

**LONG-TERM DEFORMATION OF
INDETERMINATE PSC BRIDGE
STRUCTURES**

A DISSERTATION

*Submitted in partial fulfillment of the
requirements for the award of the degree*

of

MASTER OF TECHNOLOGY

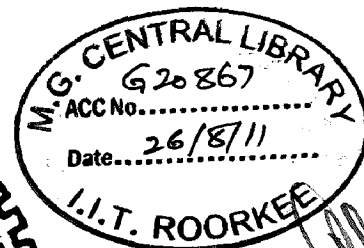
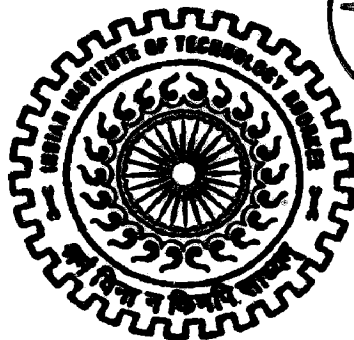
in

CIVIL ENGINEERING

(With Specialization in Structural Engineering)

By

MD AZIM EQUBAL FAROOQUI



**DEPARTMENT OF CIVIL ENGINEERING
INDIAN INSTITUTE OF TECHNOLOGY ROORKEE
ROORKEE - 247 667 (INDIA)**

JUNE, 2011

INDIAN INSTITUTE OF TECHNOLOGY ROORKEE
ROORKEE

CANDIDATE'S DECLARATION

I hereby certify that the work which is being presented in the dissertation report entitled, "**LONG-TERM DEFORMATION OF INDETERMINATE PSC BRIDGE STRUCTURES**", in partial fulfillment of the requirements for the award of degree of **Master of Technology in Civil Engineering** with specialization in Structural Engineering submitted to the Department of Civil Engineering, Indian Institute of Technology Roorkee, is an authentic record of my own work carried out under the supervision of Dr. AKHIL UPADHYAY, Associate Professor, Department of Civil Engineering, Indian Institute of Technology Roorkee, Roorkee, India.

The matter embodied in this dissertation report has not been submitted by me for the award of any other degree or diploma.

Date: - 27/06/2011

Place:-Roorkee

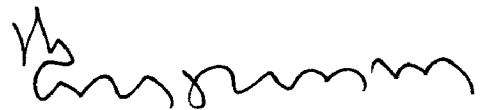
Md. Azim Equbal Farooqui
(MD AZIM EQUBAL FAROOQUI)

CERTIFICATE

This is to certify that the above statement made by the candidate is correct to the best of my knowledge.

Date: - 27/6/11

Place: - Roorkee



(DR. AKHIL UPADHYAY)

Associate Professor

Department of Civil Engineering

I.I.T. Roorkee

ACKNOWLEDGEMENT

I wish to express my deep sense of gratitude and sincere thanks to my supervisor Dr. Akhil Upadhyay, Associate Professor, Department of Civil Engineering, Indian Institute of Technology Roorkee, Roorkee, India, for his intuitive and meticulous guidance and perpetual inspiration at various stages of work.

I would also like to express my gratitude to my parents; their blessings, motivation and inspiration have always provided me with high mental support and contributed in all possible ways, in the completion of this dissertation. I am also thankful to my wife for continuous support and patience during different stages of dissertation.

Thanks are due to all my friends for giving me a good company during my stay at IIT Roorkee.

Finally, I am grateful to the Almighty for giving me the strength and patience for completion and submission of this dissertation.

Md-Azim Equbal Farooqui
(MD AZIM EQUBAL FAROOQUI)

Bridge structures are of two types namely determinate and indeterminate. Simply-supported bridge girders come under the category of determinate bridge structures. The indeterminate bridge structures include continuous and rigid frame bridges. They can be constructed either by making precast, prestressed concrete girders continuous by cast-in-situ deck slab and diaphragm or by segmental construction. Long-term behaviour of prestressed concrete girder refers to the time-dependent deformation and prestress losses due to creep and shrinkage of concrete and relaxation of prestressing steel. Long-term effects due to creep, shrinkage and relaxation cause significant increase in deflection of prestressed concrete bridges. In this dissertation long-term behaviour of indeterminate post-tensioned bridge structures has been investigated. To begin with, long-term deformation of simply-supported bridge girder has been presented. A program based on incremental time-step method has been written in MATLAB to estimate the long-term deformation and prestress losses for simply-supported bridge girder. Provisions of CEB-FIP90 and ACI 209R-92 model codes for creep and shrinkage of concrete and relaxation formulae for low relaxation and stress-relieved strands have been incorporated in the program. Numerical studies are carried out to develop quantitative information for the effect of change in material models and environmental parameters on long-term deformation and prestress loss. After that, time-dependent restraint moment for a two-span precast, prestressed bridge girders made continuous using cast-in-place diaphragm has been predicted using a computer program written in MATLAB. Numerical studies have been performed for this case also. Finally, a typical segmental bridge constructed by cantilever construction has been analyzed using LUSAS 14 software to predict the time-dependent deflection at each stage of construction and service life. The effect of construction schedule, age of precast girder at the time of erection and environmental parameters has been illustrated through numerical studies.

TABLE OF CONTENTS

Title	Page No.
CERTIFICATE	i
ACKNOWLEDGEMENT	ii
ABSTRACT	iii
TABLE OF CONTENTS	iv
LIST OF FIGURES	ix
LIST OF TABLES	xii
CHAPTER 1	
INTRODUCTION	1
1.1. IMPORTANCE OF PRESTRESSED CONCRETE BRIDGES	2
1.2. LONG-TERM DEFORMATION IN PRESTRESSED CONCRETE BRIDGES	2
1.2.1. Causes Of Long-Term Deformation	2
1.2.2. Issues Involved In Long-Term Deformation Prediction	2
1.3. LITERATURE REVIEW	3
1.3.1. Time-Dependent Effects of Creep and Shrinkage of Concrete	3
1.3.2. Prediction of Long-Term Prestress Losses	5
1.3.3. Long-Term Deformation of Determinate PSC Bridge Structures	6
1.3.4. Long-Term Deformation of Ineterminate PSC Bridge Structures	6
1.3.4.1. Precast Prestressed Concrete Girders Made Continuous Using Cast-in-Place Deck and Continuity Diaphragm	6
1.3.4.2. Segmental Bridges	8
1.3.5. Conclusions from Literature Review	11
1.4. OBJECTIVE AND SCOPE	12
1.5. GENERAL APPROACHES	12
1.6. ORGANISATION OF THE THESIS	13
CHAPTER 2	

PRESTRESSED CONCRETE: BASIC CONCEPTS

2.1.	GENERAL	15
2.2.	SHORT-TERM PRESTRESS LOSSES	15
2.2.1.	Elastic shortening	16
2.2.2.	Friction	16
2.2.3.	Anchorage Slip	16
2.3.	LONG-TERM PRESTRESS LOSSES	17
2.3.1.	Losses due to Creep of Concrete	17
2.3.2.	Losses due to Shrinkage of Concrete	18
2.3.3.	Losses due to Relaxation of Prestressing Steel	19
2.3.4.	Flowchart for the Calculation of Time-Dependent Prestress Losses	19
2.4.	MATERIAL MODELS	22
2.4.1.	CEB-FIP90	22
2.4.2.	ACI 209R-92	27
2.4.3.	Relaxation Model	29
2.5.	LONG-TERM DEFORMATION	29
2.6.	SUMMARY	29

CHAPTER 3

PREDICTION OF LONG-TERM BEHAVIOUR

3.1.	GENERAL	30
3.2.	DIFFERENT APPROACHES FOR PREDICTION OF LONG-TERM BEHAVIOUR	31
3.2.1.	PCI Multipliers Method	31
3.2.2.	Approximate Time-Step Method	32
3.2.3.	Deflection of Composite Girders	33
3.3.	INCREMENTAL TIME-STEP METHOD	33
3.4.	ASSESSMENT OF LONG-TERM RESTRAINT MOMENT IN PRECAST CONCRETE BRIDGE GIRDERS MADE CONTINUOUS	36
3.5.	SUMMARY	36

CHAPTER 4

LONG-TERM RESPONSE PREDICTION OF DETERMINATE BRIDGE STRUCTURES

4.1.	GENERAL	38
4.2.	SIGNIFICANCE OF LONG-TERM DEFORMATION IN PSC BRIDGES	38
4.3.	ANALYSIS APPROACH	38
4.4.	VALIDATION OF THE METHOD	39
4.5.	EFFECT OF NUMBER OF STEPS	41
4.6.	PARAMETRIC STUDIES	42
	4.6.1. Effect of Material Models	43
	4.6.2. Effect of Environment	46
4.7.	CONCLUDING REMARKS	50

CHAPTER 5

LONG-TERM RESPONSE PREDICTION OF INDETERMINATE BRIDGE STRUCTURES: PRECAST PRESTRESSED BRIDGE GIRDERS MADE CONTINUOUS

5.1.	GENERAL	51
5.2.	ADVANTAGES OF CONTINUOUS BRIDGE	51
5.3.	CONCEPT OF RESTRAINT MOMENT	52
5.4.	BRIEF DESCRIPTION OF THE METHODS FOR THE CALCULATION OF RESTRAINT MOMENT	53
	5.4.1. PCA Method	53
	5.4.2. Trost's Approach	54
5.5.	ANALYSIS APPROACH	55
5.6.	NUMERICAL STUDIES	56
	5.6.1. Effect of Analysis Approach	57
	5.6.2. Effect of Material Model	58
	5.6.3. Effect of Environment	59
	5.6.4. Effect of Age of Girder at Continuity	61
5.7.	CONNECTION DETAILS FOR POSITIVE RESTRAINT MOMENT	61

5.8.	SUMMARY	62
------	---------	----

CHAPTER 6

LONG-TERM RESPONSE PREDICTION OF INDETERMINATE BRIDGE STRUCTURES: SEGMENTAL BRIDGES

6.1.	GENERAL	63
6.2.	LONG-TERM DEFORMATION IN SEGMENTAL BRIDGES	64
6.3.	ANALYSIS METHODOLOGY	64
6.4.	ANALYSIS OF INTERNAL SPAN OF A SEGMENTAL BRIDGE	64
6.5.	ANALYSIS OF SEGMENTAL BRIDGE	69
	6.5.1. Deflected Shape of Bridge at Various Stages of Construction	72
	6.5.2. Effect of Construction Schedule	77
	6.5.3. Numerical Studies	78
	6.5.4. Variation of Bending Moment in the Completed Bridge with Time	81
6.6.	EXTERNAL PRESTRESSING AS A SOLUTION FOR CONTROLLING DEFORMATION AND STRESSES	81
	6.6.1. Introduction to External Prestressing	82
	6.6.2. Internal Vs External Prestressing	82
6.7.	SUMMARY	83

CHAPTER 7

DISCUSSION OF RESULTS AND CONCLUSION

7.1.	GENERAL	84
7.2.	LONG-TERM DEFORMATION OF DETERMINATE BRIDGE STRUCTURES	84
	7.2.1. Effect of Material Model	84
	7.2.2. Effect of Relative Humidity	85
	7.2.3. Effect of Temperature	85
	7.2.4. Discussion of Results	85
7.3.	LONG-TERM BEHAVIOUR OF PRECAST, PRESTRESSED CONCRETE BRIDGE GIRDERS MADE CONTINUOUS	86

7.3.1. Effect of Analysis Approach	86
7.3.2. Effect of Material model	86
7.3.3. Effect of Age of Girder at the time of Establishment of Continuity	86
7.3.4. Effect of Relative Humidity	86
7.3.5. Effect of Temperature	87
7.3.6. Discussion of Results	87
7.4. LONG-TERM DEFORMATION OF SEGMENTAL BRIDGES	87
7.4.1. Variation of Deflection with Time	88
7.4.2. Effect of Construction Schedule	88
7.4.3. Effect of Relative Humidity	88
7.4.4. Effect of Age of Precast Segment at the Time of Erection	88
7.4.5. Variation of Bending Moment in the Completed Bridge with Time	88
7.4.6. Discussion of Results	88
7.5. CONCLUSIONS	89
7.6. SCOPE FOR FUTURE RESEARCH	90
REFERENCES	91
BIBLIOGRAPHY	94
LIST OF TECHNICAL PAPERS COMMUNICATED	95
APPENDIX I	96

LIST OF FIGURES

Figure No.	Title	Page No.
Figure 1.1	Longitudinal View and Section of Typical Segmental Bridge (Tadros et al., 1979)	9
Figure 1.2	Joint Displacements before Application of Loads of Segment 2 in Match-Cast Cantilever Construction Method (Tadros et al., 1979)	9
Figure 1.3	Deflection of a Four-Segment Cantilever (Tadros et al., 1979)	10
Figure 3.1	Multipliers for Long-Term Camber and Deflection	32
Figure 3.2	Strain Changes and Rotations at Step n	34
Figure 4.1	Prestress Loss Vs Time	44
Figure 4.2	Deflection (Self Weight) Vs Time	44
Figure 4.3	Deflection (Prestress) Vs Time	45
Figure 4.4	Deflection (Total) Vs Time	45
Figure 4.5	Prestress Loss Vs Relative Humidity	46
Figure 4.6	Deflection (Self Weight) Vs Relative Humidity	47
Figure 4.7	Deflection (Prestress) Vs Relative Humidity	47
Figure 4.8	Prestress Loss Percent Vs Temperature	48
Figure 4.9	Deflection (Self Weight) Vs Temperature	48
Figure 4.10	Deflection (Prestress) Vs Temperature	49
Figure 4.11	Deflection (Total) Vs Temperature	49
Figure 5.1	Development of Restraint Moment	52
Figure 5.2	Construction Sequence of a Two-Span Bridge (Freyermuth, 1969)	52
Figure 5.3	Deformations and Restraint Moments in a Two-Span Continuous Beam (Freyermuth, 1969)	53
Figure 5.4	Restraint Moment Vs Time (CEB-FIP90 Model)	57
Figure 5.5	Restraint Moment Vs Time (ACI 209R-92 Model)	57

Figure 5.6	Restraint Moment Vs Time (Trost's Approach)	58
Figure 5.7	Restraint Moment Vs Time (PCA Method)	58
Figure 5.8	Long-Term Restraint Moment Vs Relative Humidity (CEB-FIP90 Model)	59
Figure 5.9	Long-Term Restraint Moment Vs Relative Humidity (ACI 209R-92 Model)	60
Figure 5.10	Long-Term Restraint Moment Vs Temperature (CEB-FIP90 Model)	60
Figure 5.11	Long-Term Restraint Moment Vs Girder Age at Continuity	61
Figure 5.12	Positive Restraint Moment Connection Details by Embedded Reinforcing Bars (Ma et al., 1998)	62
Figure 6.1	Construction Stages of Simplified Segmental Bridge	65
Figure 6.2	Deformed Shape of the Bridge at 0 Day	67
Figure 6.3	Deformed Shape of the Bridge at 60 Days	68
Figure 6.4	Deformed Shape of the Bridge at 120 Days	68
Figure 6.5	Deformed Shape of the Bridge at 180 Days	69
Figure 6.6	Deformed Shape of the Bridge at 10000 Days	69
Figure 6.7	Schematic of Three Span Bridge	70
Figure 6.8	Cable Profile in the Cantilever Stages	71
Figure 6.9	Cable Profile in the Side Span	71
Figure 6.10	Cable Profile in the Middle Span	72
Figure 6.11	Deflected Shape of Bridge at the End of Stage 1 (60 Days)	73
Figure 6.12	Deflected Shape of Bridge at the End of Stage 2 (120 Days)	73
Figure 6.13	Deflected Shape of Bridge at the End of Stage 3 (180 Days)	74
Figure 6.14	Deflected Shape of Bridge at the End of Stage 4 (240 Days)	74
Figure 6.15	Deflected Shape of Bridge at the End of Stage 5 (300 Days)	75
Figure 6.16	Deflected Shape of Bridge at the End of Stage 6 (360 Days)	75
Figure 6.17	Deflected Shape of Bridge at the End of Stage 7 (420 Days)	76
Figure 6.18	Deflected Shape before the Addition of Closure Segment (Case I)	77
Figure 6.19	Deflected Shape before the Addition of Closure Segment (Case II)	78

II)

Figure 6.20	Deflection at the End of Cantilever Segments Vs Time	79
Figure 6.21	Long-Term Deflection Vs Relative Humidity	79
Figure 6.22	Long-Term Deflection Vs Age of Precast Segment	80
Figure 6.23	Bending Moment Diagram of Completed Bridge	81
Figure 6.24	Typical layout of an externally post-tensioned box girder bridge	82

LIST OF TABLES

Table No.	Title	Page No.
Table 4.1	Validation of Prestress Loss Vs Time	40
Table 4.2	Validation of Deflection (prestress) Vs Time	40
Table 4.3	Percentage Loss After 30 Days Vs Number of Steps	41
Table 4.4	Deflection (Prestress) After 30 Days Vs Number of Steps	41
Table 6.1	Deflection at the End of Cantilever Segments at Different Stages	76
Table 6.2	Effect of Relative Humidity on Long-Term Deflection	80
Table 6.3	Effect of Age of Precast Segment at the Time of Erection on Long-Term Deflection	81

INTRODUCTION

1.1. IMPORTANCE OF PRESTRESSED CONCRETE BRIDGES

The development of transportation and road networks led to the development of bridges. These roads have to cross large rivers and valleys which require bridges to be constructed over them. These bridges were mostly constructed with steel and reinforced concrete. Steel bridges are also constructed nowadays but they require more maintenance. With an increase in the length of span and development of new construction technologies, slender sections for bridge girders were required to reduce the deadweight. It was also required to control the crack width. The inherent weakness of concrete in tension led people to think about improving its performance in tension. This led to the development of prestressed concrete in by a French engineer named Eugene Freyssinet in 1928. But this new type of concrete was not popular in the beginning because very little or no prestress was available after some time due to various types of prestress losses.

The development of high strength concrete associated with significant improvements in the quality of cement and high strength steel paved the way for its widespread application in bridge construction. In 1950's, prestressed concrete came to be used mostly for bridges of ever increasing spans coupled with rapidity and ease of construction and competing its costs with other alternative types like steel and reinforced concrete. In the last fifty years, many researches have been done and many new construction technologies have been developed. Prestressed concrete can be applied in various structures but its most impressive forms are found in long span prestressed concrete bridges. Prestressed concrete bridges are the widely constructed bridges nowadays.

Prestressed concrete bridges have several advantages. The use of high strength concrete and high tensile steel result in slender sections which are aesthetically superior coupled with durability and overall economy. Tensile stresses can be reduced to zero leading to the elimination of cracks. When compared to steel bridges, they require very little periodical maintenance. They are ideally suited for cantilever method of construction, both in situ and precast resulting in faster rate of construction of long span bridges. Post-tensioned bridges find extensive application in long span bridges of variable cross-section. They are ideally suited for composite bridge construction in which precast, prestressed concrete girders support the cast-in-situ deck slab. This type of construction results in minimum disruption of traffic. Combination of prestressed concrete with cable stays is ideally suited for long span bridges. They can also be designed as

partially prestressed concrete bridges with significant savings in the quantity of high tensile steel used in prestressed concrete bridge girders.

1.2. LONG-TERM DEFORMATION IN PRESTRESSED CONCRETE BRIDGES

Long-term deformation refers to the time-dependent deformation caused due the creep and shrinkage of concrete and relaxation of prestressing steel. These time-dependent effects result in gradual increase in the deformation. The calculation of long-term deformation is required to check whether the structure is safe for the serviceability criteria. Eccentrically prestressed concrete girders undergo a net upward deflection or camber after the application of prestressing force. After the stresses at each stage have been checked, the designer has to further ensure that deflection remains within the permissible limits. Thus camber prediction is one of the important aspects of bridge design. Due to the time-dependent properties of concrete and prestressing steel, the deflection goes on increasing with time. The finished surface of the bridge deck should match with the specified elevation in the bridge plan. This will be possible only if camber at each stage of the bridge is predicted accurately. Various methods have been proposed by the researchers for predicting the camber. For modeling the time-dependent phenomena of concrete and steel, the mathematical models given by the researchers and available in the design codes need to be used.

