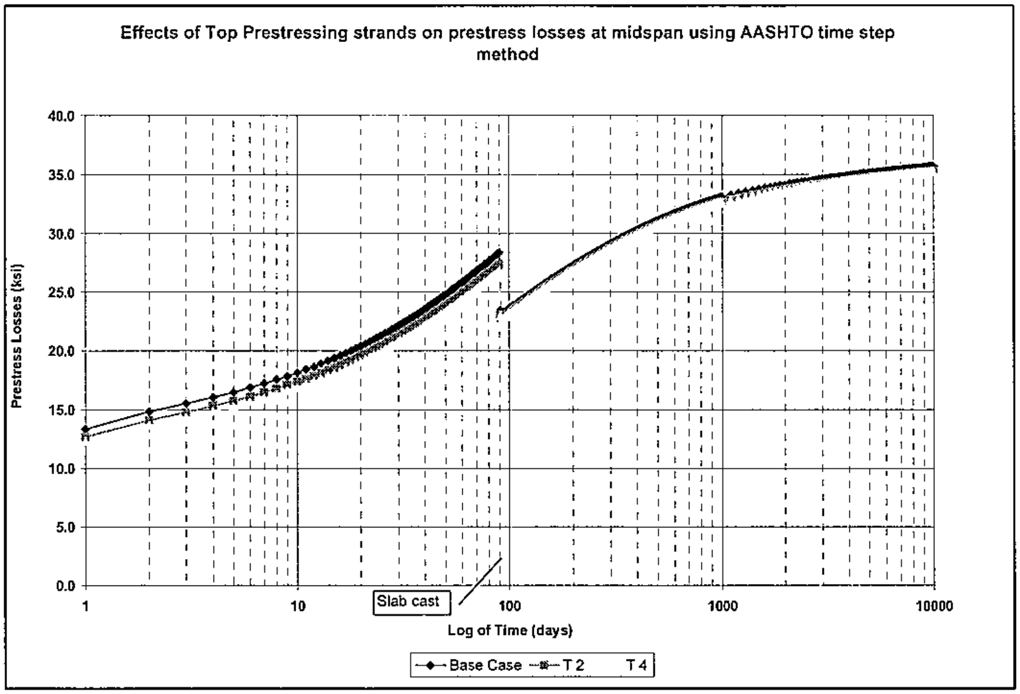

Figure 5-1 Effects of Top Prestressing strands on prestress losses at mid span using AASHTO Time Step method.

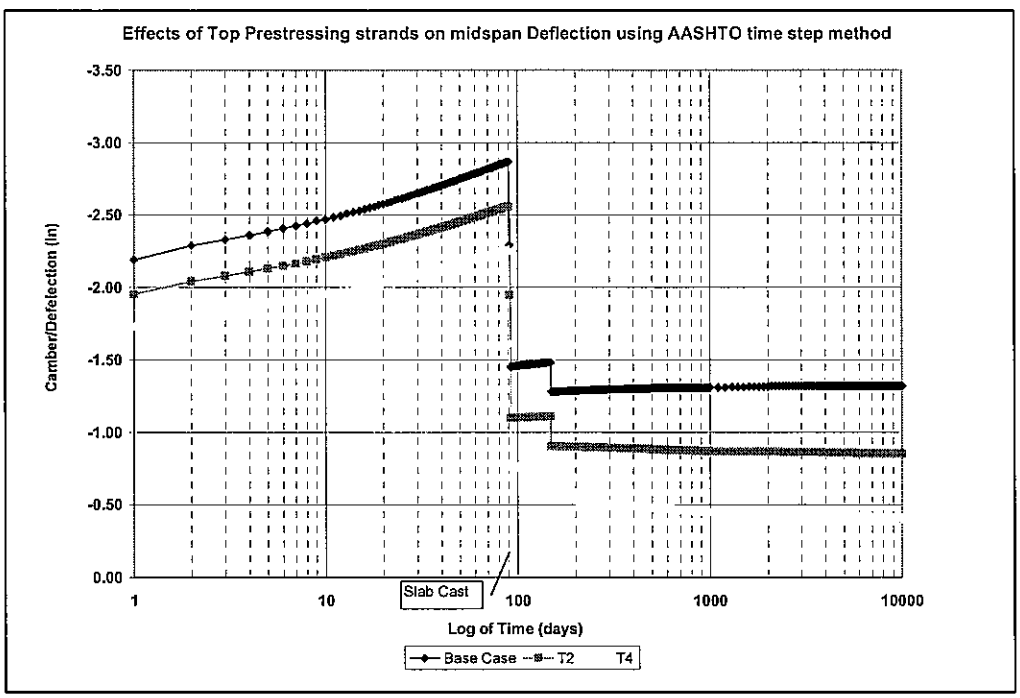

Figure 5-2 Effects of Top Prestressing strands mid span Deflection using AASHTO Time Step method.

5.2.2 Effect of addition of Mild Steel

The Figures 5-3 and 5-4 clearly show that the effects of the addition of mild steel on prestress losses at mid span and mid span deflection. The addition of mild steel of area $2.4 in^2$ and $5.0 in^2$ reduces the long term camber by 3% and 5% respectively. Prior experimental research programs [Tadros, Ghali and Meyer (1985)] and studies have proved that although the addition of non-prestressing steel poses to increase the stiffness of the concrete member, they actually tend to increase the losses and result in larger time –dependent deflection. The results confirm the statements above statements. The addition of mild steel of area $2.4 in^2$ and $5.0 in^2$ reduces the long term camber by approximately 10% and 17.4% respectively.

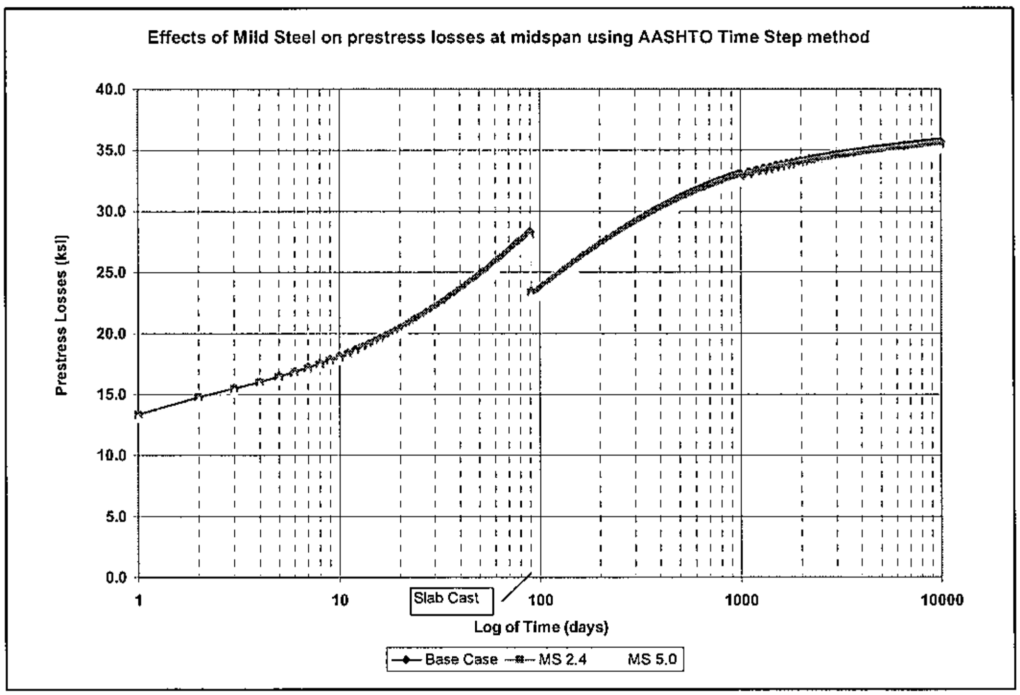

Figure 5-3 Effects of Mild Steel on prestress losses at mid span using AASHTO Time Step method