1.2.1. Causes Of Long-Term Deformation

Creep and shrinkage of concrete members and relaxation of prestressing strands cause time-dependent changes in strains in a prestressed bridge system, which result in changes in stress throughout the cross-section. There are several causes of time-dependent changes in strain. These include the creep from girder weight and initial prestressing force, the creep due to the slab weight, and the differential shrinkage between the girder and the deck (ACI 209R-92). Nearly all concrete structures are built in stages, and all have time-dependent effects from creep, shrinkage, and other factors. In many cases, the primary reason to accurately predict time dependent parameters in prestressed concrete members is to determine prestress loss and the deflection of the member.

1.2.2. Issues Involved In Long-Term Deformation Prediction

Various issues are involved in the prediction of long-term deformation of post-tensioned bridge girders. The modulus of elasticity of concrete should be estimated accurately. Appropriate

second moment of area of cross-section should be used depending on whether section is cracked or un-cracked and whether the girder is composite or non-composite. Time-dependent parameters like creep, shrinkage and relaxation depend on materials and environment. The prediction of these parameters requires suitable material models. Deflection due to prestress depends on prestress loss and prestress loss depends on creep and shrinkage of concrete as well as relaxation of prestressing steel while that due to self-weight increases with time due to creep of concrete. Principle of superposition can be used to calculate the net camber by adding the deflections due to self-weight of girder and applied prestress. In addition to material models, prediction of long-term prestress loss and deformation requires suitable analysis approach due to non-linearity involved in the problem.

1.3. LITERATURE REVIEW

1.3.1. Time-Dependent Effects of Creep and Shrinkage of Concrete

Various researchers presented their material models for the prediction of time-dependent properties of concrete and steel. Bazant (2001) presented some of the researchers who worked on the phenomenon of creep and shrinkage namely Glanville, Dischinger, Troxell et al. (1958), Pickett, McHenry, L'Hermite et al. (1965), Arutyunian, Aleksandrovskii (1959), Powers, Hansen and Mattock (1966), Rusch, Neville et al. (1983), Trost, Dilger, Wittman, Hilsdorf, Muller, Huet, Carol, to name a few (RILEM TC-69, 1988). The early mathematical models for creep and shrinkage were formulated with a view to facilitate structural analysis. With the advent of computers it became possible to use any type of model, and thus the recent modeling could focus on representing the experimental data as closely as possible. Some of the papers are briefly reviewed here.

Bazant and Baweja (1995) presented a model for characterization of concrete creep and shrinkage in design of concrete structures (Model B3). The prediction model B3 was calibrated by a computerized data bank comprising practically all the relevant test data obtained in various laboratories throughout the world. The coefficient of variation of the deviations of the model from the data was distinctly smaller than those for the CEB model (1990) and ACI 209 which was developed in the mid-1960's. The new model allowed a more realistic assessment of the creep and shrinkage effects in concrete structures which significantly affect the durability and long-time serviceability of civil engineering infrastructure.

Carreira, Daye and Greening (2000) presented a simplified, comprehensive and rational method to account for the effects of creep and shrinkage in reinforced concrete structures. The

presented method included most of the aspects affecting the effects of creep and shrinkage on concrete structures.

Bazant (2001) summarized various aspects of the prediction of concrete creep and shrinkage. They included the theories of physical mechanism, prediction models, constitutive equations, computational approaches, probabilistic aspects, and research directions. Two new prediction models were also presented.

Gardner and Lockman (2001) presented a procedure for calculating the shrinkage and creep of concrete using the information available at design, namely, the 28-day specified concrete strength, the concrete strength at loading, element size, and the relative humidity.

Huo, Al-Omaishi and Tadros (2001) proposed revised formulae for shrinkage strain and creep coefficient which could be used for both conventional concrete and HPC. It was also indicated that it would be desirable to determine experimentally the shrinkage strain, creep coefficient and modulus of elasticity using the locally available aggregates for the specified HPC mixture proportions.

Lopez, Kahn and Kurtis (2004) developed two high-performance lightweight concrete (HPLC) mixtures with average compressive strengths of 68.5 and 75.4 MPa. Experiments were conducted for determining creep and shrinkage and the results were compared with estimates from 12 models.

Al-Manaseer and Lam (2005) evaluated four shrinkage and creep prediction models, namely ACI 209 model, CEB 90 model, B3 model, and GL 2000 model. Based on the study, it was found that B3 and GL 2000 are best performing models for shrinkage strain prediction while the CEB 90, B3 and GL 2000 models perform best for creep prediction.

Goel, Kumar and Paul (2007) briefly described some of the models for prediction of creep and shrinkage of concrete. Predicted values of creep and shrinkage were compared with the experimental results of **Russel and Larson (1989)** as well as RILEM data bank. Prediction of creep and shrinkage by GL2000 model was found to be closest to the experimental results.

Karthikeyan et al. (2008) carried out and presented the results of experiments on creep and shrinkage properties of concrete. The test results obtained were compared to different models to determine which model was better one. The CEB-FIP90 model was found better in predicting time-dependent strains and deformations of concrete prepared in Indian environment and using local practices. They also developed Artificial Neural Network (ANN) model which will serve as a more rational and computationally efficient model in predicting creep coefficient and shrinkage strain.

1.3.2. Prediction of Long-Term Prestress Losses

Various loss prediction methods have been developed by the researchers. The accurate estimation of prestress loss is difficult as it requires more precise knowledge of material properties as well as interaction between creep, shrinkage of concrete and relaxation of prestressing steel. The various methods for the estimation of long-term prestress loss can be classified as time-step methods, refined methods and lump-sum methods. The time-step method is a step-by-step method implemented by using computer programs. The entire time-interval is divided into sufficient number of time-steps and prestress loss is calculated for each stage. The prestress loss at the end of the interval is obtained by the summation of losses in all the time-steps. In refined method, individual components of prestress losses are computed separately and then summed up to obtain the total loss. Lump-sum method utilizes the values and trends developed from various parametric studies conducted on prestress losses of different kinds of prestressed beams under average conditions.

PCI Committee (1975) presented report on the calculation of prestress losses. The report summarized data on creep and shrinkage of concrete and relaxation of steel and presented both general and simplified design procedure for using these data in estimating loss of prestress after any given time-period. Further, detailed design examples were presented for pre-tensioned and post-tensioned structures to explain the procedure.

Tadros et al. (2003) conducted a detailed research with an objective to develop design guidelines for estimating prestress losses in high strength pre-tensioned bridge girders. The report named as NHCRP report 496, aimed in development of new guidelines and formulae for prestress losses due to limitations in the current methods used in the estimation of losses. A thorough study was made on the time-step method, lump-sum method and refined method 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 estimate of prestress losses. Time-step method and refined method were found to have good correlation with experimental test results. Numerical examples were also provided to demonstrate the applicability of the proposed loss prediction methods.

Youakim A S, Ghali, Hida and Karbhari (2007) presented an analytical method to predict the long term prestress losses in precast pre-tensioned or post-tensioned concrete members. Design aids are presented for cast-in-place post-tensioned concrete bridges. The method was based on equilibrium and compatibility principles of solid mechanics and can be used for multi-stage loading and prestressing. The authors also showed that the presence of non-

prestressed steel has significant effects on the long term deformation and the compressive stress remaining in concrete after prestress losses.

1.3.3. Long-Term Deformation of Determinate PSC Bridge Structures

Aalami B O (1998), in his article explains the time-dependent parameters critical to the performance of post-tensioned concrete structures, as treated by the design codes and practiced by design engineers, with an emphasis on recent developments in the field. The author also introduces the basics for the practical implementation of creep, shrinkage, aging of concrete and relaxation in prestressing tendons in the context of code requirements. The application of laboratory generated data is also discussed for HPC concrete. Finally, the paper outlines a modern analytical technique for the structural modeling of prestressing tendons and its advantages over previous method. The new method eliminates the need for a separate analysis to determine immediate and long term losses in prestressing because the losses become an integral part of the overall structural analysis.

Stallings et al. (2003) used the incremental time-step method to calculate the girder strains, camber and prestress losses up to the time of deck construction. The authors also compared the results with the field measurements which showed good agreement with the calculated camber. Further they concluded that current analytical techniques can result in accurate prediction of camber and prestress losses for HPC girders if the material properties used in the analysis are representative of the actual concrete used in girder production.

Bazant, Yu, Li, Klein and Kristek (2010) predicted the deflections of Koror-Babeldaob (KB) Bridge in the Republic of Palau and compared the deflections obtained with the main existing creep and shrinkage prediction models currently used in practice.

Tadros, Fawzy and Hanna (2011) addressed several issues related to prediction, design, and construction to accommodate variability in prestressed concrete girder camber. The authors pointed out that variations in camber become more significant as the use of high-strength concrete, longer spans and more heavily prestressed concrete girders continues to increase.

1.3.4. Long-Term Deformation of Indeterminate PSC Bridge Structures

1.3.4.1. Precast Prestressed Concrete Girders Made Continuous Using Cast-in-Place Deck and Continuity Diaphragm

In the early 1960's, Mattock undertook a series of studies on precast/prestressed concrete bridges. His fifth study in the series, "Creep and Shrinkage Studies", consisted of analysis and half-size testing of two spans of precast girders with a cast-in-place deck and a continuity

diaphragm. Two methods of continuity for positive moment (tension at the bottom of the girder) at the diaphragm were studied: straight reinforcing bars welded to angles and hooked reinforcing bars with a tight bend radius. Mattock concluded that the welded straight bars provided a higher degree of continuity. Another two span bridge was evaluated without a positive moment connection. The study concluded with a method to design for the effects of creep and shrinkage. It also recommended that when designing hooked bars for a positive moment connection to carry time-dependent effects and live loads, the design strength should be limited to 60 percent of the yield strength.

Freyermuth (1969) compiled the results of Mattock's studies and presented a complete design procedure known as PCA Method. The author demonstrated that considering continuity in a bridge constructed of precast, prestressed single-span units required inclusion of the effects of creep and shrinkage. The complete design example illustrated necessary design procedures for continuity considerations. The example includes the effect of creep and shrinkage with the development of positive restraint moments at the intermediate support of a two span bridge. The discussion includes the details of the positive moment connection and recommendations for the design of the connection. The design of negative moment reinforcement over the piers is also addressed.

Oesterle et al. (1989) published NHCRP Report 322. This study was purely analytical and a revised analysis method was developed to predict time-dependent restraint moments. This method, based on the PCA method, had several improvements including a time-step analysis and is commonly known as Construction Technology Laboratories (CTL) method. The computer program BRIDGERM was developed to calculate restraint moment by CTL method. The study concluded that positive moment connections between the ends of adjacent girders in the closure diaphragm are difficult, time-consuming and costly to install and furthermore the connections add no structural benefit.

Peterman and Ramirez (1998) investigated restraint moments on bridges with full-span prestressed concrete form panels. They proposed a modification to the restraint moment calculation by PCA and CTL methods. Their modified method, referred to as the P-method, resulted in better correlation of results for the two full-scale bridges tested in the study. Because the method is intended for bridges with precast/prestressed form panels, it is not considered in the study presented here. However, the paper presented a good overview of various methods for calculating restraint moments in any precast/prestressed bridge type.

Ma Z et al. (1998) illustrated the time-dependent effects of different continuity methods and construction sequences on the behaviour of bridge system. Recommendations for achieving

continuity and for desirable construction sequence were presented to achieve favourable performance of bridges made continuous for deck loads. A new continuity detail using high strength threaded rods was also presented. The new continuity detail was shown to eliminate the potential for bottom concrete creep restraint cracking at pier locations and to increase the span capacity by as much as 20 percent.

Mirmiran (2001) proposed another method for calculating restraint moments in precast/prestressed girder bridges. Their method involves considering properties of the bridge as non-linear along the length of the bridge, due to varying amounts of reinforcements and cracking. To account for these non-linearities, the method incorporates a moment-curvature analysis at each time-step. Analytical results of the non-linear method were compared to results from previous physical testing by Mattock and results from the CTL method.

McDonagh and Hinkley (2003) presented analytical studies of Washington State Department of Transportation (WSDOT) standard precast, prestressed concrete girders and their design for continuity. These studies indicated that deeper girders with longer spans do not develop large positive restraint moments from creep and shrinkage effects. The paper also introduced the computer program RMCalc developed by the authors which simplifies the calculation of the restraint moments. This program can greatly aid the engineer in designing precast prestressed concrete girders for continuity, thereby providing greater structural efficiency in the bridge and significant cost savings.

1.3.4.2. Segmental Bridges

Tadros, Ghali and Dilger (1979) presented a step-by-step computer method for predicting the deflection and stress distribution due to creep and shrinkage of concrete and relaxation of prestressing steel. The computer program accounted for the presence of the non-prestressed steel, the difference in ages of concrete segments, the multiple stages in which the external loads and prestressing are applied, and the changes in geometry and support conditions as the construction progresses. The authors applied the proposed method to a three span continuous bridge and presented the results to indicate the significance of time-dependent effects. The longitudinal view and section of typical segmental bridge is shown in Fig. 1.1.

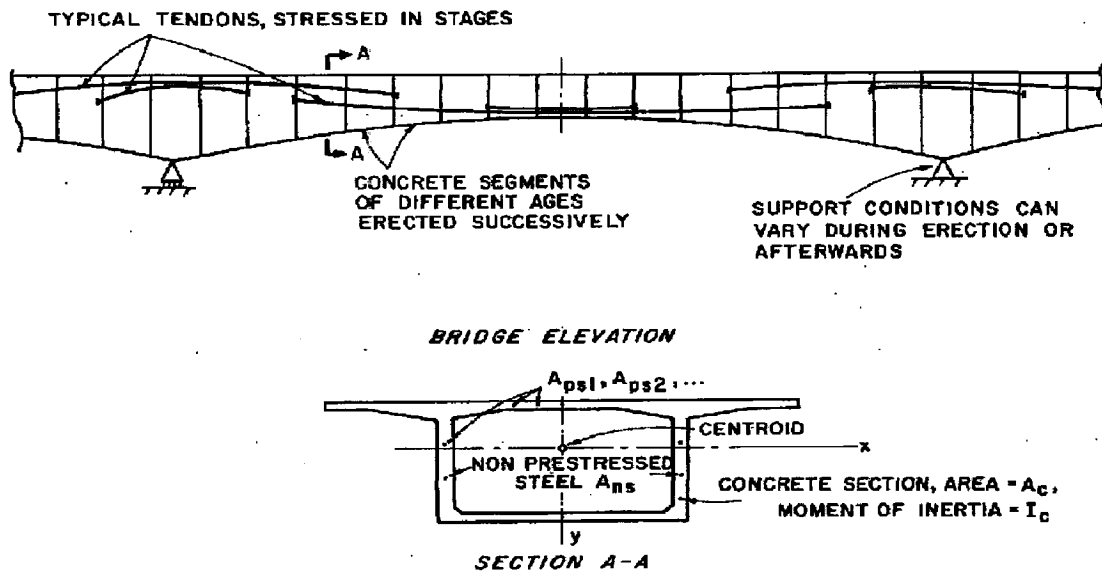


Fig. 1.1. Longitudinal View and Section of Typical Segmental Bridge (Tadros et al., 1979)

The step-by-step method described by authors gives increments of displacements occurring at each stage of construction. Fig. 1.2 shows the horizontal and vertical translations and the rotation of Joint B just before erecting Segment 2. The displacement increments in Stage 2, caused by the new prestressing and the self weight of Segment 2, are to be measured from a displaced "datum" AB' C' to give the total displacements measured from the horizontal ABC.

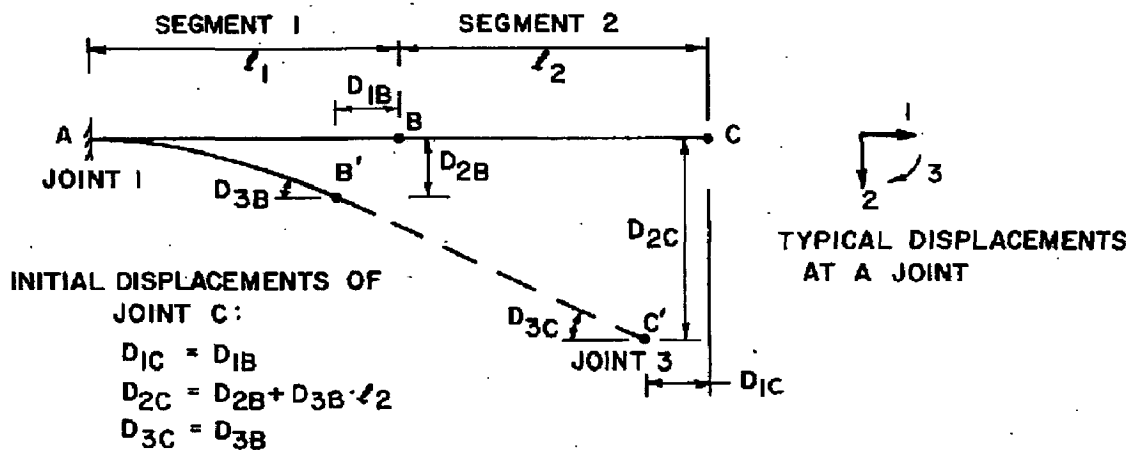


Fig. 1.2. Joint Displacements before Application of Loads of Segment 2 in Match-Cast Cantilever Construction Method (Tadros et al., 1979)

Fig. 1.3 shows the time-dependent deflection of a cantilever built in four stages. For clarity, the deflected shape of each segment is represented by a straight line and no horizontal displacements are shown. At the end of Stage 4, and just before connecting the cantilever with the remainder of the structure, the deflected shape of the cantilever is given by the line AB'C'D'E'.

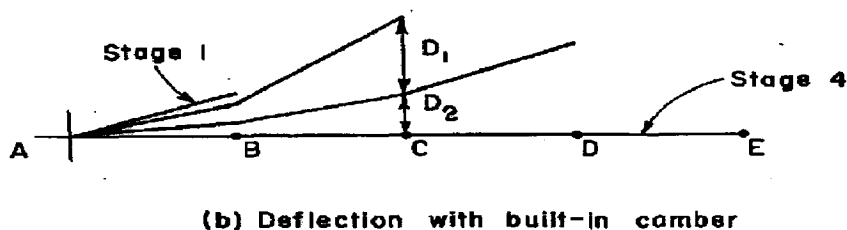
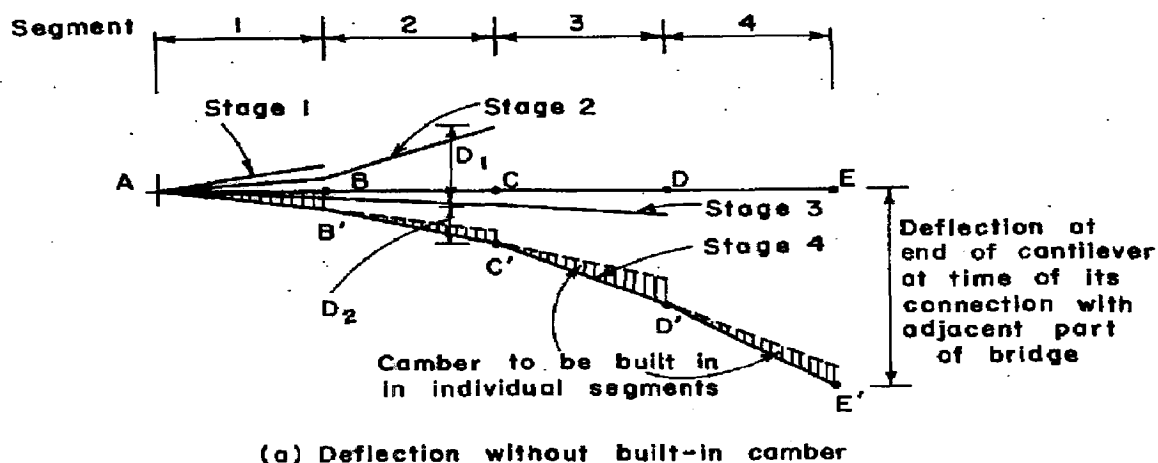


Fig. 1.3. Deflection of a Four-Segment Cantilever (Tadros et al., 1979)

Shushkewich K W (1986) presented a computer program for the time-dependent analysis of segmental bridges. Loadings considered at each stage of construction includes self-weight, time-dependent effects, temperature and construction loads. The program was intended to be used for routine day-to-day design. The author used the direct stiffness method for the analysis of stresses and deformations of segmental bridges. The usefulness of the program was illustrated through five numerical examples.

Herbert J T (1990) presented a methodology and computer program for the analysis of stage constructed bridges and two dimensional prestressed concrete structures. The method uses step-by-step superposition of strains and accounts for time-dependent effects including creep, shrinkage and relaxation. Structural elements comprise supports, hinges and temporary links as well as the concrete segments, and can be erected in any logical order. The computer program is

used to illustrate the method with an example problem involving a simple three-span bridge. An important feature of the method is that it can be applied independently of a particular code of practice and computer program demonstrates this by incorporating the provisions of six different codes. The author based the computer approach on the step-by-step superposition of strains as described by Tadros, Ghali and Dilger.

Guo-Jing HE, Yuan-yuan LI, Zhong-quan Zou and Ling-liang Duan (2008) investigated the effects of concrete creep on the pre-camber of a long span prestressed concrete continuous rigid-frame bridge constructed by cantilever casting method. The difference of creep coefficients calculated with two Chinese codes, CC-1985 and CC-2004 was discussed. Based on the calculations, the pre-camber of a prestressed concrete continuous rigid-frame box bridge was computed for construction control purpose.

Malm R and Sundquist H (2010) presented the time-dependent analysis of segmentally constructed balanced cantilever bridges. The analysis presented in the paper was performed with finite element (FE) software Abaqus/Standard 6.7. The modeling approach used was using three-dimensional shell elements. Three-dimensional element is chosen to capture the effect of shear lag in box section. The shell elements were used in the main and two adjacent spans. The remaining spans and pier were modeled with beam elements.

1.3.5. Conclusions from Literature Review

The literature review on the material models and long-term term behaviour of determinate and indeterminate bridge structures has been presented. Based on the literature review, it can be concluded that creep, shrinkage and relaxation affect the long-term deformation as well as prestress losses significantly. In case of precast prestressed concrete girders made continuous, restraint to the time-dependent deformation gives rise to the restraint moment at intermediate support. Therefore, in the design of the diaphragm in such bridges, the design engineer needs to consider the effect of long-term restraint moment at intermediate support. In case of segmental bridges, the long-term deformation has to be considered at each stages of construction and service life. Prediction of these quantities depends on material models as well as the analysis approach to take care of non-linearity involved in the problem. In this context, in the present study, incremental time-step method is used to develop a program for prediction of long-term deformation for a simply-supported post-tensioned girder. Two different material models, CEB-FIP90 and ACI 209R-92 are incorporated and numerical studies are carried out to develop quantitative information for the effect of change in material models. For the precast prestressed concrete girders made continuous, a computer program has been written in MATLAB to assess

the restraint moment and numerical studies have been performed to evaluate the effect of various parameters. For this also, provisions of CEB-FIP90 and ACI 209R-92 codes have been used. The analysis of segmental bridges has been done using the LUSAS software. Numerical studies have been performed to show the effect of various parameters.

1.4. OBJECTIVE AND SCOPE

The objective of this dissertation is to predict the long-term deformation of indeterminate post-tensioned bridge structures. The indeterminate bridges include continuous and rigid frame bridges. They can be constructed either by making precast, prestressed concrete girders continuous by cast-in-situ deck slab and diaphragm or by segmental construction. To realize the objective the scope of the dissertation is defined as follows:

- Literature review.
- Development of a computer program for the prediction of long-term prestress loss including two material models namely CEB-FIP90 and ACI 209R-92
- Development of a computer program for the prediction of long-term deformation of simply-supported post-tensioned bridge girder based on incremental time-step method including two material models namely CEB-FIP90 and ACI 209R-92.
- Validation of developed programs.
- Development of computer program for the assessment of restraint moment for precast, prestressed bridge girders made continuous using cast-in-place diaphragm.
- Prediction of long-term deformation of segmental bridges using available software.

1.5. GENERAL APPROACHES

- Prediction of long-term deformation of simply-supported post-tensioned bridge girder:
A computer program based on incremental time-step method has been written in MATLAB to predict the long-term deformation of simply-supported post-tensioned bridge girder. The deflection has been calculated for the mid-span. Single-stage post-tensioning has been considered. Self-weight of girder and prestressing have been considered for the calculation of deflection. Gross moment of inertia of cross-section has been used as the section has been assumed to be uncracked. Two material models for creep and shrinkage of concrete namely CEB-FIP90 and ACI 209R-92 have been incorporated in the program. Relaxation formulae for low relaxation and stress-relieved strands have been used to model the time-dependent effects of

relaxation of prestressing steel. Numerical studies have been carried out to show the effect of various parameters on the long-term deformation.

- Assessment of continuity moment for precast girders made continuous using cast-in-place diaphragm:

Restraint moment at intermediate support has been assessed for the precast, prestressed bridge girders made continuous using cast-in-place diaphragm. The casting of deck slab is not done thereby eliminating the need to consider restraint moment due to differential shrinkage. Thus restraint moment due to creep only has been considered. A computer program has been written in MATLAB based on PCA method and Trost's approach for assessing restraint moment at support. Only two-span bridge has been considered. Numerical studies have been performed to show the effect of various parameters like girder age at the time of establishment of continuity, environmental parameters etc. Long-term deformation has not been predicted as they are restrained due the introduction of indeterminacy.

- Prediction of long-term deformation of segmental bridges:

Long-term deformation of segmental post-tensioned has been predicted using LUSAS version 14 software. The deflections have been presented for each stage of construction and service life. A three-span continuous rigid frame bridge constructed by segmental cantilever construction has been analysed considering dead load and prestress. Only CEB-FIP90 model for creep and shrinkage of concrete has been considered in the LUSAS modeling. The effect of relative humidity age of precast segment at the time of loading on long-term deformation has been presented. The variation of bending moment diagram with time has also been presented.

1.6. ORGANISATION OF THE THESIS

This thesis has been divided into seven chapters and one appendix.

Chapter 1: INTRODUCTION

Chapter 2: PRESTRESSED CONCRETE: BASIC CONCEPTS

Chapter 3: PREDICTION OF LONG-TERM BEHAVIOUR

Chapter 4: LONG-TERM RESPONSE PREDICTION OF DETERMINATE BRIDGE STRUCTURES

Chapter 5: LONG-TERM RESPONSE PREDICTION OF INDETERMINATE BRIDGE STRUCTURES: PRECAST PRESTRESSED BRIDGE GIRDERS MADE CONTINUOUS

Chapter 6: LONG-TERM RESPONSE PREDICTION OF INDETERMINATE BRIDGE STRUCTURES: SEGMENTAL BRIDGES

Chapter 7: DISCUSSION OF RESULTS AND CONCLUSION

Appendix I: LONG-TERM PRESTRESS LOSS CALCULATION USING IRC:18-2000

PRESTRESSED CONCRETE: BASIC CONCEPTS

2.1. GENERAL

Prestressing means intentional application of load on a system to resist the internal stresses due to external loads. Concrete is strong in compression but weak in tension. When tensile stresses are applied on a concrete member, cracks develop and the strength of concrete is not utilized efficiently. In a prestressed concrete, compressive stresses are applied beforehand which nullifies or reduces the tensile stresses occurring due to service loads. Thus cracks are eliminated or reduced and strength of the concrete can be utilized efficiently. The elimination of cracks is highly beneficial for structures such as liquid storage tanks and nuclear structures. Prestressed concrete structures have several other advantages over reinforced concrete structures. Larger span to depth ratios are possible in case of prestressed concrete bridge girders and beams of a building. Prestressed concrete is also suitable for precast construction which can result in better quality control. Shear and torsional capacity of prestressed concrete members are also improved compared to reinforced concrete members.

Prestressing can be classified in a number of ways which are as follows:

- Pre-tensioning or post-tensioning
- Linear or circular prestressing
- External or internal prestressing
- Partial prestressing, limited prestressing or full prestressing

In prestressed concrete works, high strength materials have to be used. High strength steel has to be used so that required prestress is available even after the prestress losses. High strength concrete should be used to prevent failure of concrete especially at anchorage zones due to the transfer of heavy prestressing force of concrete. Further, in high strength concrete, less creep and shrinkage occurs which result in less loss of stress in prestressing steel.

2.2. SHORT-TERM PRESTRESS LOSSES

Short-term or immediate loss is the reduction in the level of prestress in the strands immediately after applying the jack pressure. It is caused due to the following factors.

- Elastic shortening
- Friction
- Anchorage slip

2.2.1. Elastic shortening

Pre-tensioned Members: In case of pre-tensioned members when tendons are cut after the concrete has attained required strength, the concrete undergoes shortening due to prestress. The tendon also shortens by the same amount which leads to loss of prestress. This type of loss is termed as elastic shortening.

Post-tensioned Members: In case of only one tendon, there is no loss due to elastic shortening as the prestress is recorded by the gauge after the shortening has occurred. On the other hand, if there are more than one tendon, if the tendons are stressed one by one, there is a loss in the previously stressed tendons due to the stressing of a particular tendon. It implies that loss is largest for the tendon which is stressed first and zero for the last stressed tendon.

2.2.2. Friction

When the tendon is stretched, the friction occurs between the steel and concrete interface due to curvature and wobble of the tendon which leads to a drop in prestress along the tendon from the jacking end. This loss does not occur in case of pre-tensioned members because no concrete is present at the time of stretching of tendon. The loss due to friction is calculated from the following formula:

$$\delta f = f_o \left(1 - e^{-(\mu\alpha + kx)} \right) \quad \text{Eqn. (2.1)}$$

where,

δf = Loss due to friction

f_o = Initial prestress

α = cumulative angle (in radians) by which tangent to the tendon profile has changed between the jacking end and location x

μ = Friction coefficient

k = Wobble coefficient

x = Distance from the jacking end

For small values of $(\mu\alpha + kx)$, Eqn. (2.1) can be written as:

$$\delta f = f_o (\mu\alpha + kx) \quad \text{Eqn. (2.2)}$$

2.2.3. Anchorage Slip

The loss due to anchorage slip occurs in post-tensioned member. It occurs because of reduction of length of the tendon due to the slip of wedges through a distance before they fit into

the conical space and the movement of the anchorage block before it settles on the concrete. The total anchorage slip depends on the type of the anchorage system. There occurs a reverse friction due to the settling of the anchorage block. Hence the effect of anchorage slip is present upto a certain length. This length is called l_{set} .

$$l_{set} = \sqrt{\left(\frac{\Delta_s E_p A_p}{P_0 \eta}\right)} \quad \text{Eqn. (2.3)}$$

where,

l_{set} is the length of slip travel.

Δ_s is the anchorage slip.

E_p is the modulus of elasticity of prestressing steel.

A_p is the area of prestressing steel.

$P_0 \eta$ is the loss of prestress per unit length.

2.3. LONG-TERM PRESTRESS LOSSES

Long-term prestress losses are caused due to creep and shrinkage of concrete and relaxation of prestressing steel. These losses are discussed briefly as follows:

2.3.1. Losses due to Creep of Concrete

Creep is defined as deformation of concrete under sustained loading or increase in strain under constant stress. Creep deformation depends on various factors like strength of concrete, age of concrete at the time of loading, applied stress level, humidity and atmospheric conditions, time at which creep deformation is measured. This creep deformation or strain in concrete causes loss in stress in prestressing steel. The calculation of creep strain is complex due to the variation of concrete strain due to time-dependent parameters and along the length of the beam. At low level of stress, creep deformation or strain is proportional to the elastic strain, the proportionality constant being creep coefficient, ϕ_t , which can be estimated by using various creep models. Thus prestress loss due to creep at any time can be estimated as:

$$\Delta f_{pCR} = \phi_t \frac{E_{ps}}{E_c} f_{cs} \quad \text{Eqn. (2.4)}$$

where,

Δf_{pCR} is the prestress loss due to creep of concrete.

ϕ_t is the creep coefficient at time t.

E_{ps} is the modulus of elasticity of prestressing steel.

E_c is the modulus of elasticity of concrete

f_{cs} is the stress of concrete at the level of prestressing steel.

If there is a variation in stress in concrete at various cross-sections, average value should be taken taking suitable number of cross sections. The entire time-interval should be divided into sufficient number of time-steps and prestress should be calculated by deducting the losses from the previous step. This stress should be used to calculate the stress in concrete at the level of prestressing steel and strain increment is calculated by dividing it by modulus of elasticity of concrete. Average of stresses at the beginning and end of the interval can also be taken for the calculation of incremental strain. Eqn. (2.4) can be applied at the beginning and end of the time-step to get the prestress loss due to creep in that time-step. The summation of losses in all the time-steps gives the loss due to creep at the end of the interval.

2.3.2. Losses due to Shrinkage of Concrete

Shrinkage is defined as the decrease in volume of concrete with time. It depends on various factors like the amount of cement in concrete, relative humidity, type of cement and water-cement ratio. The shrinkage strain can be estimated by using various shrinkage models. The loss due to shrinkage of concrete can be calculated as follows

$$\Delta f_{pSH} = \epsilon_{SH} E_{ps} \quad \text{Eqn. (2.5)}$$

where,

Δf_{pSH} is the prestress loss due to shrinkage of concrete.

ϵ_{SH} is the shrinkage strain.

E_{ps} is the modulus of elasticity of prestressing steel.

As in case of the calculation of creep loss at the end of a time-interval, the entire interval should be divided into sufficient number of time-steps and shrinkage incremental strain should be calculated for each time step. Eqn. (2.5) should be applied to calculate the shrinkage loss in a particular time-step. The prestress loss at the end of the interval is obtained by summing the shrinkage losses of each time step.

2.3.3. Losses due to Relaxation of Prestressing Steel

Relaxation is defined as the change in stress in prestressing steel under a constant strain. It is a function of applied stress level, temperature and relative humidity. Relaxation loss can be estimated as follows:

$$\Delta f_{pR} = f_{pi} \frac{\log 24t}{k} \left(\frac{f_{pi}}{f_{py}} - 0.55 \right) \quad \text{Eqn. (2.6)}$$

where,

Δf_{pR} is the prestress loss due to relaxation of prestressing steel.

f_{pi} is the initial stress in prestressing steel.

t is the time in days.

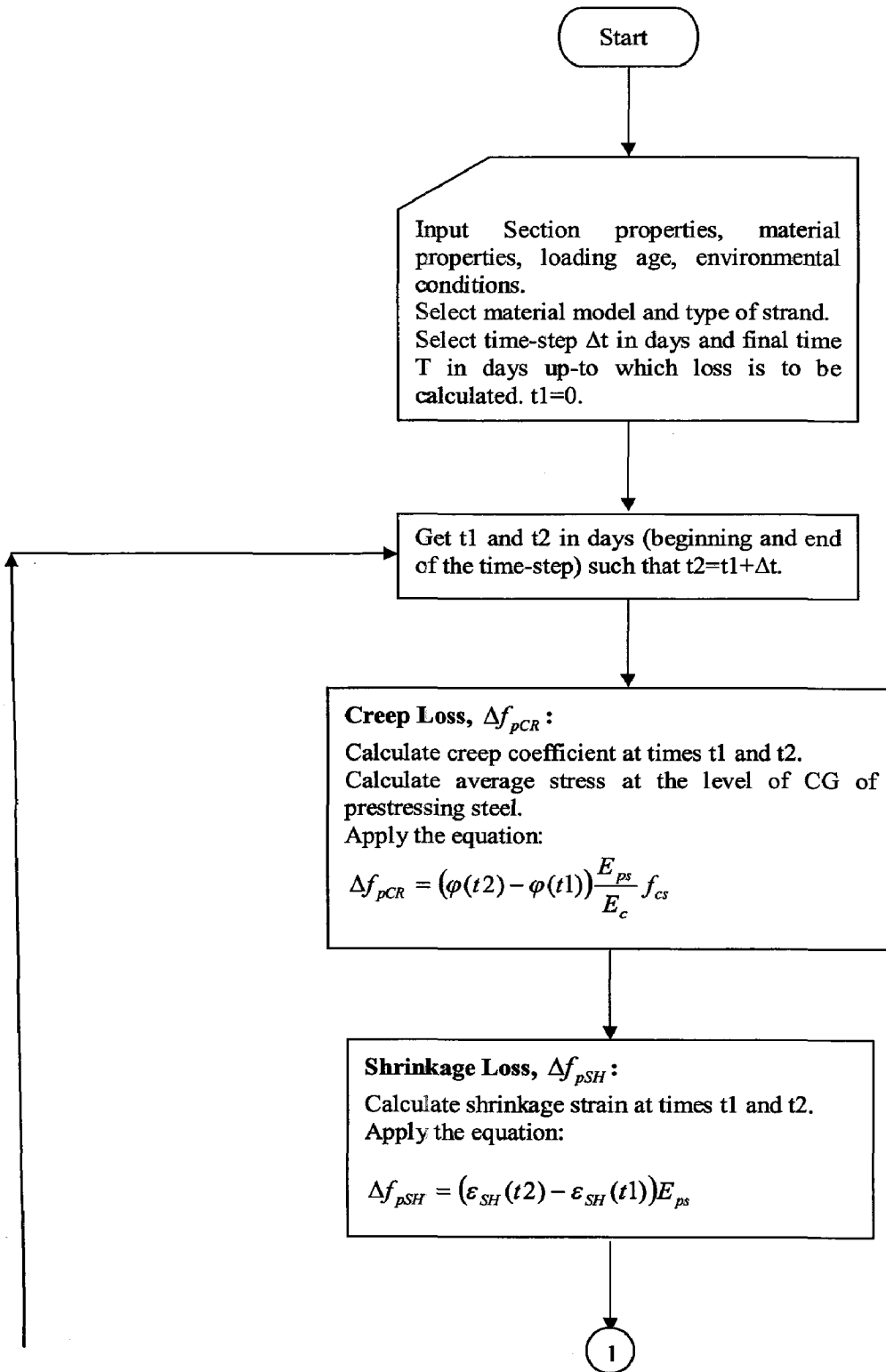
k is equal to 10 for stress-relieved strands and equal to 45 for low-relaxation strands.