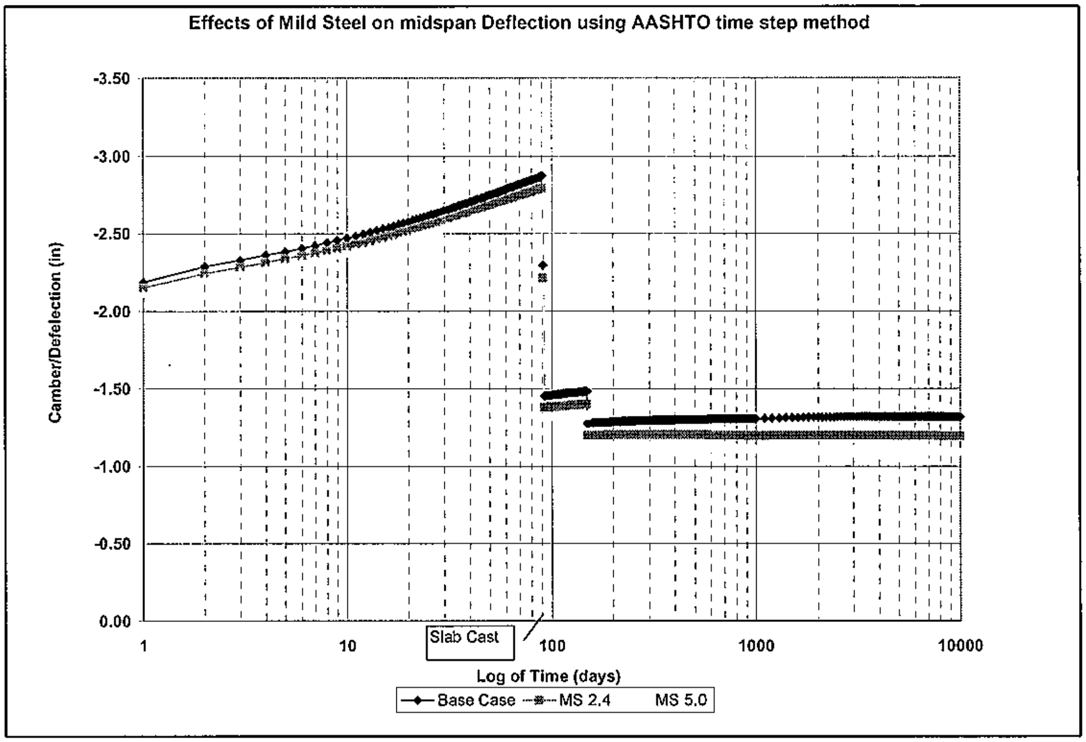

Figure 5-4 Effects of Mild Steel on mid span Deflection using AASHTO Time Step method

5.2.3 Effect of variation in Creep Coefficient

Creep coefficient which is the ratio of the creep strain to the elastic strain is influenced by various factors such as the volume to surface ratio of the concrete member, type and volume of aggregate, age of concrete at the time of loading and geometry of the concrete member. The proposed AASHTO Time Step method utilizes the AASHTO LRFD creep coefficient equation 5.4.2.3.2-1 (AASHTO LRFD Manual).

The Time Step method accounts for the variation of creep coefficient of concrete. The reduction in the value of creep coefficient increases the effective modulus and hence causes a reduction in creep effect and prestress losses. Figure 5-5 indicates that a 20%

decrease in the creep coefficient of concrete also lowers the long term losses by approximately 6% and hence vise versa. Figure 5-6 points out that 20% reduction in creep coefficient reduces the camber at 90 days by approximately 5%, and the long term camber by 6.8% when compared to the base case with 100% creep coefficient. The excel sheet also analyzed the effect of the combination of the varying creep coefficient with addition the top prestressing steel or mild steel bars. The results form Figure 5.7 shows that the combination of the 80% creep coefficient with the addition of four top strands reduced the camber by 69% form the base case.

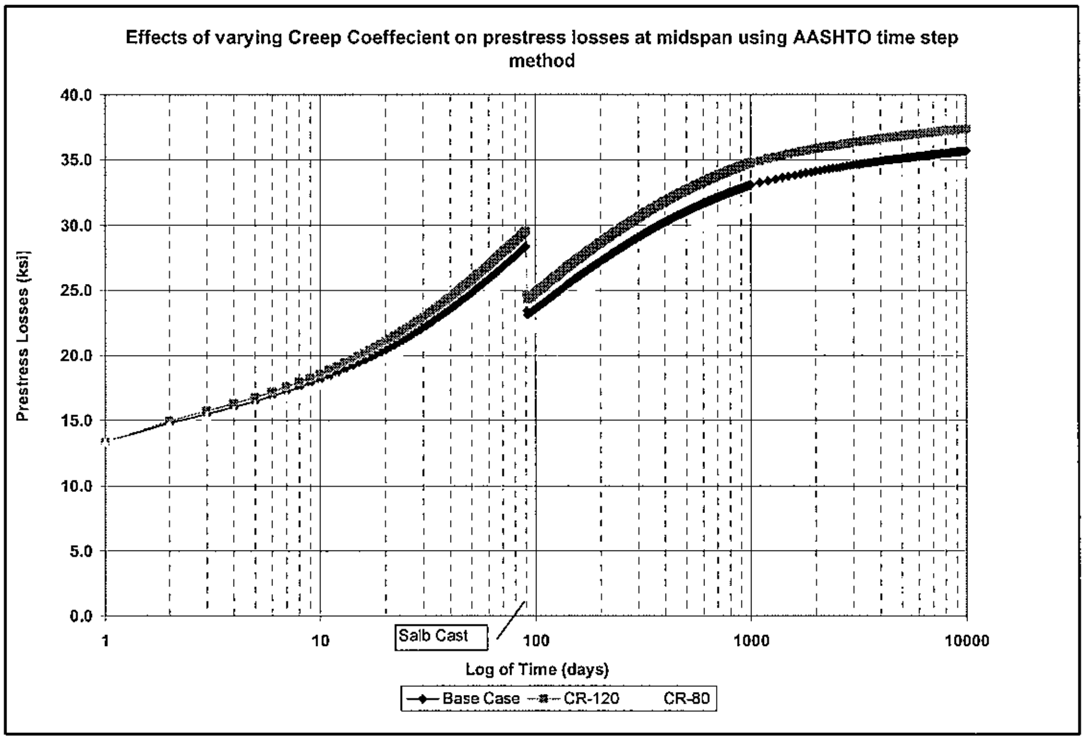

Figure 5-5 Effects of variation in Creep coefficient on prestress losses at midspan using AASHTO Time Step method