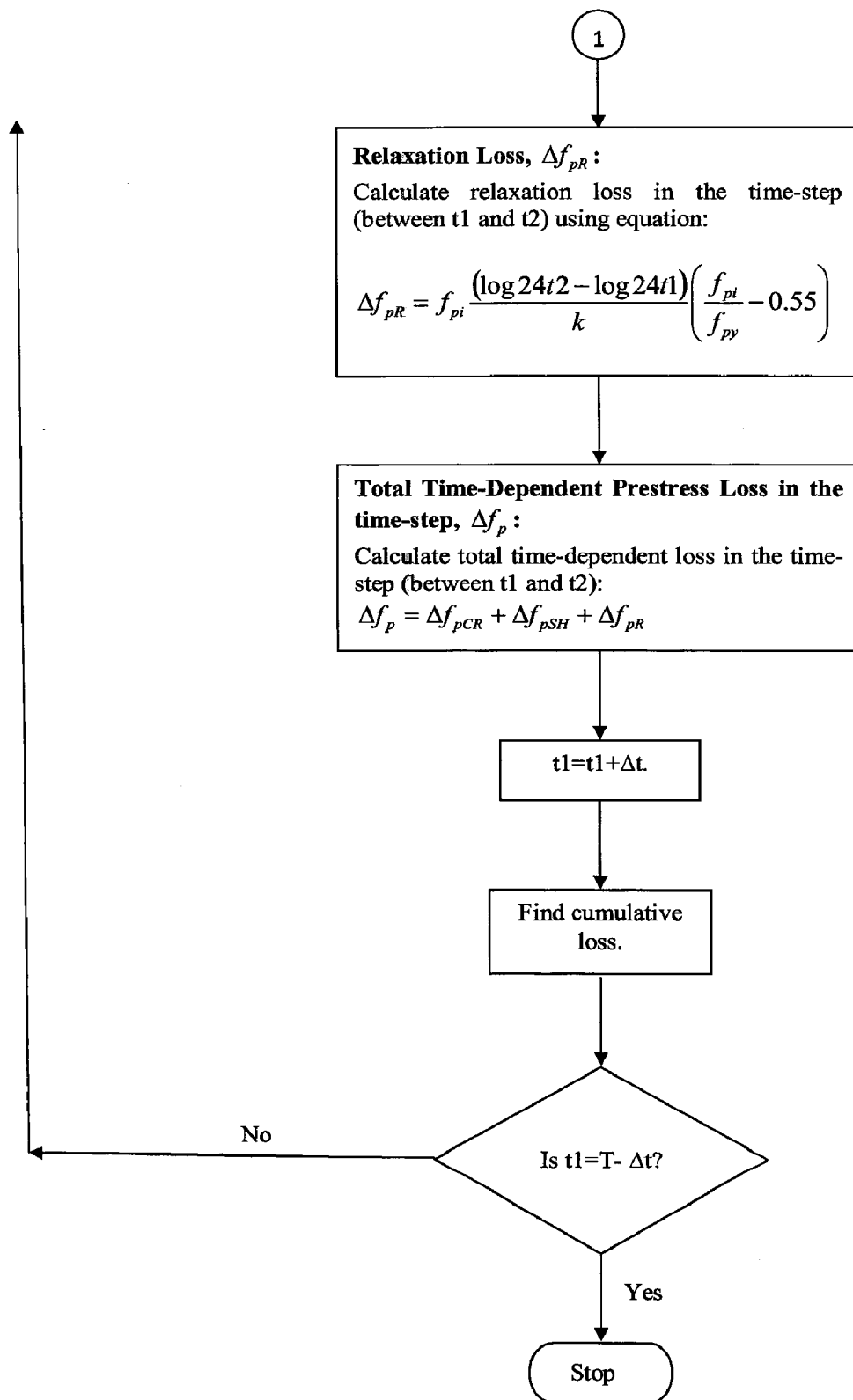
f_{py} is the yield strength of prestressing steel or can be taken equal to 0.9 times the ultimate tensile strength (0.9*UTS).

Similar to the calculation of creep and shrinkage loss, prestress loss due to relaxation can also be calculated for each time-interval and summation can be done to get the prestress loss due to relaxation at the end of time-interval.

2.3.4. Flowchart for the Calculation of Time-Dependent Prestress Losses

The flowchart for the calculation of time-dependent prestress losses due to creep and shrinkage of concrete and relaxation of prestressing steel are presented here:





2.4. MATERIAL MODELS

For the prediction of creep and shrinkage of concrete, CEB-FIP90 and ACI 209R-92 have been used. The relaxation formula for stress-relieved and low relaxation strands have been used in the analysis. These models are being presented below.

2.4.1. CEB-FIP90

According to CEB-FIP Model Code 1990, the total strain at time t , $\varepsilon_c(t)$, of a concrete member uniaxially loaded at time t_0 with a constant stress $\sigma_c(t_0)$ may be expressed as follows:

$$\varepsilon_c(t) = \varepsilon_{ci}(t_0) + \varepsilon_{cc}(t) + \varepsilon_{cs}(t) + \varepsilon_{cT}(t) = \varepsilon_{c\sigma}(t) + \varepsilon_{cn}(t) \quad \text{Eqn. (2.7)}$$

where,

$\varepsilon_{ci}(t_0)$ is the initial strain at loading

$\varepsilon_{cc}(t)$ is the creep strain at time $t > t_0$

$\varepsilon_{cs}(t)$ is the shrinkage strain

$\varepsilon_{cT}(t)$ is the thermal strain

$\varepsilon_{c\sigma}(t)$ is the stress dependent strain: $\varepsilon_{c\sigma}(t) = \varepsilon_{ci}(t_0) + \varepsilon_{cc}(t)$

$\varepsilon_{cn}(t)$ is the stress independent strain: $\varepsilon_{cn}(t) = \varepsilon_{cs}(t) + \varepsilon_{cT}(t)$

This model does not predict local rheological properties within the cross-section of a concrete member such as variations due to internal stresses, moisture states or the effects of local cracking. At the same time, this model is also not applicable to:

1. Concrete subjected to extreme temperatures, high (e.g. nuclear reactors) or low (e.g. LNG tanks)
2. Very dry climatic conditions (average relative humidity RH < 40%)
3. Structural lightweight aggregate concrete.

This prediction model only predicts the mean behaviour of a concrete cross-section. Unless special provisions are given, this model is valid for ordinary structural concrete f_{ck} between 12 MPa to 80 MPa and only subjected to a compressive stress $|\sigma_c| < 0.4f_{cm}(t_0)$ at an age of loading t_0 and exposed to mean relative humidities in the range of 40 to 100% at mean temperature 5°C to 30°C. The model is also directed towards the prediction of creep of concrete subjected to compressive stresses.

In this model, creep is assumed to be linearly related to stress within the range of service stresses $|\sigma_c| < 0.4f_{cm}(t_0)$. For constant stress applied at time t_0 , this leads to:

$$\beta_H = 150 \left\{ 1 + \left(1.2 \frac{RH}{RH_0} \right)^{18} \right\} \frac{h}{h_0} + 250 \leq 1500 \quad \text{Eqn. (2.17)}$$

b) Shrinkage

The total shrinkage strain may be calculated from

$$\varepsilon_{cs}(t, t_s) = \varepsilon_{cso} \beta_s (t - t_s) \quad \text{Eqn. (2.18)}$$

where,

ε_{cso} is the notional shrinkage coefficient

β_s is the coefficient to describe the development of shrinkage with time

t is the age of concrete (days)

t_s is the age of concrete (days) at the beginning of shrinkage

The notional shrinkage coefficient may be obtained from

$$\varepsilon_{cso} = \varepsilon_s(f_{cm}) \beta_{RH} \quad \text{Eqn. (2.19)}$$

with

$$\varepsilon_s(f_{cm}) = [160 + 10 \beta_{sc} (9 - f_{cm} / f_{cmo})] \times 10^{-6} \quad \text{Eqn. (2.20)}$$

where,

β_{sc} is a coefficient which depends on the type of cement: $\beta_{sc} = 4$ for slow hardening cements SL,

$\beta_{sc} = 5$ for normal or rapid hardening cements N and R and $\beta_{sc} = 8$ for rapid hardening high strength

cements RS

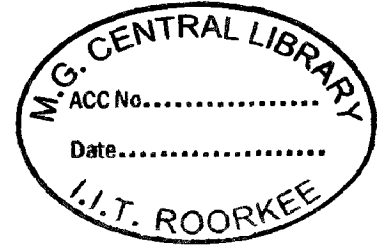
$$\beta_{RH} = -1.55 \beta_{sRH} \quad 40\% \leq RH < 99\% \quad \text{Eqn. (2.21)}$$

$$\beta_{RH} = +0.25 \quad RH \geq 99\% \quad \text{Eqn. (2.22)}$$

where,

$$\beta_{sRH} = 1 - \left(\frac{RH}{RH_0} \right)^3 \quad \text{Eqn. (2.23)}$$

The development of shrinkage with time is given by



2.4.3. Relaxation Model

The relaxation formulae used are as follows:

a) Low Relaxation Strands

$$\frac{f_{pR}}{f_{pi}} = \frac{\log(24t)}{45} \left(\frac{f_{pi}}{f_{py}} - 0.55 \right) \quad \text{Eqn. (2.45)}$$

b) Stress Relieved Strands

$$\frac{f_{pR}}{f_{pi}} = \frac{\log(24t)}{10} \left(\frac{f_{pi}}{f_{py}} - 0.55 \right) \quad \text{Eqn. (2.46)}$$

where,

f_{pR} is the remaining stress after relaxation

f_{pi} is the initial prestress applied

t is the time in days after application of prestress

2.5. LONG-TERM DEFORMATION

Long-term deformation refers to the time-dependent deformation caused due to the effects of creep and shrinkage of concrete and relaxation of prestressing steel. These time-dependent effects results in gradual increase in the deformation. The calculation of long-term deformation is required to check whether the structure is safe for the serviceability criteria. Long-term effects due to creep and shrinkage of concrete and relaxation of prestressing steel cause significant increase in deflection and result in overstressing in the concrete and the reinforcement.

2.6. SUMMARY

In this chapter, basic concepts of prestressed concrete have been presented. A flow chart for the calculation of long-term prestress loss has been given. Two material models for prediction of creep and shrinkage of concrete namely CEB-FIP90 and ACI 209R-92 have been presented. Relaxation formulae for low relaxation and stress-relieved strands have been given.

PREDICTION OF LONG-TERM BEHAVIOUR

3.1. GENERAL

Prestressed concrete girders are more slender than reinforced concrete girders. The prediction of deflection demands special attention due to the high span-to-depth ratios of prestressed concrete girders. For the girder which is designed as fully prestressed concrete member, the problem may be due to upward deflection or camber which increases with time due to the effect of creep of concrete. The prestress loss which increases with time also affect the development of camber with time. Camber of bridge girders causes uneven road profile which affects the riding quality.

For a typical post-tensioned bridge girder, the self weight of girder and super-imposed dead load causes downward deflection which increases with time due to the effect of creep of concrete. When prestress is applied, it produces upward deflection in the girder. The prestress gradually reduces with time due to the time-dependent phenomena of creep and shrinkage of concrete and relaxation of prestressing steel. Due to the loss of prestress, initial camber due to prestress reduces with time. But the creep strains in the concrete increases the negative curvature associated with prestress and hence increases the camber. Depending on the details of design and material properties, either of these two effects may predominate. The increase of camber due to creep associated with prestress is usually more than the decrease in camber due to loss of prestress. The resultant of these effects gives the upward deflection due to prestress. For a typical post-tensioned bridge girder, the upward deflection due to prestress is generally more than the downward deflection due to self weight resulting in the net upward camber.

In case of post-tensioned bridges in which precast concrete girders are made continuous due to the continuity diaphragm and slab, initially the girders behave as simply supported up to the the time the continuity diaphragm and slab attains sufficient strength. The girders camber upward unrestrained up to this time. The hardening of the slab and diaphragm restrains further increase in curvature which gives rise to restraint moment at the support. This moment increases with time due to the time-dependent phenomena of concrete. The design of continuity diaphragm should include these effects.

For bridges constructed by using segmental cantilever construction, comprehensive calculations should be performed for every construction stage to account for the time-dependent material behaviour of the segments under successive load stages. The deflection at every stage

should be calculated and the needed camber should be determined and applied. The structural form changes from cantilever to continuous during the construction stage and service stage respectively. During the construction stage, the structure continues to deflect with time, so do the segments of the bridge which have been previously erected. When the cantilevers from the adjacent piers meet, there may be considerable difference in the level of the ends to be joined. This difference can be reduced by jacking or by providing a predetermined camber during the erection or casting of segments

3.2. DIFFERENT APPROACHES FOR PREDICTION OF LONG-TERM BEHAVIOUR

The prediction of long-term deformation is complicated due to factors such as gradual reduction of prestress force, effect of creep, influence of concrete cracking. Relatively simple procedures can be followed that permit calculation of deflection within acceptable limits of accuracy. The calculation of deflection is further simplified if the prestress force is accurately known, materials are stressed only within their elastic ranges and the concrete remains uncracked. For uncracked section, gross moment of inertia can be used in the calculation without serious error.

3.2.1. PCI Multipliers Method

PCI multipliers method provides a multiplier C_1 for immediate elastic deflection which takes into account the long-term effects in prestressed concrete members. To use these multipliers, which result in the total (immediate plus time-dependent) deflection components, the upward and downward components of the initial elastic camber should be separated to take into account the effects of loss of prestress which affects only the upward component. Fig. 3.1, which has been taken from the book titled "Prestressed Concrete: A Fundamental Approach" by Edward Nawy, gives the deflection multipliers. The term "erection" refers to the time the precast member is placed in the structure. The term "final" refers to the stage at which all loss and creep effects have occurred.

Table 7.1 C_1 Multipliers for Long-Term Camber and Deflection

	Without composite topping	With composite topping
<i>At erection:</i>		
(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
<i>Final:</i>		
(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	2.20
(5) Deflection (downward)—apply to the elastic deflection due to the superimposed dead load only	3.00	3.00
(6) Deflection (downward)—apply to the elastic deflection caused by the composite topping	—	2.30

Fig. 3.1. Multipliers for Long-Term Camber and Deflection

Shaikh and Branson proposed that substantial reduction can be achieved in long-term camber by the addition of non-prestressed steel. The reduced multiplier C_2 can be used which is given by

$$C_2 = \frac{C_1 + \frac{A_s}{A_{ps}}}{1 + \frac{A_s}{A_{ps}}} \quad \text{Eqn. (3.1)}$$

where,

C_1 = Multiplier from Fig. 3.1

A_s = Area of non-prestressed steel

A_{ps} = Area of prestressing steel

3.2.2. Approximate Time-Step Method

The approximate time-step method is based on the simplified form of summation of constituent deflections due to various time-dependent factors. Curvature due to prestress is calculated as:

$$\phi_{pi} = -\frac{P_i e_x}{E_c I_c} + (P_i - P_e) \frac{e_x}{E_c I_c} - \left(\frac{P_i + P_e}{2} \right) \frac{e_x}{E_c I_c} C_u \quad \text{Eqn. (3.2)}$$

where C_u is the creep coefficient. e_x is the eccentricity at a distance x from support. The first term is the initial negative curvature, the second term is the reduction in that curvature due to loss of prestress and the third term is the increase in negative curvature due to concrete creep. Here, the approximation is that creep occurs under a constant prestress force equal to the average of initial and final prestress forces.

The final deflection of the member due to prestress is given by

$$\delta = -\delta_{pe} - \frac{\delta_{pi} + \delta_{pe}}{2} C_u \quad \text{Eqn. (3.3)}$$

Adding the deflection due to self weight, δ_D and due to super-imposed dead load δ_{SD} , the final time-dependent deflection due to sustained load due to prestressing and sustained loads is given by:

$$\delta = -\delta_{pe} - \frac{\delta_{pi} + \delta_{pe}}{2} C_u + (\delta_D + \delta_{SD}) C_u \quad \text{Eqn. (3.4)}$$

The net deflection at full service loading is obtained by adding the live load deflection, δ_L

$$\delta = -\delta_{pe} - \frac{\delta_{pi} + \delta_{pe}}{2} C_u + (\delta_D + \delta_{SD}) C_u + \delta_L \quad \text{Eqn. (3.5)}$$

The intermediate deflection is obtained by using C_t in place of C_u in the above equations.

3.2.3. Deflection of Composite Girders

For the calculation of long-term deflection for composite girders, the change of moment of inertia of the cross section once the slab hardens have to be taken into account. Further, difference in shrinkage characteristics of precast girder and cast in situ slab increases the rigour of the calculation. The use of computer program facilitates the evaluation of camber and deflection in composite girders.

3.3. INCREMENTAL TIME-STEP METHOD

For better accuracy in the calculation of deflection than that obtained using approximate methods, incremental time-step method should be used. Creep does not proceed under a constant force but under a force that is reducing with time due to losses of prestress. This may be accounted for using a summation procedure based on incremental changes occurring in a series of discrete time steps. Loss of prestress can also be predicted with better accuracy using this

method. Appropriate material models should be used to incorporate the time-dependent effects of creep and shrinkage of concrete and relaxation of prestressing steel.

To obtain curvature at any section at time t, Eqn. (3.2) is replaced by the following equation:

$$\phi_{pi} = -\frac{P_i e_x}{E_c I_c} + \sum_0^i (P_{n-1} - P_n) \frac{e_x}{E_c I_c} - \sum_0^i (C_n - C_{n-1}) P_{n-1} \frac{e_x}{E_c I_c} \quad \text{Eqn. (3.6)}$$

where

P_1 = Initial prestress before losses

e_x = Eccentricity of tendon at any section along the span

Subscript (n-1) = Beginning of a particular time-step

Subscript (n) = End of the time-step

C_{n-1}, C_n = Creep coefficient at beginning and end, respectively, of a particular time-step

$P_n - P_{n-1}$ = Prestress loss in a particular time-step from all causes

A schematic of the changes in strains and rotations from the time-step (n-1) to (n) is shown in Fig. 3.2.

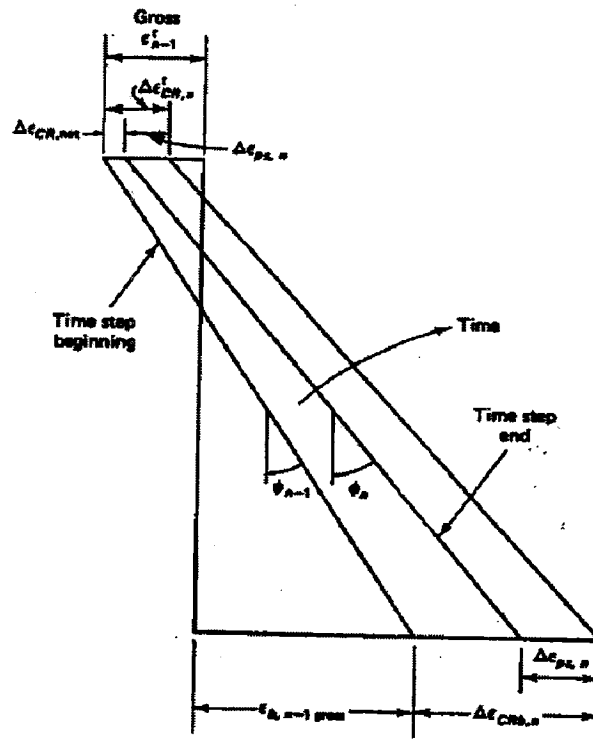


Fig. 3.2. Strain changes and rotations at step n

The steps of the method are as follows:

- i. Input section properties, material properties, load data, cable data, span data, material model data, times at which deformation is required, time steps for each time interval.
- ii. Calculate the net prestress in the cable after deducting the short-term loss. Choose sections (mid-span and support sections).
- iii. Choose material model for creep, shrinkage and relaxation model based on type of strands and calculate incremental creep strain and shrinkage strain and relaxation parameter.
- iv. Calculate the stress due to prestress force at the top and bottom fibers of the chosen sections. Calculate strains by dividing corresponding stresses by short-term modulus of elasticity of concrete.
- v. Calculate prestress loss for the time-interval and stresses and strains at the top and bottom fibers of the chosen sections.
- vi. Calculate the fiber gross strain due to incremental creep strain at the top and bottom fibers of the chosen sections.
- vii. Calculate the net strain by subtracting the strains due to prestress loss from fiber gross strains at the top and bottom fibers of the chosen sections. The calculation is done for both top and bottom fibers as follows:

$$\Delta \varepsilon_{CR,net} = \Delta \varepsilon_{CR,n} - \Delta \varepsilon_{ps,n} \quad \text{Eqn. (3.7)}$$

- viii. Calculate the incremental curvature at the chosen sections from the net strains and add it to the previous curvature to get total curvatures. Incremental curvature is given by

$$\Delta \phi_n = \frac{\Delta \varepsilon_{CR,net}^t - \Delta \varepsilon_{CR,net}^b}{h} \quad \text{Eqn. (3.8)}$$

where superscripts t and b denote top and bottom fibers respectively and h is the depth of the cross-section.

The total curvature is given by

$$\phi_T = \phi_{n-1} + \Delta \phi_n \quad \text{Eqn. (3.9)}$$

- ix. Calculate the deflection at mid-span due to prestress using the formula available for given tendon profile. Also calculate the new prestress at the chosen sections. For trapezoidal tendon profile, the deflection at mid-span due to prestress is given by:

$$\delta_{prestress} = \phi_c \frac{l^2}{8} + (\phi_s - \phi_c) \frac{a^2}{6} \quad \text{Eqn. (3.10)}$$

where,

l is the span.

a is the distance of cable hold-down point from the support.

ϕ_c is the curvature at mid-span due to prestress.

ϕ_s is the curvature at support due to prestress.

Note that when $a=l/2$, tendon profile becomes triangular.

- x. Calculate the deflection at mid-span due to self-weight by multiplying initial elastic deflection by $(1+\text{creep coefficient})$. Initial elastic deflection is calculated using the principles of structural analysis. For a simply-supported beam carrying uniformly distributed loading, formula is available for the calculation of elastic deflection at mid-span.

$$\delta_{self-weight} = \delta_{elastic}(1 + \varphi(t)) \quad \text{Eqn. (3.11)}$$

where,

$\varphi(t)$ is the creep coefficient at time t

3.4. ASSESSMENT OF LONG-TERM RESTRAINT MOMENT IN PRECAST CONCRETE BRIDGE GIRDERS MADE CONTINUOUS

In a precast, prestressed concrete girder made continuous using cast-in-place deck and diaphragm, the development of upward camber is restrained when the continuity diaphragm attains sufficient strength. This gives rise to positive (tension at bottom fiber) restraint moment in the continuity diaphragm which increases with time due to creep of concrete. Further, differential shrinkage occurs between precast girder and cast-in-situ deck slab. The restraint imposed on the deformation due to differential shrinkage gives rise to negative (tension at top fiber) restraint moment in the diaphragm which increases with time. The proper prediction of the restraint is one of the important aspect of the design of diaphragm at the intermediate support.

From the literature review in chapter 1, it is found that various researchers have given different methods for the assessment of the long-term restraint moment. The details and application of some of the methods are given in detail in chapter 5.

3.5. SUMMARY

In this chapter, different approaches for the prediction of long-term behaviour of bridge structures have been presented. Incremental time-step method for the calculation of long-term deflection has been explained in detail. Long-term restraint moment at the intermediate support

of precast, prestressed girders made continuous through cast-in-place deck slab and diaphragm has been introduced briefly.

LONG-TERM RESPONSE PREDICTION OF DETERMINATE BRIDGE STRUCTURES

4.1. GENERAL

Prestressed concrete bridges are being constructed widely nowadays. Eccentrically prestressed concrete girders undergo a net upward deflection or camber after the application of prestressing force. After the stresses at each stage have been checked, the designer has to further ensure that deflection remains within the permissible limits. Thus camber prediction is one of the important aspects of bridge design. Due to the time-dependent properties of concrete and prestressing steel, the deflection goes on increasing with time. The finished surface of the bridge deck should match with the specified elevation in the bridge plan. This will be possible only if camber at each stage of the bridge is predicted accurately. Various methods have been proposed by the researchers for predicting the camber. For modeling the time-dependent phenomena of concrete and steel, the mathematical models given by the researchers and available in the design codes need to be used.

4.2. SIGNIFICANCE OF LONG-TERM DEFORMATION IN PSC BRIDGES

The calculation of long-term deformation is required to check whether the structure is safe for the serviceability criteria. Long-term effects due to creep and shrinkage of concrete and relaxation of prestressing steel cause significant increase in deflection.

The literature review reveals that creep, shrinkage and relaxation affect the long-term deformation as well as prestress losses significantly. Prediction of these quantities depends on material models as well as the analysis approach to take care of non-linearity involved in the problem. In this context, in the present study, incremental time-step method is used to develop a program for prediction of long-term deformation for a simply-supported post-tensioned bridge girder. Two different material models, CEB-FIP90 and ACI 209R-92 are incorporated and numerical studies are carried out to develop quantitative information for the effect of change in material models.

4.3. ANALYSIS APPROACH

Incremental time-step method given in literature has been used to calculate the long-term prestress loss and deformation. The method has been programmed using MATLAB. The design life of the structure is divided into several time intervals. Each time interval is divided into a

number of small time steps. Interval of time-steps can be decided by user. These time steps should be smaller in the initial intervals and larger in the end time intervals. The strain distributions, curvatures and prestressing forces are calculated for each time together with the incremental creep, shrinkage and relaxation losses during the particular time step. The procedure is repeated for all subsequent time steps and an integration or summation of incremental steps is made to give the total time-dependent deflection at the particular section along the span. These calculations should be made for a sufficient number of points along the span.

For the present study, mid-span and support sections are chosen for the implementation of the above method. Short-term losses due to friction, anchorage slip and elastic shortening are taken as input parameter. Only single stage post-tensioning has been used. The deflections are calculated at the mid-span of the girder. Downward deflection is taken as positive. Hereafter, deflection means long-term deflection due to creep, shrinkage of concrete and relaxation of prestressing steel and loss means long-term prestress loss unless otherwise specified.

4.4. VALIDATION OF THE METHOD

The validation of incremental time-step method has been done with the example given in the book titled “Prestressed Concrete: A Fundamental Approach” by Edward Nawy. The details of the prestressed concrete girder are as follows:

Girder Details

Span	18.2 m
Area of cross-section	0.6310 m ²
Second moment of area of cross-section	0.0359 m ⁴
Distance of CG of cross-section from top	0.209 m
Distance of CG of cross-section from bottom	0.655 m

Cable Details

Cable profile	Triangular
Cable height from bottom at mid-span	0.0957 m
Cable height from bottom at support	0.331 m
Area of prestressing steel	1530 mm ²

Material Properties

Concrete density	23.58 kN/m ³
------------------	-------------------------

Characteristics strength of concrete	35 N/mm ²
Cement type	Normal hardening
Ultimate tensile strength of prestressing steel	1616.7 N/mm ²
Modulus of elasticity of prestressing steel	196 GPa
Type of strand	stress relieved

Loading Details

Initial prestress after short-term loss	1303 N/mm ²
Concrete age at the time of prestress loading	7 days

Material model

Creep and shrinkage model	ACI-209R-92
---------------------------	-------------

The above input data are fed into the computer program and prestress loss and mid-span deflection due to prestress are calculated at various times. The results are presented in Tables 4.1 and 4.2.

Table 4.1. Validation of Prestress Loss Vs Time

Time (days)	Prestress loss (N/mm ²) (A)	Prestress loss (N/mm ²) (Nawy) (B)	Percentage Error (B-A)/B*100
0	0	0	0
30	248.85	252.53	1.46
90	332.96	337.58	1.37
365	409.4	414.76	1.29
1825	457.15	462.93	1.25

Table 4.2. Validation of Deflection (prestress) Vs Time

Time (days)	Deflection (prestress) mm (A)	Deflection (prestress) mm (Nawy) (B)	Percentage Error (B-A)/B*100
0	-46.0	-48.0	4.17
30	-79.0	-82.0	3.66
90	-88.8	-93.0	4.52
365	-98.8	-104.0	5.00
1825	-105.4	-110.0	4.18

From Table 4.1 and 4.2, it can be observed that percentage error in prestress loss and deflection due to prestress are within 2 percent and 5 percent respectively. The error can be due to the round-off involved in the conversion of units (from FPS to SI). Thus the program is validated.

4.5. EFFECT OF NUMBER OF STEPS

The effect of the number of steps on the results is presented here. For studying this effect, prestress loss and deflection due to prestress have been calculated at 30th day after prestressing by varying the number of time steps between 0 and 30 days. The duration of 30 days has been chosen based on the fact that variation of creep and shrinkage is more in the initial stage of the design life. The details of prestressed concrete girder are the same as those used for validation of results. CEB-FIB90 model is used for creep and shrinkage effects. The results are presented for prestress loss and deflection due to prestress after 30 days v/s the number of steps given in Tables 4.3 and 4.4.

Table 4.3. Percentage Loss After 30 Days Vs Number of Steps

No. of time steps	%age prestress loss after 30 days	% change from the result of single time-step
1	12.78	0
2	12.35	-3.36
3	12.14	-5.01
4	12.00	-6.10
5	11.91	-6.81
10	11.66	-8.76
20	11.45	-10.41
30	11.35	-11.19

Table 4.4. Deflection (Prestress) After 30 Days Vs Number of Steps

No. of time steps	Deflection (prestress) mm	% change from the result of single time-step
1	-69.8	0
2	-69.2	-0.86

3	-68.9	-1.29
4	-68.8	-1.43
5	-68.7	-1.58
10	-68.5	-1.86
20	-68.3	-2.15
30	-68.2	-2.29

In Tables 4.3 and 4.4, percentage prestress loss and deflection due to prestress after 30 days are calculated by varying the number of steps. Further, these values are compared with the values obtained using single time-step and percentage change is evaluated which are presented in column 3 of Tables 3 and 4. It is observed that as the number of steps increases, percentage change in prestress loss and deflection due to prestress increases numerically but the difference in successive percentage changes goes on reducing. Thus, the number of steps can be chosen based on the desired level of accuracy.

4.6. PARAMETRIC STUDIES

The objective of the parametric studies is to evaluate the effect of the variation of relative humidity and temperature on the prestress loss and deformation as well as effect of change in material model for prediction of creep and shrinkage. For this purpose, a typical post-tensioned bridge girder is used for which data are presented here.

Girder Details

Span		27.85 m
Area of cross-section	Mid-span	0.8425 m ²
	Support	1.37 m ²
Second moment of area of cross-section	Mid-span	0.3234 m ⁴
	Support	0.405 m ⁴
Distance of CG of cross-section from top	Mid-span	0.89 m
	Support	0.85 m
Distance of CG of cross-section from bottom	Mid-span	0.91 m
	Support	0.95 m
Perimeter of the cross-section	Mid-span	3.6 m

exposed to atmosphere	Support	3.6 m
-----------------------	---------	-------

Cable Details

Cable profile	Trapezoidal
Cable height from bottom at mid-span	0.18 m
Cable height from bottom at support	0.775 m
Cable hold-down point	10.444 m
Area of prestressing steel	3750.6 mm ²

Material Properties

Concrete density	24 kN/m ³
Characteristics strength of concrete	45 N/mm ²
Cement type	Normal hardening
Ultimate tensile strength of prestressing steel	1860 N/mm ²
Modulus of elasticity of prestressing steel	195 GPa
Type of strand	Low relaxation

Loading Details

Initial prestress applied	1399.5 N/mm ²
Concrete age at the time of prestress loading	7 days

Material model

Creep and shrinkage model	CEB-FIP90 and ACI-209R-92
---------------------------	---------------------------

The above input data is used in the computer program and results are tabulated and plotted by varying different parameters. Short-term losses due to friction, anchorage slip and elastic shortening are taken as input parameter. Downward deflection is taken as positive.

4.6.1. Effect of Material Models

The effect of material models on the long-term prestress loss and deformation are presented here. Comparisons are presented for two material models, i.e. CEB-FIP90 and ACI 209R-92. Prestress losses and deflections are calculated at 0, 21, 38, 90, 365, 500, 1000, 3000, 5000, 7000, 9000 and 10000 after the application of prestressing force.

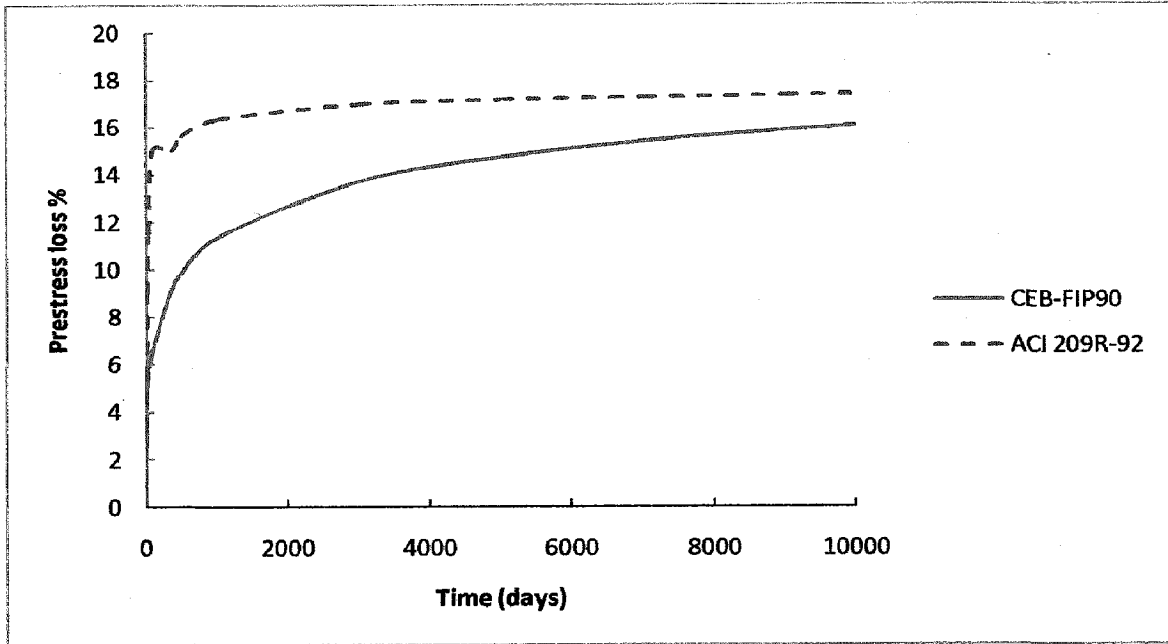


Fig. 4.1. Prestress Loss Vs Time

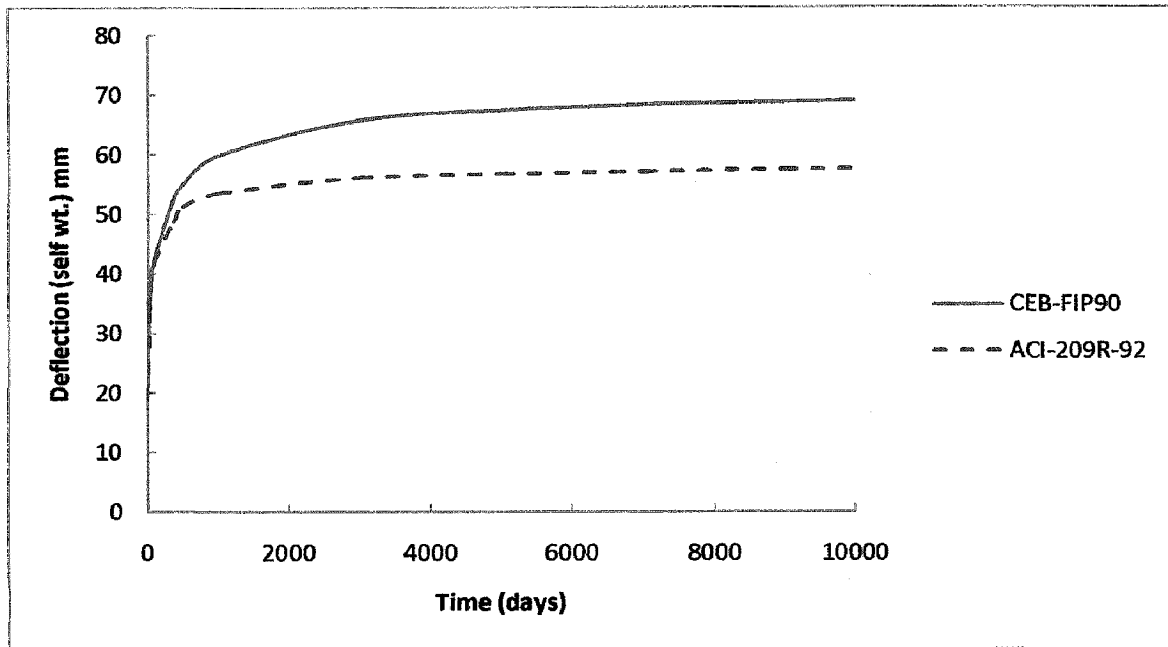


Fig. 4.2. Deflection (Self Weight) Vs Time

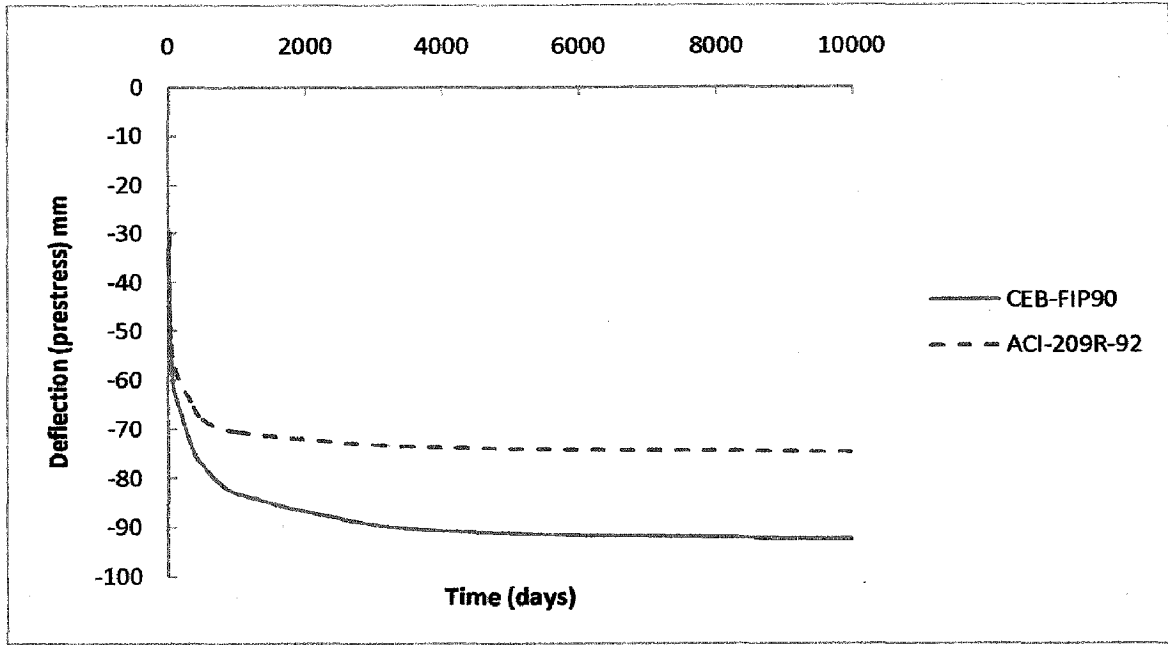


Fig. 4.3. Deflection (Prestress) Vs Time

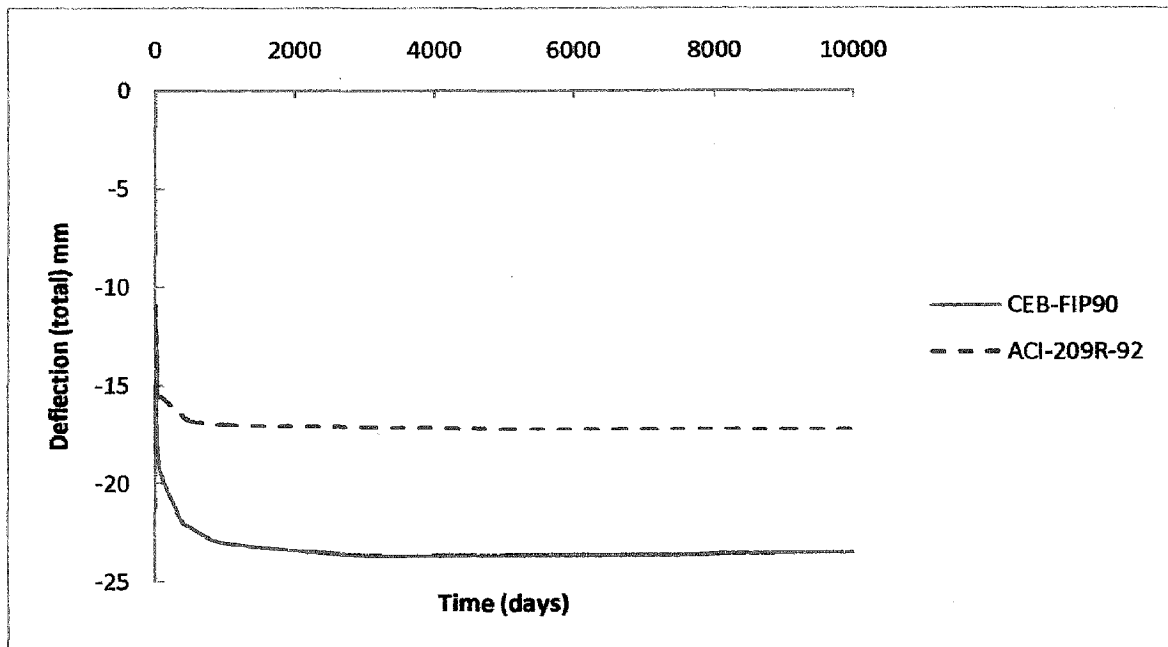


Fig. 4.4. Deflection (Total) Vs Time

From Fig. 4.1 to Fig. 4.4, it can be observed that CEB-FIP90 model predicts less prestress loss and more deflection compared to the ACI-209R-92 model.