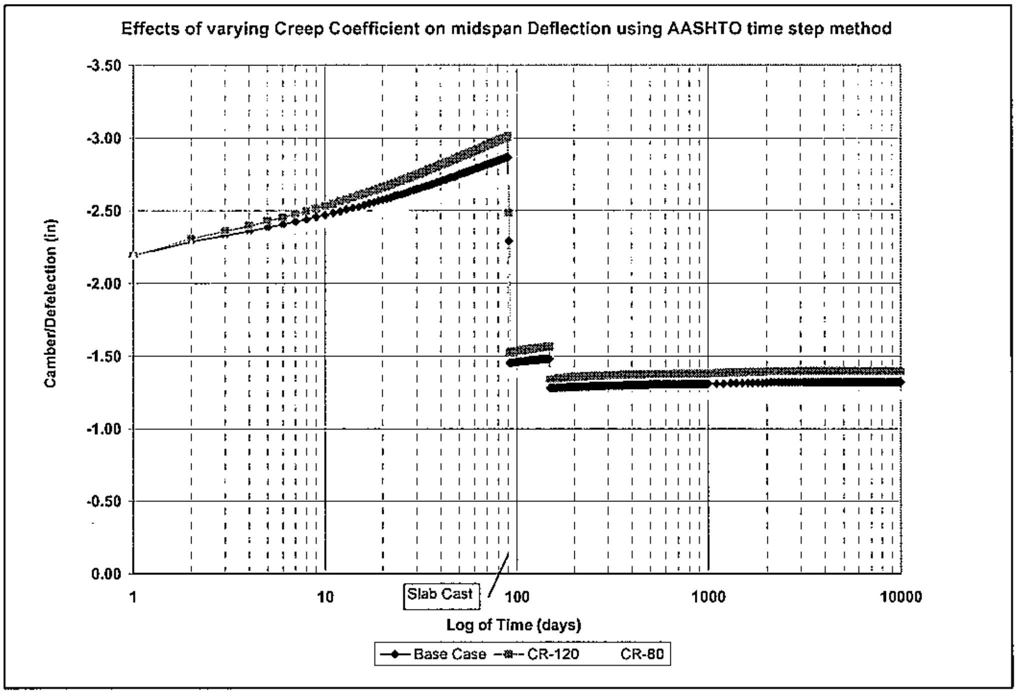

Figure 5-6 Effects of variation in Creep coefficient on mid span Deflection using AASHTO Time Step method

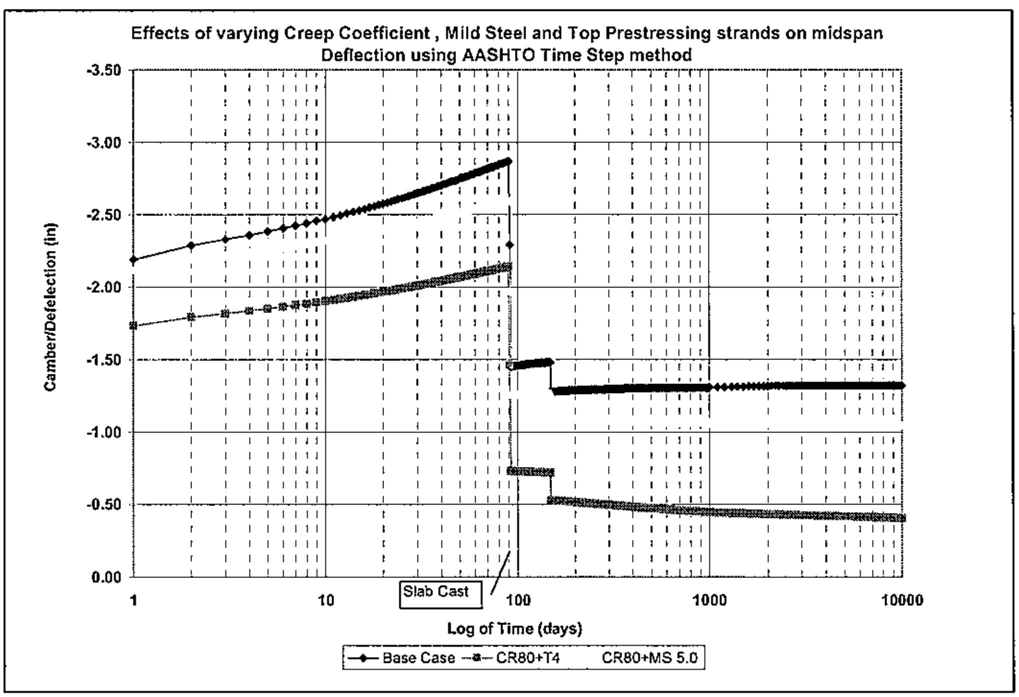

Figure 5-7 Effects of variation in Creep coefficient with Mild steel or top prestressing strands on mid span Deflection using AASHTO Time Step method

5.2.4 Effect of variation in Elastic Modulus of Concrete.

The AASHTO Time Step method is also modulated for the varying the modulus of elasticity of concrete. It is known from ACI 318-02 equation (ACI 310-02 Section 8.5.1). the modulus of elasticity increases approximately with the square root of the concrete compressive strength. The proposed Time Step method takes into account the effective modulus of elasticity of concrete which is an indirect function of concrete creep coefficient.

From the Figures 5-7 and 5-8 shown below we can see that a 20 % increase or decrease in the modulus of elasticity of concrete causes a significant increase or decrease in the initial and long term losses. It is interesting to note that the increase of the elastic modulus of concrete to 120% reduces the long term losses only by 6.6% but reduces the long term camber by 12% when compared to the base case with 100% elastic modulus. This is due to the fact that the increased modulus of elasticity reduces the transformed cross section properties of the concrete thereby reducing the curvature due to prestress resulting in lower camber. The sheet was also modulated for the combined effects of increased modulus of elasticity with the addition of four top prestressing strands as shown in Figure 5-10. The above case also reduced the camber form 1.32in to 0.38in.

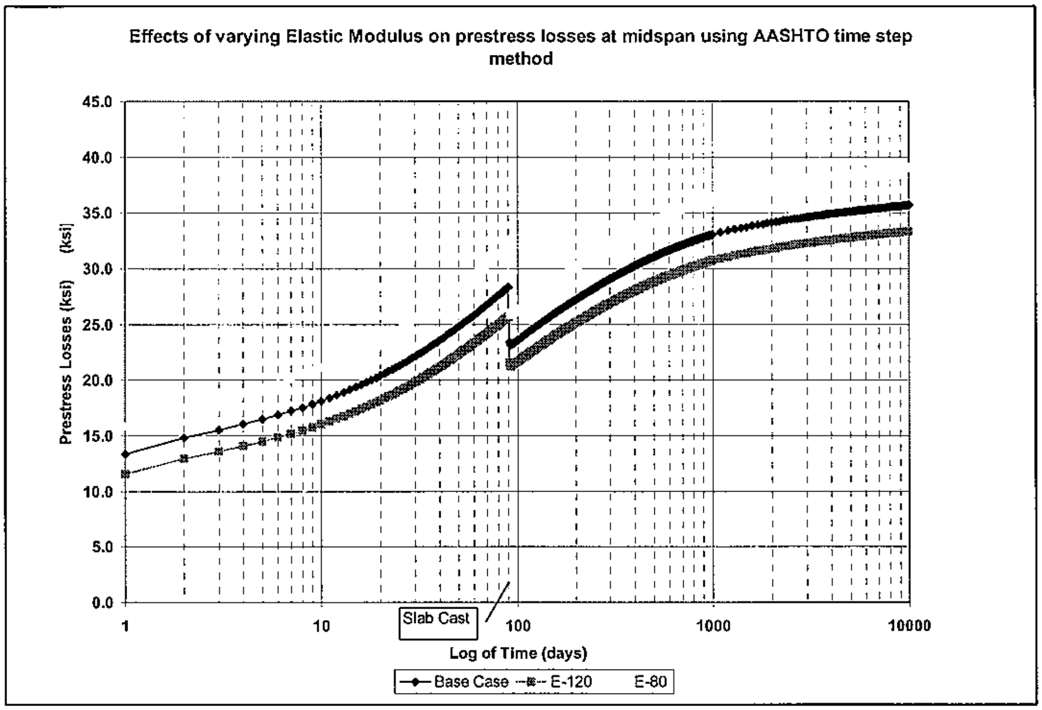

Figure 5-8 Effects of variation in Elastic Modulus of concrete on prestress losses at midspan using AASHTO Time Step method

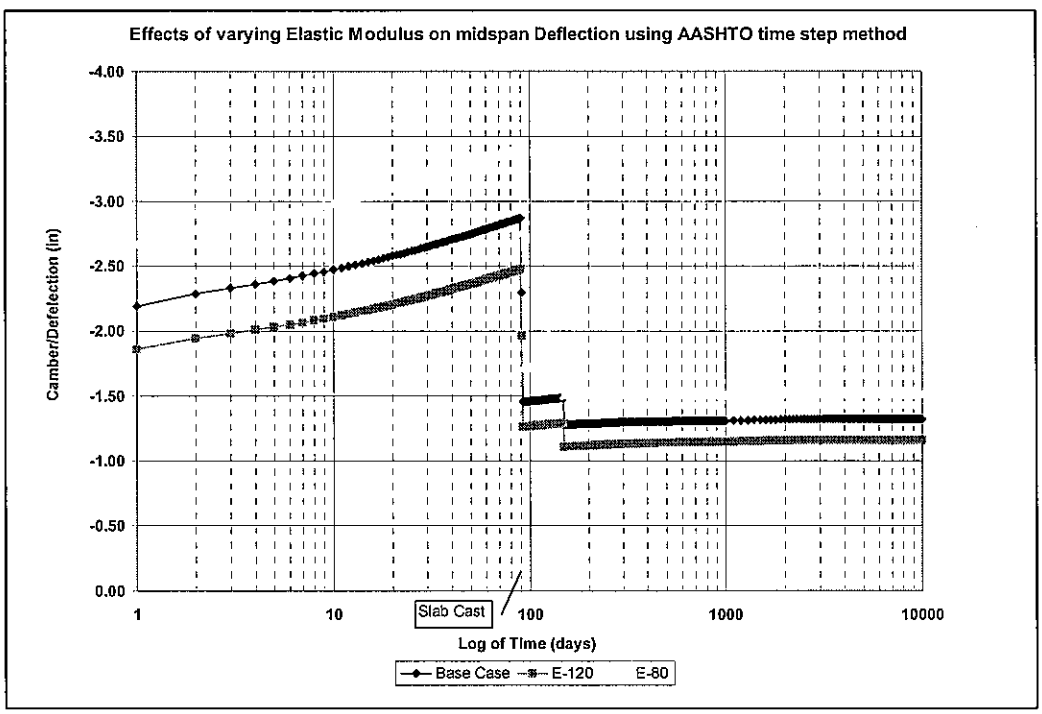

Figure 5-9 Effects of variation in Elastic Modulus of concrete on mid span Deflection using AASHTO Time Step method

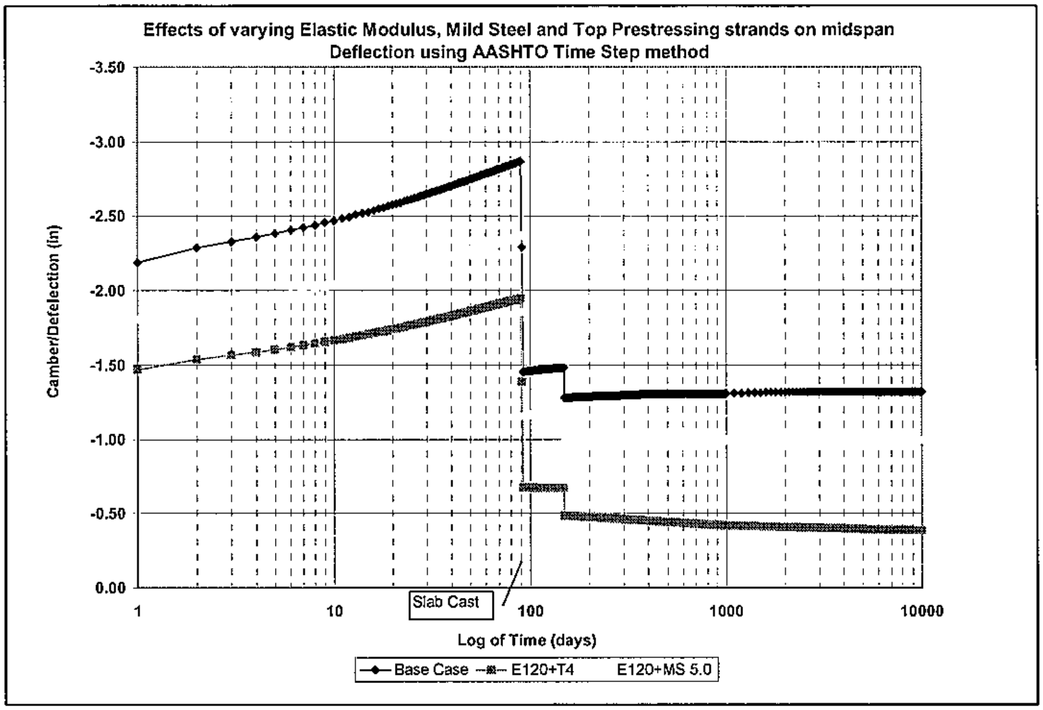

Figure 5-10 Effects of variation in Elastic Modulus of concrete with Mild steel or top prestressing strands on mid span Deflection using AASHTO Time Step method

5.3 Comparison of prestress losses between AASHTO Time Step method and NCHRP 496 detailed method.

Figure 5-11 shown below compares the prestresss losses at mid span predicted using the AASHTO Time Step method and the NCHRP 496 method. Prestress losses have been compared for two loading stages: prior to slab casting at 90 days and after 10 years. The results show that both the methods almost predict the same losses at loading stage of 90 days. The long term losses predicted using proposed method and the NCHRP 496 method were 35.69ksi and 33.32ksi.Both the AASHTO Time Step and the NCHRP 496 detailed method take into account the variability of material properties which influences the creep and shrinkage coefficient of the curing concrete. The reported literature [Hale and Russell

(2006)] showed that the NCHRP 496 method predicted losses within 18% of the measured losses. From the results it is concluded that the computation of losses using the proposed AASHTO Time Step method and predicted losses that were comparable to the losses predicted using the NCHRP Report 496 method.