4.6.2. Effect of Environment

The effect of environment on the long-term deflection and prestress loss has been studied. Temperature remains constant and relative humidity is varied and vice-versa. For a particular case, relative humidity or temperature is assumed to remain constant during the entire construction and service life or average relative humidity and temperature can be taken. This is done for the sake of simplicity. The effect of temperature has been incorporated using CEB-FIP90 model only. Prestress loss and deflections are calculated at 10000 days from the application of prestress.

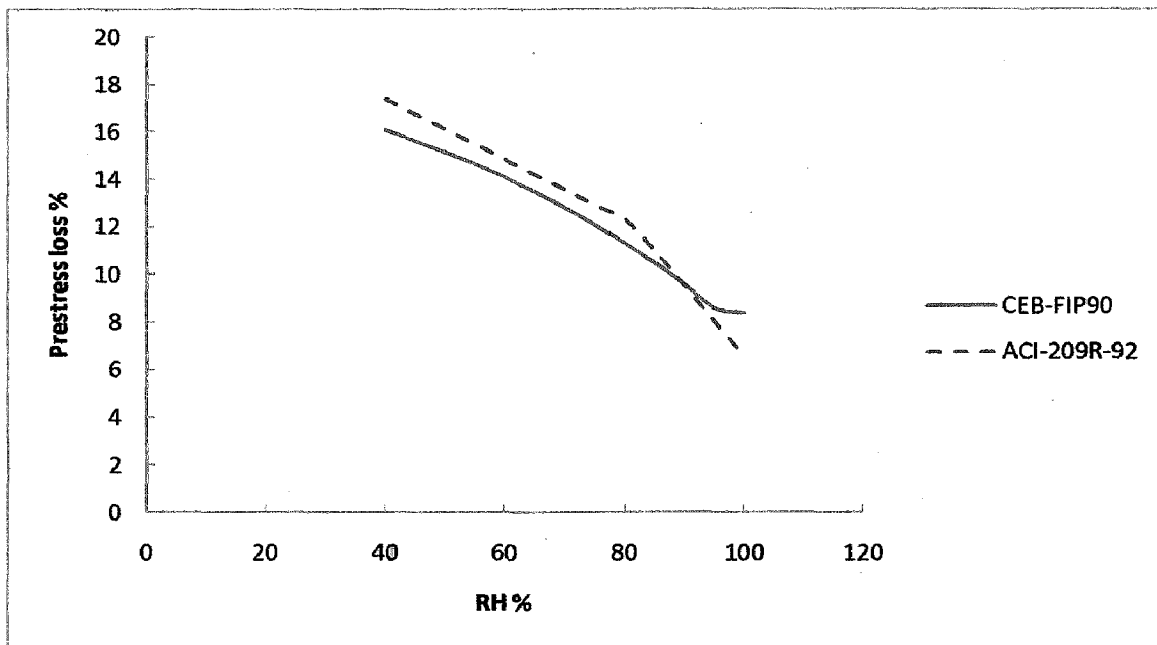


Fig. 4.5. Prestress Loss Vs Relative Humidity

From Fig.4.5., it can be observed that prestress loss decreases with the increase in relative humidity for both the models but the difference between the two models is more at low humidity while at high humidity, both the models give almost same prestress loss.

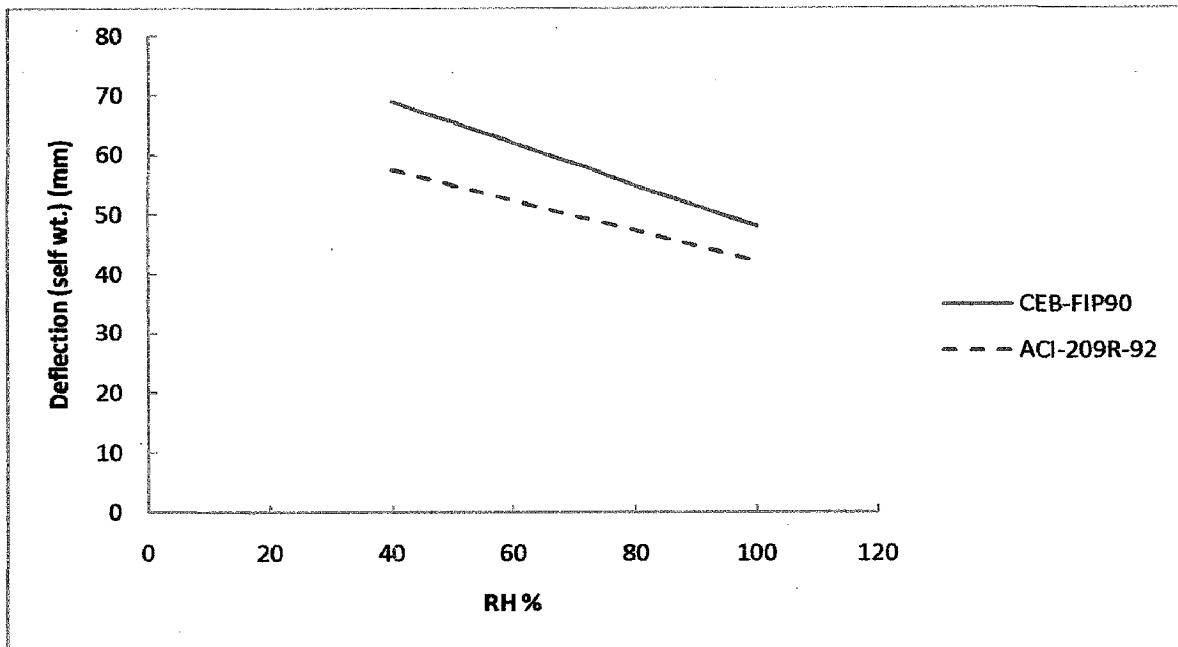


Fig. 4.6. Deflection (Self Weight) Vs Relative Humidity

From Fig.4.6., it can be observed that the deflection due to self weight decreases with the increase in relative humidity for both the models. The rate of change is more in CEB-FIP90 model.

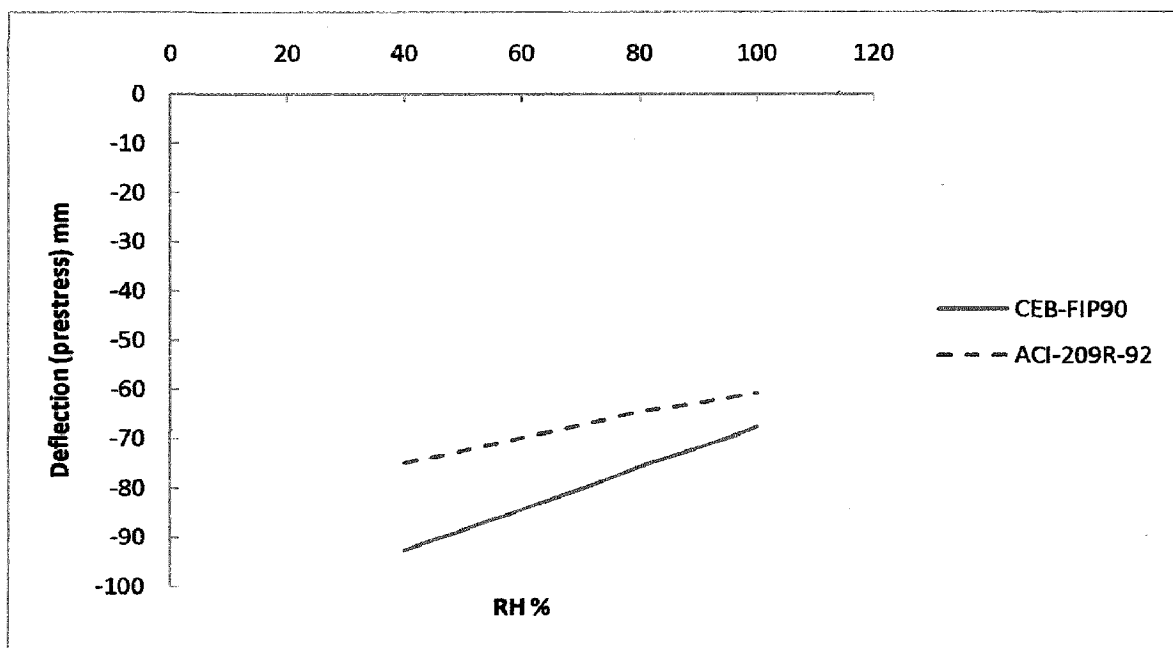


Fig. 4.7. Deflection (Prestress) Vs Relative Humidity

From Fig.4.7., it can be observed that the deflection due to prestress decreases with the increase in relative humidity for both the models. The rate of change is more in CEB-FIP90 model.

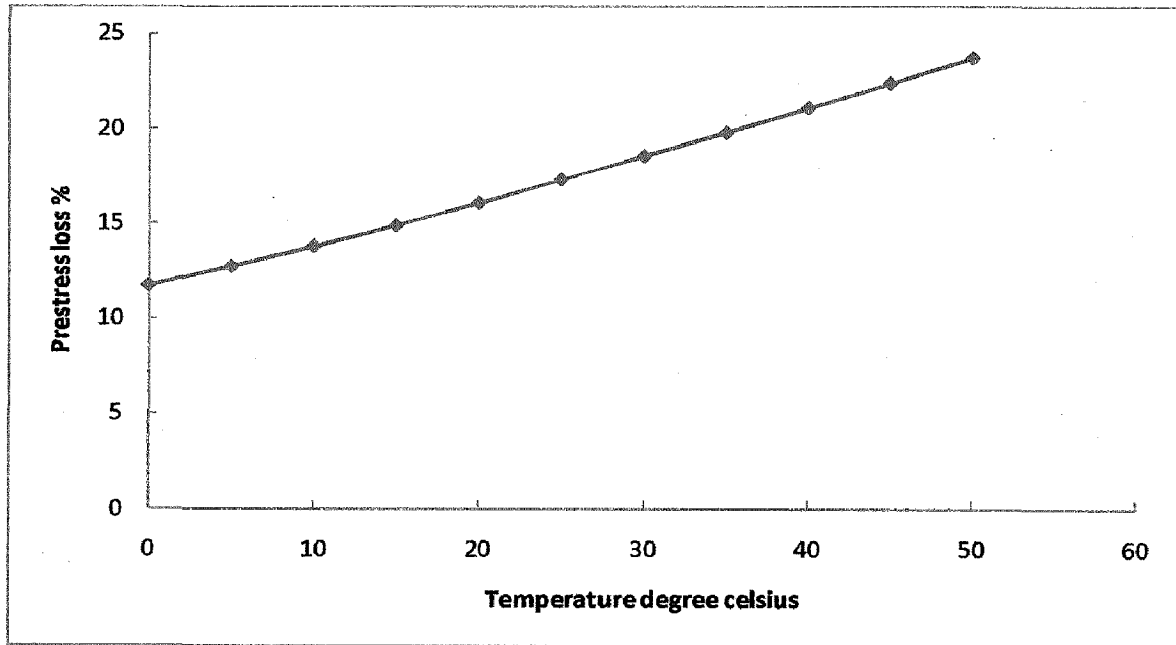


Fig. 4.8. Prestress Loss Percent Vs Temperature

From Fig.4.8., it can be observed that prestress loss increases as the temperature increases.

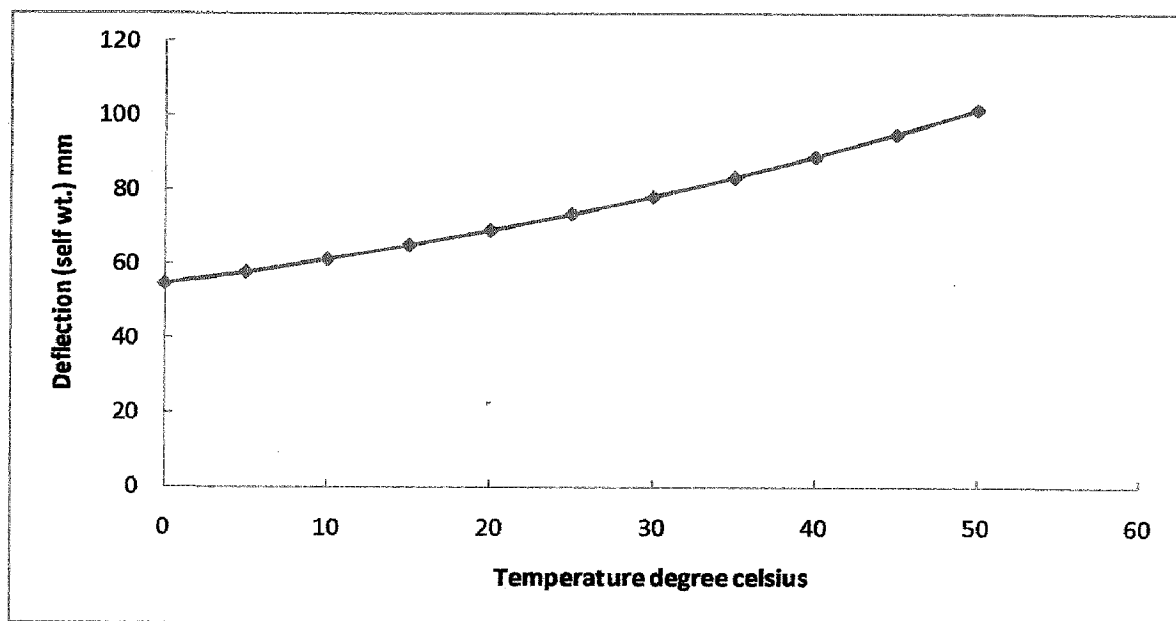


Fig. 4.9. Deflection (Self Weight) Vs Temperature

From Fig.4.9., it can be observed that deflection due to self weight increases as the temperature increases.

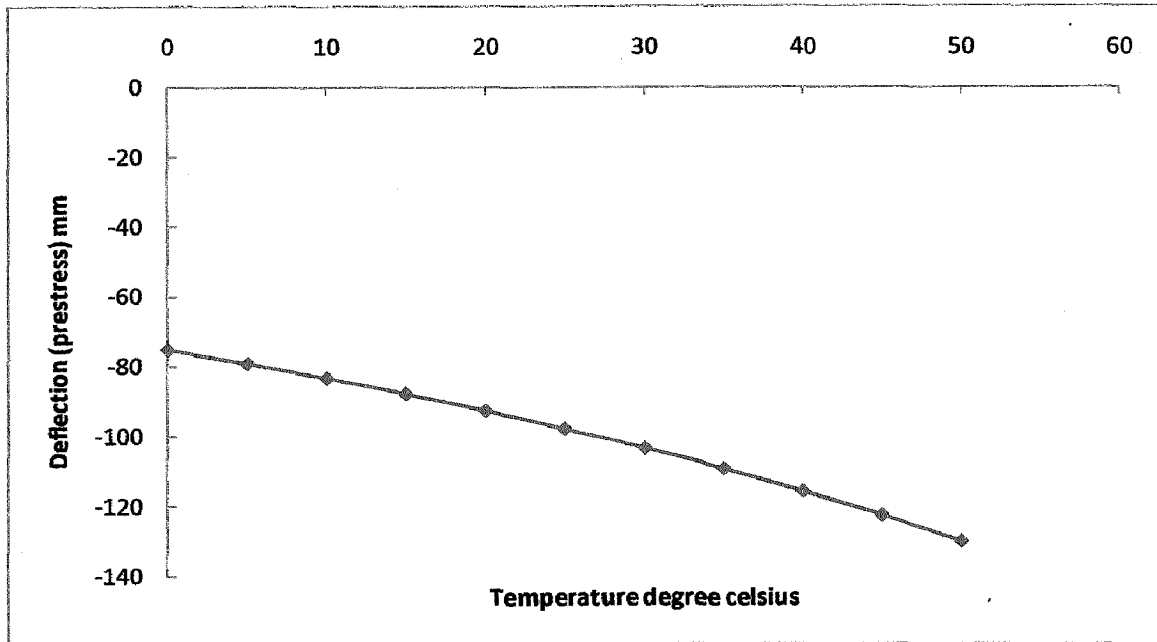


Fig. 4.10. Deflection (Prestress) Vs Temperature

From Fig.4.10., it can be observed that deflection due to prestress increases as the temperature increases.

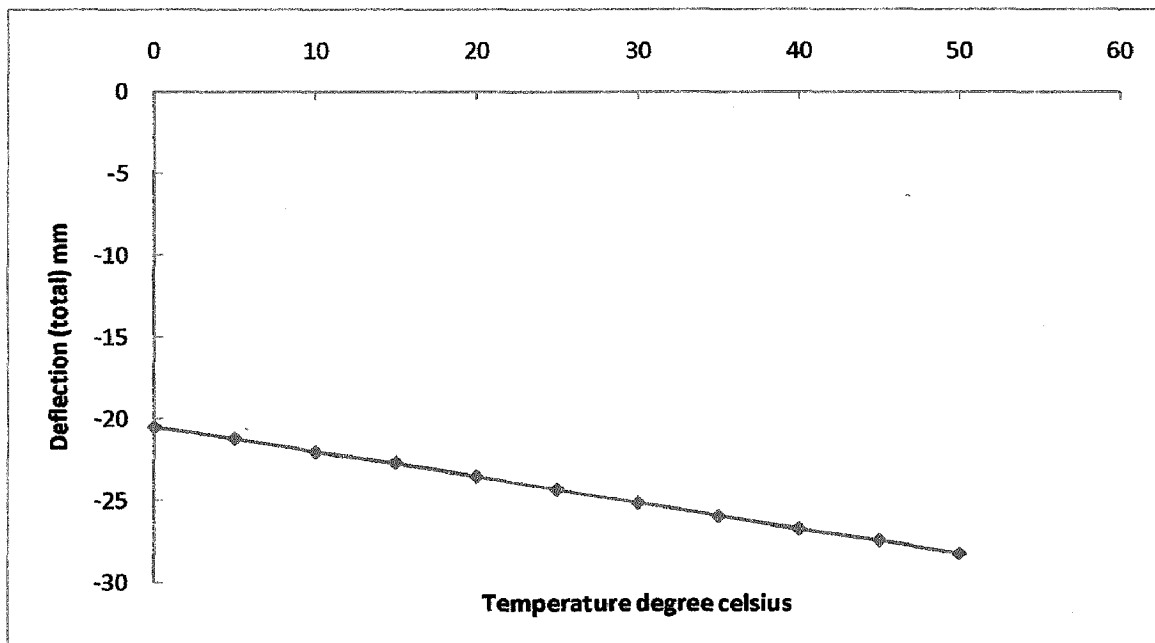


Fig. 4.11. Deflection (Total) Vs Temperature

From Fig.4.11., it can be observed that total deflection increases as the temperature increases.

4.7. CONCLUDING REMARKS

Creep, shrinkage and relaxation affect the long-term deformation as well as prestress losses significantly. Prediction of these quantities depends on material models and the analysis approach. In the present study, a program based on incremental time-step method is developed and numerical studies are carried out, using two different material models namely CEB-FIP90 and ACI 209R-92, to predict long-term prestress loss and deformation for a simply-supported post-tensioned bridge girder.