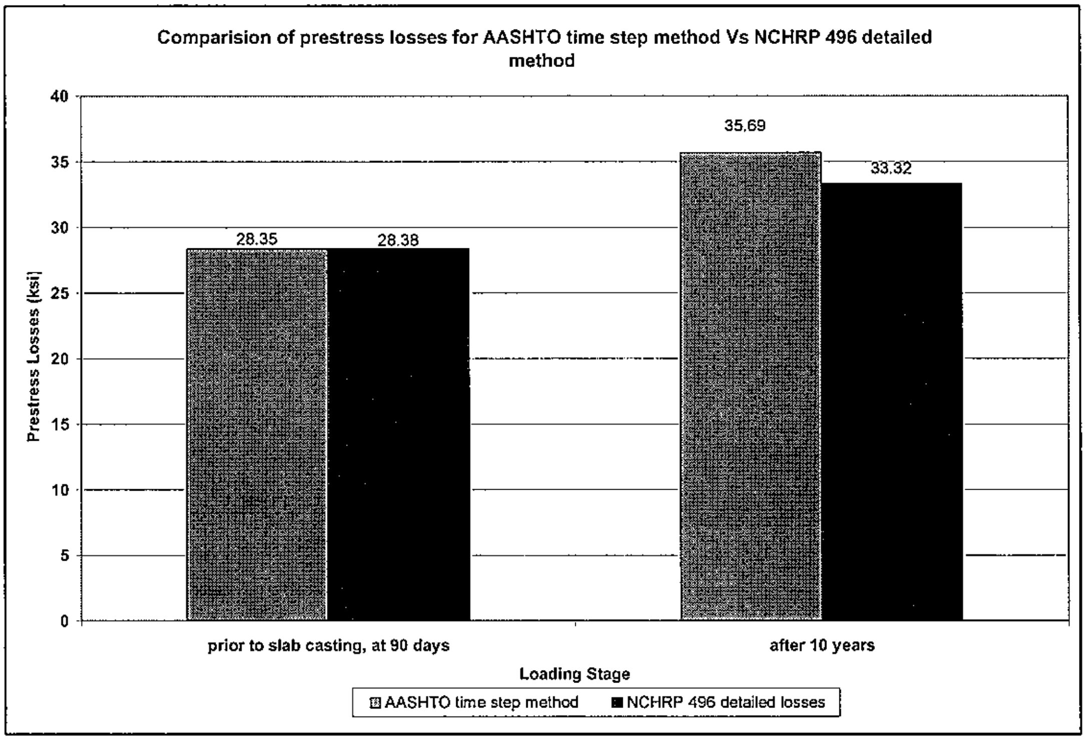

Figure 5-11 Plot showing the comparison of Prestress losses at mid span using the AASHTO Time Step and NCHRP 496 detailed method for two stages of loading.

5.4 Comparison of prestress losses between AASHTO Time Step method and AASHTO LRFD refined method.

Figure 5-12 shows that the AASHTO Time Step method predicts long term losses of about 1% lower than that predicted using the AASHTO LRFD method. The AASHTO LRFD refined method however estimates long term losses and is formulated using the normal concrete strength. The ultimate value of creep coefficients and shrinkage coefficients are used in the computation of the prestress losses for the same. However the AASHTO Time Step method takes into account the additional factors such as the age of concrete and the change in material properties and loading and calculates the creep coefficient on a day to day basis. In comparison to the AASHTO LRFD refined method, the Time Step method takes into consideration the reduction in creep coefficient associated with the increase in concrete strength.

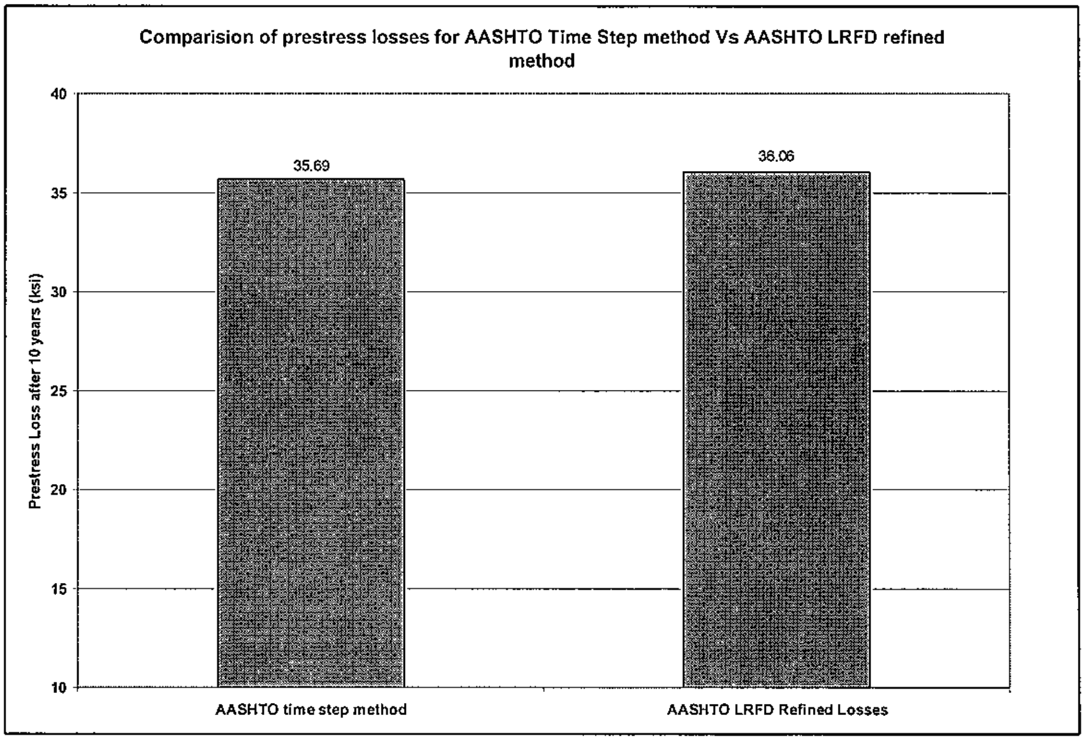

Figure 5-12 Plot showing the comparison of long term Prestress losses at mid span using the AASHTO Time Step and AASHTO LRFD refined method.

5.5 Comparison of prestress losses between AASHTO Time Step method and PCI Design Handbook method.

Figures 5-13 compares the losses predicted using the AASHTO Time Step and the PCI(Zia et al) method.. Where as the total losses at the final stage is about 8 ksi more than the PCI losses. The PCI Design Handbook method (Zia et al) method assumes a ultimate creep coefficient of 2.0 for the both the stress due to prestress loads and all the super imposed dead loads. Due to this fact the PCI Design Handbook method overestimates the losses for the loading stage prior to slab casting. The results prove that the proposed AASHTO Time Step method predicts more improved losses than the PCI Design Handbook method for intermediate loading stages.

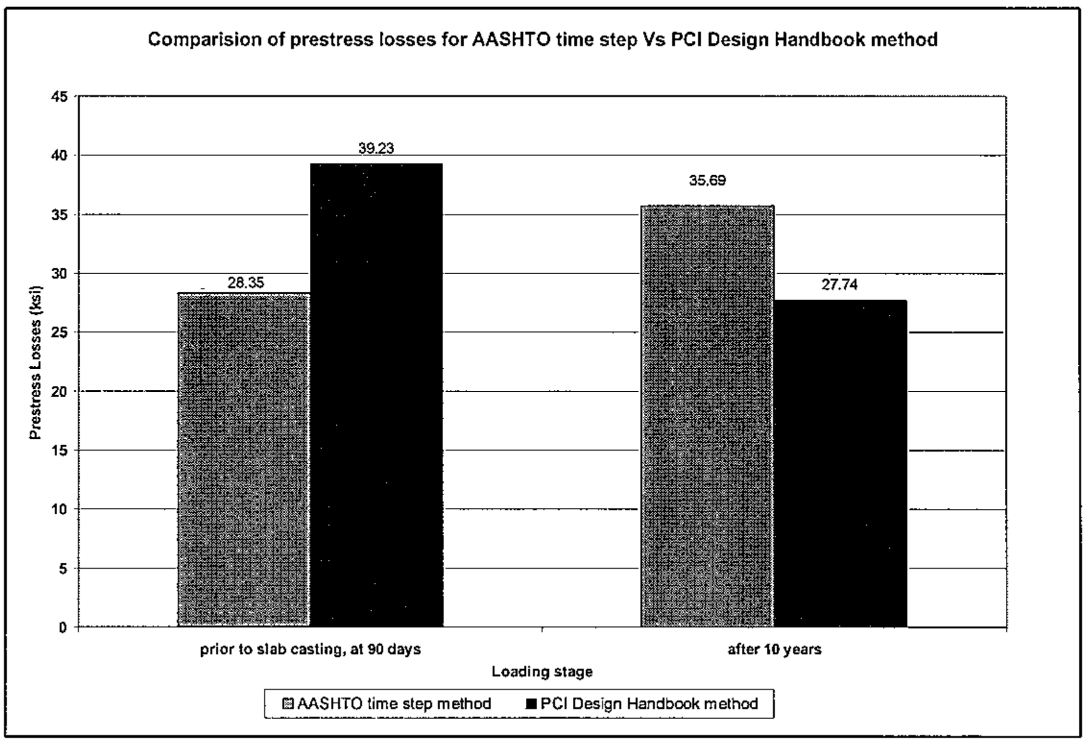

Figure 5-13 Plot showing the comparison of final Prestress losses at mid span using the AASHTO Time Step and PCI method.

Figure 5-14 displays the comparisons made between the camber predicted using the proposed AASHTO Time Step method and the PCI Design Handbook method. The AASHTO Time Step method calculates the curvature due to prestress , gravity and other super imposed loads , and predicts the camber using moment area method. Where as, the

PCI Design Hand book method makes use of multipliers for the prediction of the long term losses. Previous studies by Tadros,Ghali and Meyer(1985) affirm that the PCI Design Handbook multiplier method produce more convincing values for long term camber and deflection in prestressed concrete bridge girders. We can also conclude that the proposed method also predicts long term deflections within the acceptable range.

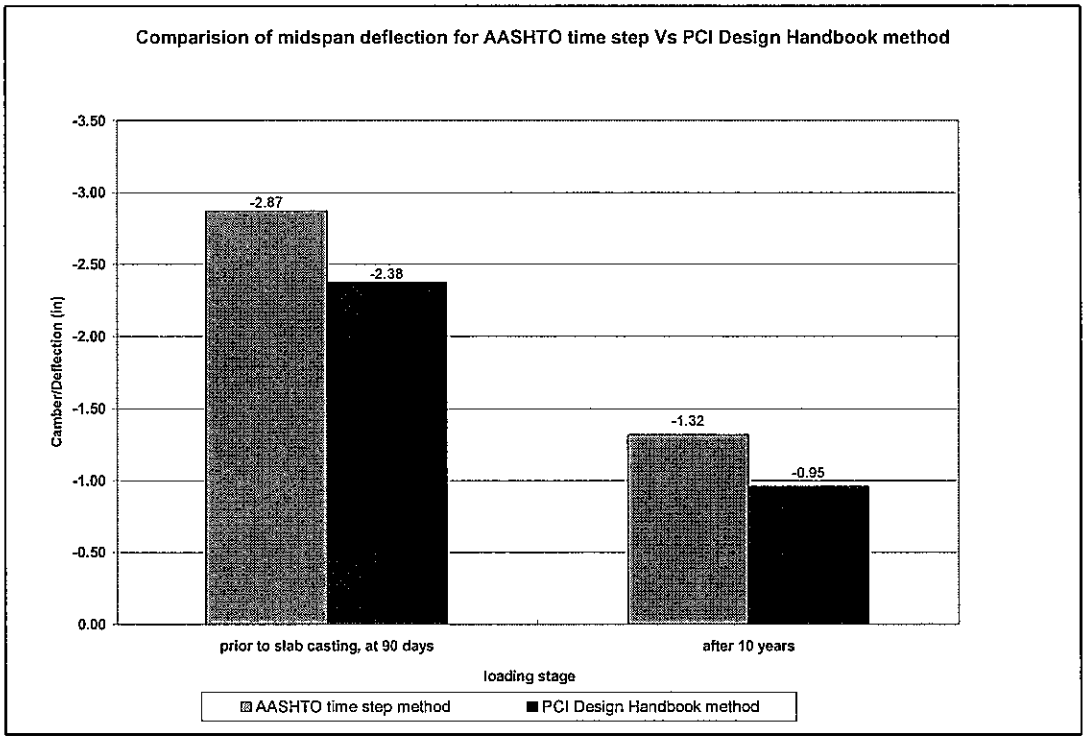

Figure 5-14 Plot showing the comparison of final Deflection at mid span using the AASHTO Time Step and PCI Design Handbook method.

CHAPTER VI

6.0 Summary, Conclusions and Recommendations

The foremost objective of the research program was to investigate the relevant literature pertaining to the prediction of prestress losses. Prestress losses were predicted using the available loss prediction methods. The proposed AASHTO Time Step method was developed by modulating the AASHTO LRFD refined equations to accommodate time dependent creep effects. Prestress losses and camber and deflection were predicted using the proposed method as well as the PCI Design Hand book method. The AASHTO type IV girder properties were selected to perform the analysis for each of the methods.