LONG-TERM RESPONSE PREDICTION OF INDETERMINATE BRIDGE STRUCTURES: PRECAST PRESTRESSED BRIDGE GIRDERS MADE CONTINUOUS

5.1. GENERAL

In a precast, prestressed concrete bridge made continuous, two or more simply-supported spans are made continuous by using the cast-in-place deck and continuity diaphragm. One of the early methods of making a bridge continuous over two or more spans was to place the ends of the girders close to each other and post-tension them together. However, this method was not efficient because the anchorages and tensioning were relatively expensive and there was considerable friction loss due to the severe curves necessary to make the post-tensioning effective (Mattock, et al. 1960). Because of these disadvantages, an alternative method began to develop. The improved method called for leaving a small space between the ends of the girders and extending positive moment reinforcing steel, instead of post-tensioning strands, into that region from the beams. Concrete would be added to this section at the time when the deck was poured to provide continuity over the joint. This area is known as a continuity diaphragm. In the present chapter, time-dependent restraint moment in the continuity diaphragm has been investigated.

5.2. ADVANTAGES OF CONTINUOUS BRIDGE

Continuous bridges have several advantages compared to simply-supported bridges. Bending moments and deflections are less which result in reduction in structural depth of girder and in the number of prestressing strands. Elimination of expansion joints of the deck and parapet joints at the continuity supports leads to better riding surface of the bridge as well as savings both in the construction and material cost. In addition of this, the durability of the bridge will be increased due to the removal of joints because water and salts from the deck will not drain onto the substructure. The number of bearings required can be nearly halved. Lesser number of bearings would result in savings in cost. The dimensions of the piers and cross-heads can be reduced due to only one bearing at each support. Further, in a continuous bridge, redistribution of moments will occur if the load capacity increases in a particular girder of the bridge system.

5.3. CONCEPT OF RESTRAINT MOMENT

In case of a simply-supported girder, the deflections and rotations occur freely due to the time-dependent effects of creep, shrinkage and relaxation. When these simply-supported girders are made continuous, the deformations are restrained. This restraint imposed on deformation gives rise to the restraint moment. In Fig. 5.1, the development of restraint moment is shown.

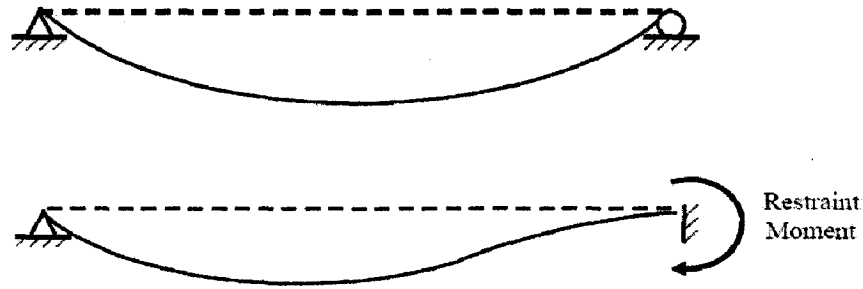


Fig. 5.1. Development of Restraint Moment

Fig. 5.2 and Fig. 5.3 shows the construction sequence of a typical two-span bridge with precast/prestressed girders made continuous and the deformations and restraint moments caused by creep respectively.

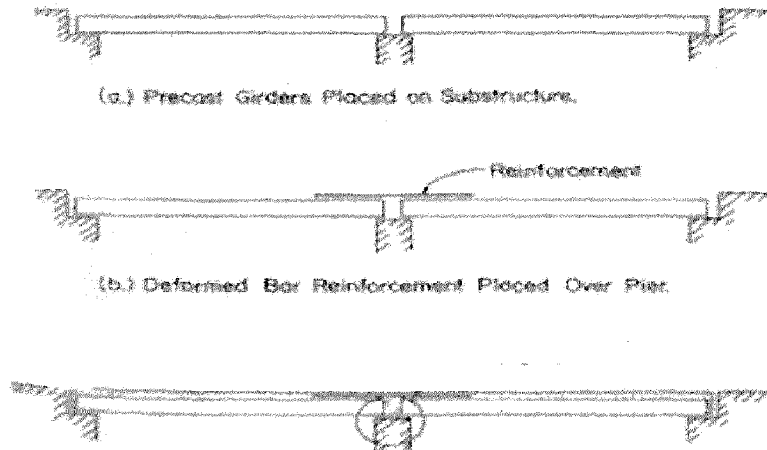


Fig. 5.2. Construction Sequence of a Two-Span Bridge (Freyermuth, 1969)

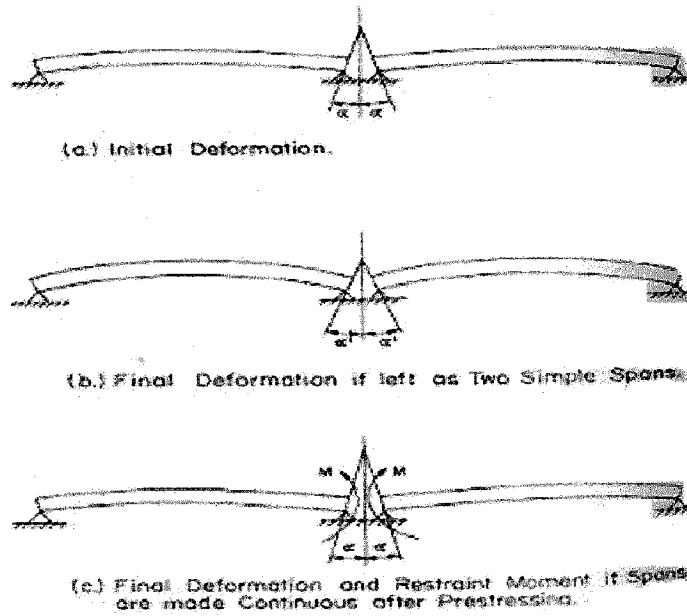


Fig. 5.3. Deformations and Restraint Moments in a Two-Span Continuous Beam (Freyermuth, 1969)

5.4. BRIEF DESCRIPTION OF THE METHODS FOR THE CALCULATION OF RESTRAINT MOMENT

From the literature review on precast, prestressed girders made continuous, it is found that various researchers have given different methods for the calculation of restraint moment at intermediate support. Here two of the methods are presented which are used in the calculation of restraint moment.

5.4.1. PCA Method

PCA method stands for the Portland Cement Association Method which was explained by Freyermuth in the bulletin "Design of Continuous Highway Bridges with Precast Prestressed Girders" which was published in 1969. The findings of Mattock (1961) were also included in this method. In the 1950's the Portland Cement Association (PCA) undertook several projects that focused on composite construction so that an analysis method could be developed (Hongstad, et al. 1960). The findings of a well-known researcher, Mattock, were also included in the development of the PCA Method which states, "the effects of creep under prestress and dead load can be evaluated by an elastic analysis assuming that the girder and slab were cast and prestressed as a monolithic continuous girder" (Mattock, et al. 1961).

The restraint moment due to the effect of creep due to dead load and prestressing force and differential shrinkage is given by:

$$M_r = (M_{ps} - M_{DL})(1 - e^{-\phi}) - M_s \left(\frac{1 - e^{-\phi}}{\phi} \right) \quad \text{Eqn. (5.1)}$$

where,

M_{ps} = Moment at the intermediate support due to prestressing force

M_{DL} = Moment at the intermediate support due to dead load

M_s = Moment at the intermediate support due to differential shrinkage.

ϕ = Creep coefficient

The moment due to differential shrinkage is calculated as:

$$M_s = \epsilon_s E_b A_b d \quad \text{Eqn. (5.2)}$$

where,

ϵ_s = Differential shrinkage strain

E_b = Modulus of elasticity of girder

A_b = Area of cross-section of composite section

d = Distance of mid-depth of slab and centroid of composite section

5.4.2. Trost's Approach

Rajagopalan N in his book titled "Bridge Superstructure" has given a method to calculate the effect of creep of concrete on due to restraints caused by continuity on the deformations due to dead load and prestressing. The method is based on Trost's approach explained as follows:

Effect of Creep of Concrete Due to Restraints Caused by Continuity on Dead Load Deformation:

The value of moment caused by the self-weight of girder and deck slab on the structural system as a continuous unit is evaluated. The moment caused by dead load on the simply-supported girder is evaluated. The difference in these moments will be subjected to creep effect. For arriving at the value, these changes in moment will be multiplied by the creep coefficient, ϕ . Hence, creep restraining moment is given by $k \times \phi \times$ (moment on the structure considering it in one-go as continuous minus moment on the structure considering it as simply-supported). The value 'k' is the creep relaxation coefficient since the creep coefficient itself varies with time because of variation in the applied load. 'k' is given by $1/(1+\rho\phi_\infty)$, where ρ is the creep relaxation factor and ϕ_∞ is the creep coefficient at infinite time.

Effect of Creep of Concrete Due to Restraints Caused by Continuity on Prestress Deformation:

The deformation due to prestress will also be restrained due the restraining effect of continuity which will give rise to restraint moment. Similar procedure as done in the case of dead load can be adopted for prestressing also. The dead load should be replaced by prestress force.

5.5. ANALYSIS APPROACH

The methods given in previous section are used for the calculation of restraint moment due to creep at the intermediate support of a two-span bridge girder made continuous using cast-in-place diaphragm only. The restraint moment due to differential shrinkage is not considered as the deck slab is not casted. The restraint moment due to dead load and prestress at intermediate support at the time of establishment of continuity can be obtained using methods of structural analysis like three-moment equation, stiffness approach, etc. Sagging moments are taken as positive while hogging moments are taken as negative. The steps for the calculation of time-dependent restraint moment at intermediate support is are explained as follows:

i. Calculation of Moment Due to Dead Load and Prestress at Intermediate Support.

The moment due to dead load at intermediate support of a two-span bridge is given by:

$$M_{DL} = -\frac{wl^2}{8} \quad \text{Eqn. (5.3)}$$

where,

w = Uniformly distributed dead load

l = Span length

Negative sign is used for hogging moment.

The moment due to prestress at intermediate support of a two-span bridge is given by:

$$M_{PS} = \frac{3P}{2l}((e_s - e_c)a + e_c l) \quad \text{Eqn. (5.4)}$$

where,

P = Prestress force (obtained after deducting short-term losses from initial prestress force)

l = Span length

e_s = eccentricity of prestress force at the support (eccentricity below the CG of cross-section is taken as positive)

e_c = eccentricity of prestress force at the mid-span (eccentricity below the CG of cross-section is taken as positive)

a = Distance of cable hold-down point from support

The total moment, M at intermediate support of a two-span bridge is given by the sum of the moments due to dead load and prestress. Summation is done with proper sign.

$$M = M_{DL} + M_{PS} \quad \text{Eqn. (5.5)}$$

ii. Calculation of Time-Dependent Restraint Moment.

The time-dependent restraint moment due to creep effect is obtained by applying the methods explained in the previous section. The calculations are explained here:

Restraint Moment by PCA Method

$$M_r = M * (1 - e^{-(\varphi(t) - \varphi(t_c))}) \quad \text{Eqn. (5.6)}$$

where,

M_r = Time-dependent restraint moment

M = restraint moment due to dead load and prestress at the time of establishment of continuity

φ = Creep coefficient

t_c = Time of establishment of continuity

Restraint Moment by Trost's Approach

$$M_r = M * k * (\varphi(t) - \varphi(t_c)) \quad \text{Eqn. (5.7)}$$

where,

M_r = Time-dependent restraint moment

M = restraint moment due to dead load and prestress at the time of establishment of continuity

φ = Creep coefficient

k = Creep relaxation coefficient $1/(1 + \rho(\varphi_\infty - \varphi_{t_c}))$, where ρ is the creep relaxation factor which is taken as 0.8, φ_∞ is the creep coefficient at infinite time and φ_{t_c} is the creep coefficient at the time of establishment of continuity, t_c .

5.6. NUMERICAL STUDIES

Numerical studies are presented here to evaluate the effects of various parameters like age of precast girder at the time of establishment of continuity, material models, environmental parameters, analysis methodology on the long-term continuity moment at the support. For carrying out the studies, two-span continuous bridge girder is considered. The precast girders are made continuous by casting continuity diaphragms. The details of the girder are same as that used in the parametric studies of simply-supported post-tensioned bridge in the previous chapter 4.

5.6.1. Effect of Analysis Approach

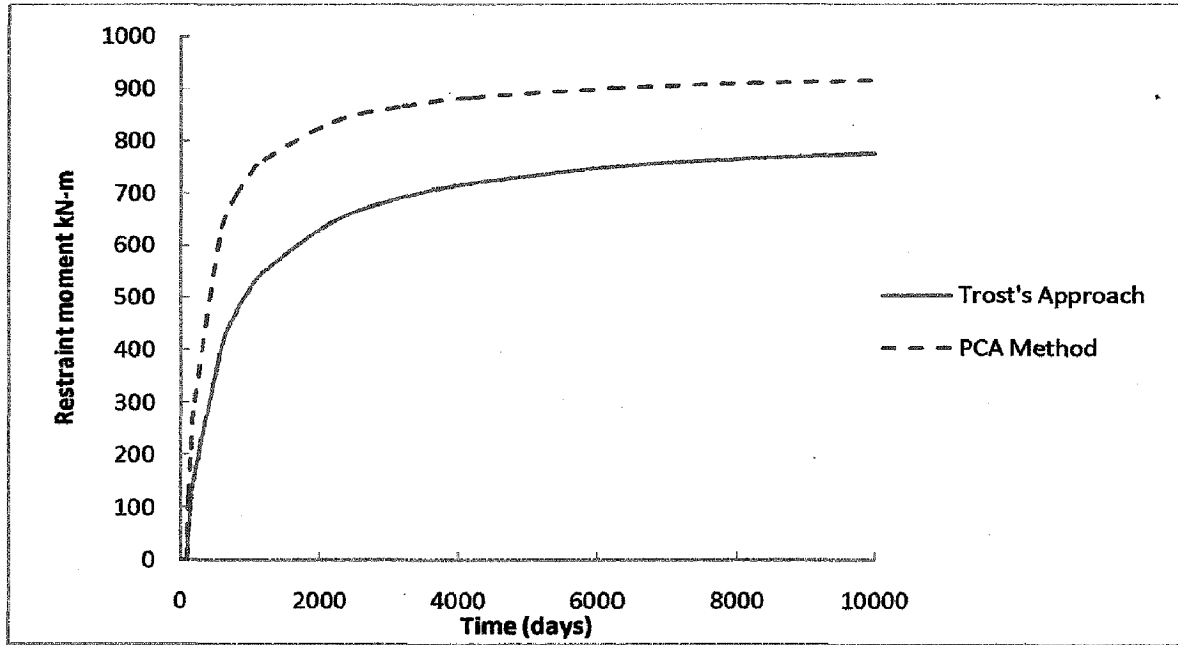


Fig. 5.4. Restraint Moment Vs Time (CEB-FIP90 Model)

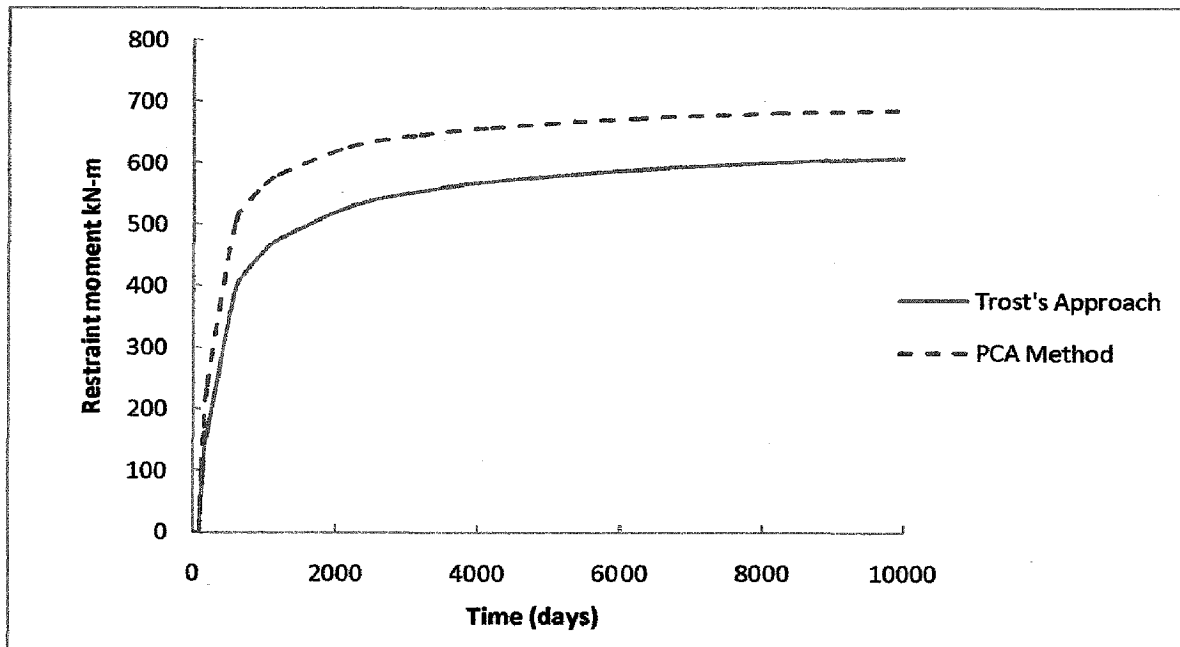


Fig. 5.5. Restraint Moment Vs Time (ACI 209R-92 Model)

From Fig. 5.4 and Fig. 5.5, it is observed that restraint moment increases with time for both PCA method and Trost's approach. The rate of increase is more in the beginning and less in the end. PCA method predicts more restraint moment using both the material models.

5.6.2. Effect of Material Model

The effect of material model on long-term continuity moment is presented here. Two different material models are considered namely CEB-FIP90 and ACI 209R-92. The results are presented for both analysis methods, i.e., Trost's method and PCA method.