The research further investigates the varying concrete material properties. Material properties of the concrete were varied by changing the creep coefficient and the modulus of elasticity of concrete ±20 percent. The design properties accommodate the addition of top prestressing strands and the mild steel. The findings of the research are summarized below:

- The AASHTO Time Step method takes into account the incremental increase in creep and calculates the losses and camber on a day to day basis.
- The AASHTO Time Step method takes into account the variability in the material properties and is formulated for HPC when compared to the AASHTO LRFD refined method.

- The AASHTO Time Step method accommodates the use of effective modulus which combines the effect due to elastic strain and the creep of concrete as an elastic deformation on the concrete section.
- Addition of two and four top prestressing strands reduces the camber by 35% and 69% respectively.
- The addition of four (4) #7 and five (5) # 9 mild steel bars do not alter appreciably the values of long term losses; however, their inclusion decreases the long term camber by approximately 10% and 17.4% for inclusion of mild steel, A_{ms} = 46%A_{ps} and A_{ms} = 96%A_{ps} respectively.
- The decrease in creep coefficient to 80% decreases the long term camber by 6.8% from the base case of 100% creep.
- 20% increase in Elastic modulus of concrete lowers the long term prestress losses by 6% and long term camber by 12%.
- The AASHTO Time Step method predicts long term losses equivalent to losses predicted using the NCHRP 496.
- The camber predicted using the proposed AASHTO Time Step method is comparable to the camber predicted using the PCI Design Handbook method.

6.1 Recommendations

The following are the design guidelines and the recommendations made to ODOT and OTA for the more refined prediction of prestress losses, camber and deflection.

- Add top prestressing strands in prestressed concrete beams to lower the long term losses and camber by approximately 69%.
- Add mild steel which increases stiffness to the concrete beam as well as reduces the long term camber by approximately 17.4%.
- Use of the proposed AASHTO Time Step method to determine the losses for the prediction of intermediate and long term prestress losses and camber/deflection in prestressed concrete bridge girder

REFERENCES

AASHTO 2004,AASHTO LRFD Bridge Design Specifications Third Edition, American Association of State Highway and Transportation Offcials,Washington,DC

AASHTO 1994,AASHTO LRFD Bridge Design Specifications Third Edition, American Association of State Highway and Transportation Offcials,Washington,DC

ACI Committee 318,2005,Building Code Requirements for Structural Concrete (ACI 318-05) and Commentary (ACI 318R-05),American Concrete Institute, Farmington Hills, MI

ACI Committee 363, "State of the Art of Report on High- Strength Concrete," American Concrete Institute, Detroit, MI (1992) p.3.

ACI Committee 363, "State of the Art of Report on High- Strength Concrete," ACI Structural Journal, V.81, No.4, July-August 1984, pp. 363-411.

ACI Committee 318, "Building Code Requirements for Reinforced Concrete," ACI 318-89, American Concrete Institute, Detroit, MI (1989).

ACI Committee 318, “Building Code Requirements for Reinforced Concrete,” ACI 318-98/318R-99, American Concrete Institute, Detroit, MI (1999).

ACI Committee 318, “Building Code Requirements for Structural Concrete, (ACI 318-95)”, American Concrete Institute, Farmington Hills, MI, (1995).

ACI Committee 209, “Prediction of creep, (ACI 209R-45)”, ACI Manual of Concrete Practice, part-1, 2002.

ACI 209R-92, “ Prediction of Creep Shrinkage and Temperature Effects in Concrete Structures”, Reported by ACI Committee 209.(1992)

Abdel Karim, A. and Tadros, M.K., “Computer Analysis of Spliced Girder Bridges,” ACI Structural Journal, Vol.90,No.1, Detroit, MI(January-February 1993),pp.21-31.

A Ghali, AM Neville, “Effect of Elastic and Creep Recoveries of Concrete on Loss of Prestress”, PC Jha - ACI Journal Proceedings, Volume: 64. Issue: 12. December 1, 1967.

American Association of State Highway and Transportation Officials, “AASHTO-LRFD Bridge Design Specifications,” First Edition, 1994 and Interim Specifications through 1996.

American Association of State Highway and Transportation Officials, "AASHTO-LRFD Bridge Design Specifications," Second Edition, Washington, DC (1998).

American Association of State Highway and Transportation Officials , Standard Specification for Highway Bridges, 14th Edition.Washington,D.C,(1989),420pp.

American Association of State Highway and Transportation Officials (AASHTO), Standard Specification for Highway Bridges: Fifteenth Edition.Washington,D.C,(1992).

American Association of State Highway and Transportation Officials (AASHTO), Standard Specification for Highway Bridges: Fifteenth Edition.Washington,D.C,(1993).

American Association of State Highway and Transportation Officials (AASHTO), Standard Specification for Highway Bridges: Sixteenth Edition.Washington,D.C(1996).

American Association of State Highway and Transportation Officials (AASHTO), (1998).Standard Specification for Highway Bridges: Sixteenth Edition.Washington,D.C(1998).

American Concrete Institute (ACI),(2002). "Building code requirement for structural concrete and commentary." Manual Concrete Practice,ACI 318-02,Farmington Hills, MI.

American Concrete Institute (ACI),(1992). "Prediction of creep, shrinkage, and temperature effects in concrete structures." Manual of Concrete Practice, ACI 209R-92, Farmtington Hills, MI.

American Concrete Institute (ACI),(1992). "State-of-the-art report on high strength concrete" Manual of Concrete Practice,ACI 363R-92,Farmington Hills,MI

American Concrete Institute (ACI),(2002). "Evaluation of strength test results of concrete." Manual of Concrete Practice, ACI 214R-02, Farmington Hills,MI.

Al-Omashi,N., "Prestress losses in Pretensioned High-Strength Concrete Bridge Girders," Ph.D. Dissertation, Department of Civil Engineering, University of Nebraska, Lincoln, NE(2001).

ASTM C469-94, "Standard Test Method for Static Modulus of Elasticity and Poission's Ratio of Concrete in Compression," American Society of Testing and Materials, Annual Book, Philadelphia, PA (1994) pp.238-241.

Bruce W. Russell and W.Micah Hale., “Effects of Allowable Compressive Stress at Release on Prestress Losses and on the Performance of Precast, Prestressed Concrete Bridge Girders”, PCI Journal, Precast/Prestressed Concrete Institute,vol51,no.2, (March-April, 2006).

Bazant, Z.P. and Baweja, S.,(1995a). “Creep and shrinkage prediction model for analysis and design of concrete structures-model B3, RILEM recommendations.” Materials and Structures, 28,357-365.

Bazant, Z.P. and Baweja, S., (1995b). “Justification and refinements of model B3 for concrete creep and shrinkage 1.Statistics and sensitivity.” Materials and Structures, 28,415-430.

Bazant, Z.P. and Baweja, S., (1995c). “Justification and refinements of model B3 for concrete creep and shrinkage 2.Updating and theoretical basis.” Materials and Structures, 28,488-495.

Branson,D.E., and Christaison, M. L., “Time-Dependent Concrete Properties Related to Design-Strength and Elastic Properties, Creep and Shrinkage,” SP-27, American Concrete Institute, Detroit, 1971, pp.257-277.

Branson D.E and Panarayanan K.M, “Loss of Prestress of Non-Composite and Composite Prestressed Concrete Structures,” PCI Journal, September-October 1971.

CEB-FIP, 'Model Code for Concrete Structures', English Translation. Paris: Comite Euro-international due Beton-Federation International de la precontrainte,1978.

Comite Euro-International du Beton (CEB),(1990). "CEB-FIP model code 1990." Buletin D'Information No.213/214, Lausanne, Switzerland.

Concrete Technology Associates, "Prestress Losses," Technical Bulletin 73-B7, Concrete Technology Associates, Tacoma, WA (July 1973).

Gross, S.P and Burns, N.H., "Field Performance of Prestressed High Performance Concrete Bridges in Texas," Research Report 580/589-2, Center for Transportation Research, University of Texas at Austin, Austin, TX (1999).

Ghali, A., Tadros, M. K., and Dilger, W. H., "Accurate Evaluation of Time Dependent Deflection of Prestressed Concrete Frames," Deflection of Concrete Structures, SP-43, American Concete Institute, Detroit, 1974, pp. 357-375.

Gardener, N.J. and Lockman, M.J., (2001). "Design provisions fordrying shrinkage and creep of normal-strength concrete." ACI Materials Journal,98(2),159-167

Gruel, A., Rogers, B.T., Miller, R.A., Shahrooz, B.M., and Baseheart, T.M., "Evaluation of a High Performance Concrete Box Girder Bridge, "Evaluation of a

High Performance Concrete Box Girder Bridge," Research Report, University of Cincinnati, Cincinnati, OH (2000).

Micah Hale, "Investigating the need for and effect of air entrainment in high performance concrete," Ph.D. Dissertation, Department of Civil Engineering and Environmental Science, University of Oklahoma, Oklahoma, OK(2002).

Hansen, T.C. and Mattock, A.H., "Influence of Size and Shape of Member on the Shrinkage and Creep of concrete." ACI Materials Journal, vol.63, No.2, Detroit, MI (1966) pp.267-289.

Hou, X.S., Al-Omashi, N., and Tadros, M.K., "Creep, Shrinkage and Modulus of Elasticity of High Performance Concrete," ACI Materials Journal, Vol.98, No.6, Detroit, MI (2001) pp.440-449.

Le Roy, R., De Larrard, F., and Pons,G.(1996). "The AFREM code type model for creep and shrinkage of high-performance concrete." Proceedings of the 4th International Symposium on Utilization of High-Strength/High-Performance Concrete, Paris.

IS-1343, 'Code of practice for prestressed concrete', Bureau of Indian Standards, New Delhi,1980.

Lwin, M.M., Khaleghi, B., and Hsieh, J, C., "Prestressed I-Girder Design Using High Perfomance Concrete and the New AASHTO LRFD Specifications," PCI/FHWA International Symposium on High Performance Concrete New Orleans, LA (October 1997) pp.406-418.

Mokhtarzadeh, a., "Mechanical Properties of High Strength concrete," Ph.D Dissertation, Department of Civil Engineering, University of Minnesota, Minneapolis, MN (1996).

Naaman, A.E., and Siriaksorn, A., Prestressed Concrete Analysis and Design – Fundamentals, First Edition, MacGraw Hill Book Co., New York,N.Y.,1982,670pp.

PennDOT, "Design Manual Part 4- Structures," Pennsylvania Department of Transportation, Harrisburg, PA, August 1993.

Pickett, G., "Shrinkage stresses in concrete," ACI Journal, Proceedings V.42, No.3, Jan-Feb. 1946, pp. 165-204, 361-400.

Precast/Prestressed Concrete Institute (PCI), (1997).Bridge Design Manual, Chicago, IL.

Precast/Prestressed Concrete Institute (PCI) Committee on Prestress Losses, (1975). "Recommendation for estimating prestree losses."PCI Journal, 20(4), 43-75.

PCI Design Handbook Precast Prestressed Concrete, Second Edition, Prestressed Concrete Institute, Chicago, Illinois, 1978.

PCI Design Handbook, Precast/Prestressed Concrete Institute, Chicago, IL, 1999.

PCI Design Handbook, Precast/Prestressed Concrete Institute, Chicago, IL, 2005.

Pessiki, S., Kaczinski, M., and Wescott, H,.H., "Evaluation of Effective Prestress Forces in 28-Year-Old Prestressed Concrete Bridge Beams," PCI Journal, Vol.41, No.5, Chicago,IL (November-December 1996) pp.78-89.

P.Shreeshylam, K.Ravishankar,S.Parivallal,K.Kesavan and S.Sridhar., "Measured long-term prestress losses versus design code estimates",Journal of Structural Engineering (Madras), v32, n1, April/May, 2005, Full-Scale Testing and Evaluation of Structures,p37-44.

PCI Committee on Prestress Losses, "Recommendations for Estimating Prestress Losses", PCI JOURNAL, V.20, No.4, July-August 1975, pp,45-70.

Shenoy, C.V., and Frantz, G.C., "A Simple Method of Gripping Prestressing Strands for Tension Tests, "PCI JOURNAL, V. 36, No. 4, July-August 1991, pp. 58-65.

Shams M.K and Kahn, L.F.,(2000). "Time-dependent behavior of high-strength concrete task 3, use of high strength/high performance concrete for precast prestressed concrete bridge in Georgia." Structural Engineering, Mechanics, and Materials Research Report No.00-1, Georgia Institute of Technology, and Atlanta GA.

Shenoy, C.V., and Frantz, G.C., "Structural test of 27-Year-Old Prestressed Concrete Bridge Beams,"PCI Journal, September-October, Chicago, IL(1991) pp80-90.

Tadros, M.K., Ghali, A. and Dilger, W.H., "Time- Dependent Prestress Loss and Deflection in Prestressed Concrete Members," PCI Journal, Vol.20, No.3, Chicago, IL (May-June 1975) pp.86-98.

Tadros, M.K., Ghali, A. and Dilger, W.H., "Time- Dependent Analysis of Composite frames," ASCE Journal of Structural Engineering, New York, NY (April 1977) pp. 871-884.

Tadros, M.K., Ghali, A., and Meyer, A.W., "Prestress Loss and Deflection of Precast Concrete Members," PCI Journal, Vol.30, No.1, Chicago, IL (January-February 1985) pp.114-141.

Maher K. Tadros, Nabil AL-Omashi, Stephen J.Seguirant and James G. Gallt, "NCHRP Report 496", Washington D.C., 2003.

T.Y Linn and Ned H. Burns. "Design of Prestressed Concrete Structures", Third Edition , John Wiley & Sons, 2004.

Waldron, Christopher J., "Investigation of long-term prestress losses in prestensioned high performance concrete girders," Ph.D.Dessertation, Department of Civil Engineering, Virginia Polytechnic Institute and State University, Blacksburg, A (November 16, 2004) p.220.

Window,A.L.,and Holister, G.S., "Strain gauge technology", 1st Edition, Applied Science Publishers, England,1982

Zia, P., Preston, H., Scott, N., and Workman, E., June 1979, "Estimating Prestress Losses," Concrete International, V.1, No.6, pp.32-38.