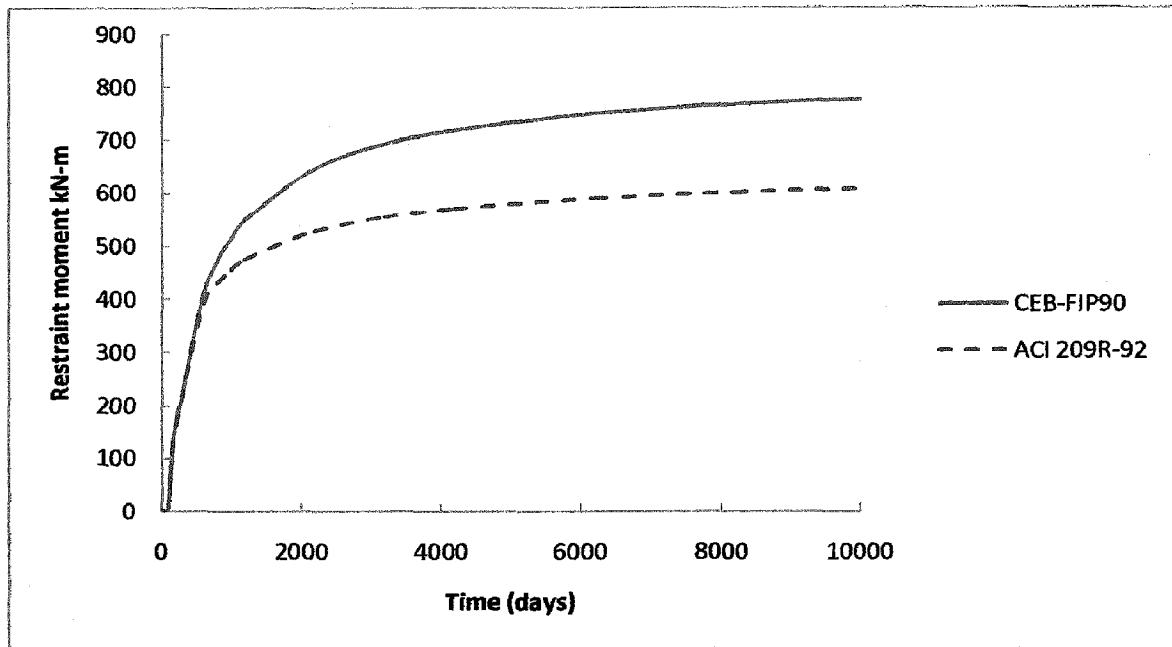


Fig. 5.6. Restraint Moment Vs Time (Trost's Approach)

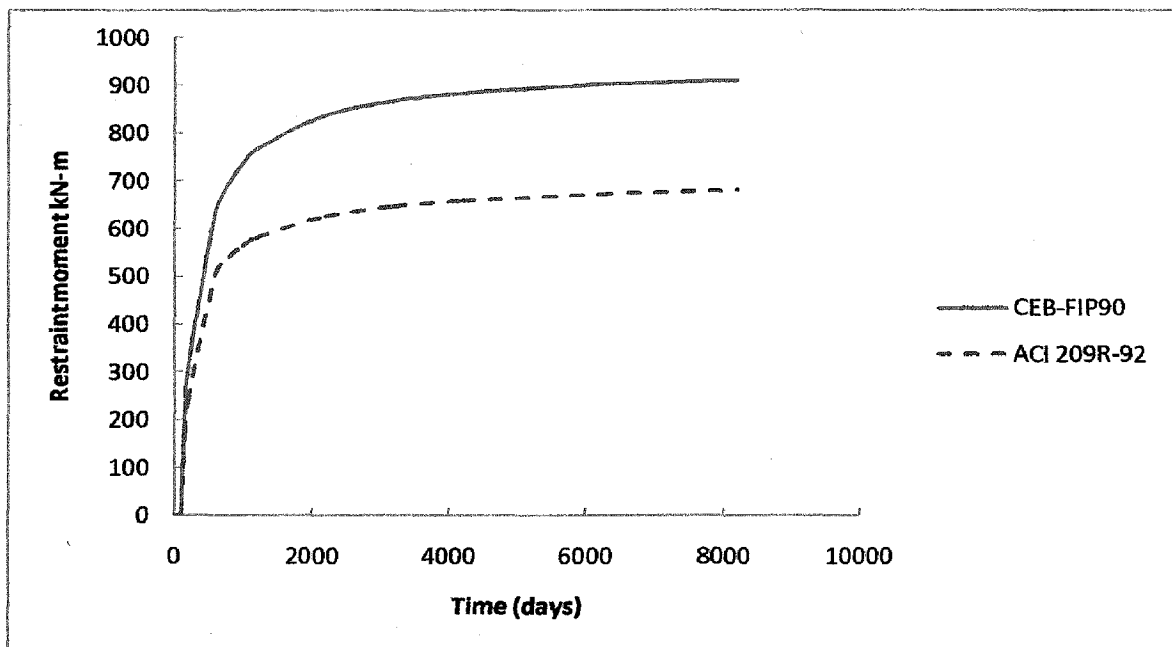


Fig. 5.7. Restraint Moment Vs Time (PCA Method)

From Fig. 5.6 and Fig. 5.7, it is observed that restraint moment increases with time for both CEB-FIP90 and ACI 209R-92. The rate of increase is more in the beginning and less in the end. The restraint moments predicted by using CEB-FIP90 model is found to be more than those predicted by ACI model.

5.6.3. Effect of Environment

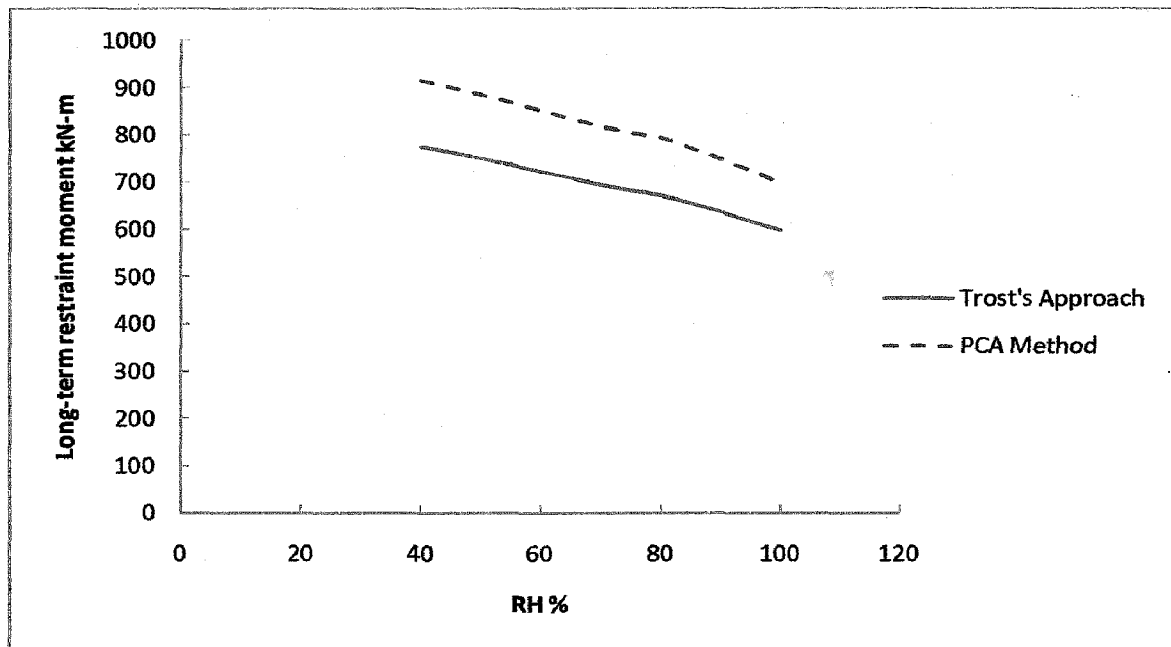


Fig. 5.8. Long-Term Restraint Moment Vs Relative Humidity (CEB-FIP90 Model)

From Fig. 5.8, it is observed that long-term continuity decreases with an increase in relative humidity for both the methods. For Trost's approach, it decreases by 23 % while for PCA method it decreases by 23.7 % as the relative humidity is increased from 40 % to 100 %.

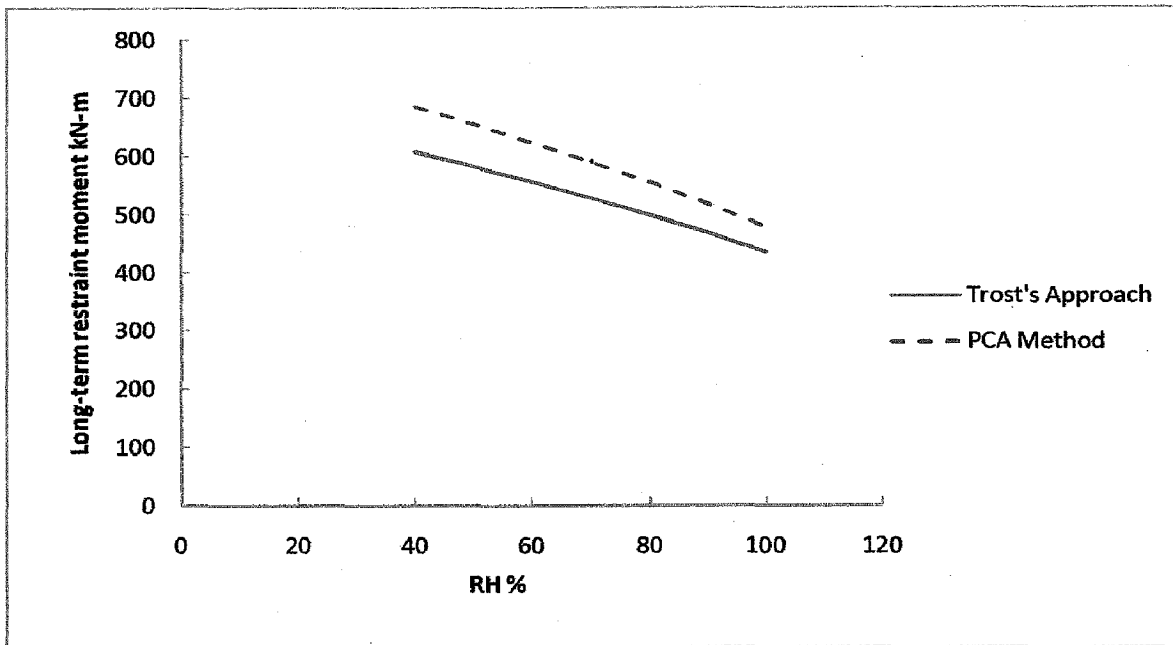


Fig. 5.9. Long-Term Restraint Moment Vs Relative Humidity (ACI 209R-92 Model)

From Fig. 5.9, it is observed that long-term continuity decreases with an increase in relative humidity for both the methods. For Trost's approach, it decreases by 28.7 % while for PCA method it decreases by 30.4 % as the relative humidity is increased from 40 % to 100 %.

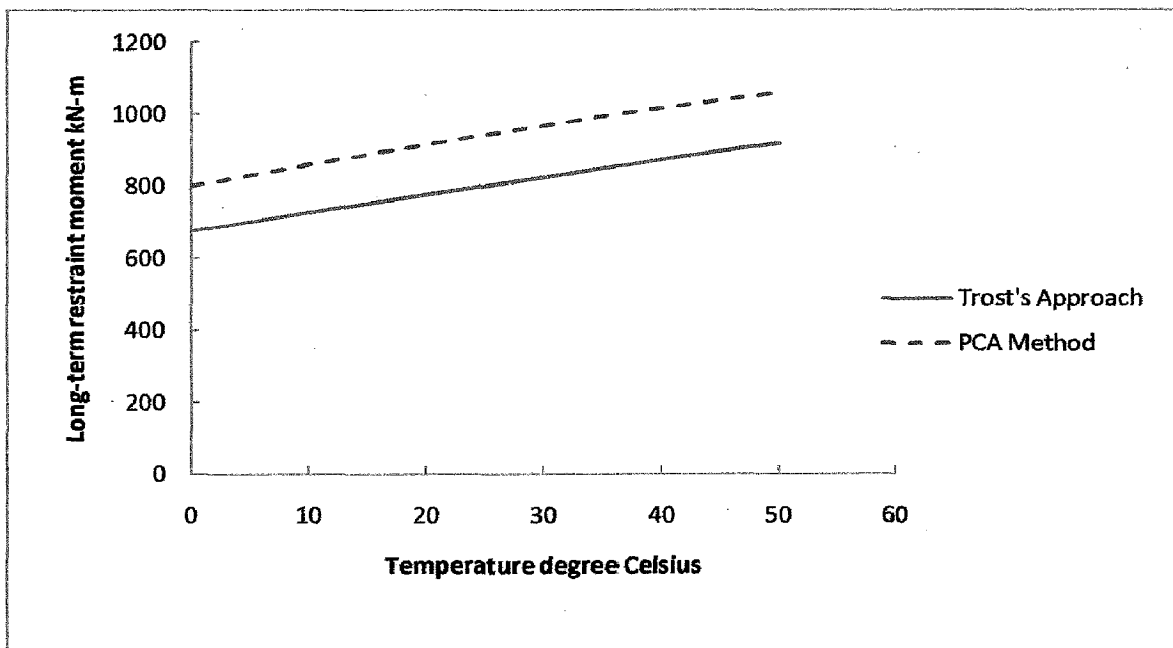


Fig. 5.10. Long-Term Restraint Moment Vs Temperature (CEB-FIP90 Model)

From Fig. 5.10, it is observed that, long-term restraint moment increases with the increase in the temperature. As the temperature increases from 0°C to 50°C, long-term restraint moment at 10000 days increases by 36 % for Trost's approach and 32.2 % for PCA method.

5.6.4. Effect of Age of Girder at Continuity

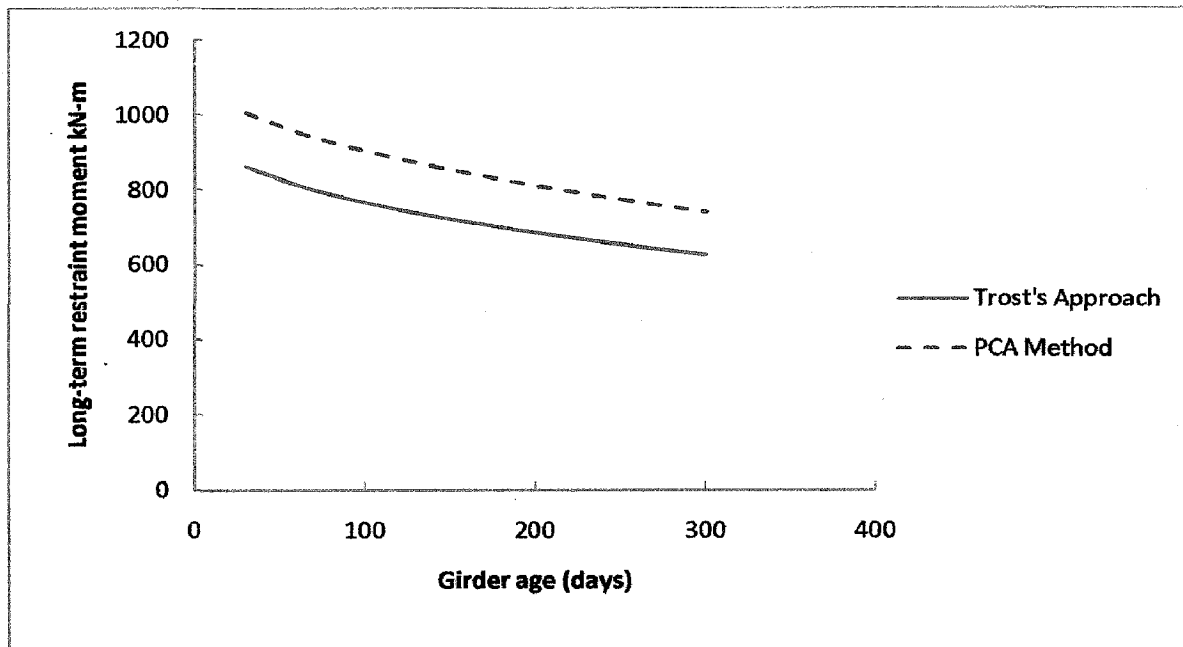


Fig. 5.11. Long-Term Restraint Moment Vs Girder Age at Continuity

In Fig. 5.11, the effect of age of girder at the time of establishment of continuity has been evaluated. It can be observed that long-term restraint moment decreases with an increase in the age of girder at continuity. An increase of girder age at continuity from 30 days to 300 days results in a decrease of 27.5 % for Trost's approach and 26.4 % for PCA method.

5.7. CONNECTION DETAILS FOR POSITIVE RESTRAINT MOMENT

From the previous sections, it becomes clear that positive restraint moment develops at the intermediate support due to the time-dependent effects of creep of concrete. The bridge designer has to design suitable reinforcement and connection details to resist this positive restraint moment. In the 1960's Portland Cement Association (PCA) developed several connection details to resist these positive moments. One of these details is shown in Fig. 5.12.

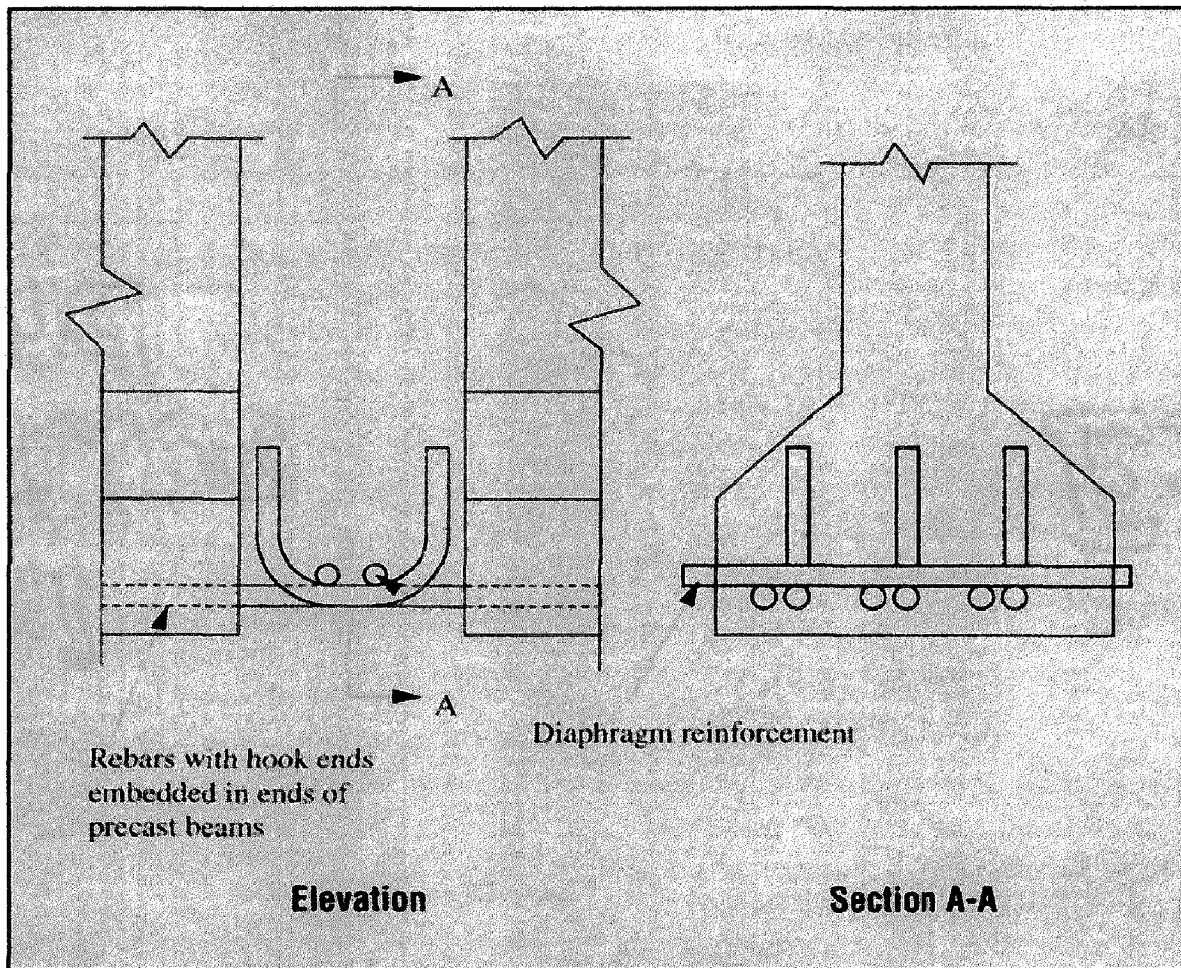


Fig. 5.12. Positive Restraint Moment Connection Details by Embedded Reinforcing Bars (Ma et al., 1998)

To avoid additional reinforcing bars, bottom prestressing strands of the precast, prestressed girder can be extended into the continuity diaphragm. This detail not only reduces the time-dependent restraint moment cracking but also improves shear resistance.

5.8. SUMMARY

Restraint moment at intermediate support of a two-span precast/prestressed bridge girder made continuous using cast-in-place diaphragm has been evaluated using a computer program. Numerical studies have been done to show the effect of various parameters like girder age at the time of establishment of continuity, environmental parameters etc. Long-term deformation has not been predicted as they are restrained due the introduction of indeterminacy. The connection details of the continuity diaphragm have also been provided.

LONG-TERM RESPONSE PREDICTION OF INDETERMINATE BRIDGE STRUCTURES: SEGMENTAL BRIDGES

6.1. GENERAL

In segmental bridges, construction is done by assembling the bridge segments in appropriate position and post-tensioning them together. One of the common methods of construction of bridges is the cantilever construction which involves addition of segments from both sides of the pier and post-tensioning them together. Using segmental cantilever construction, the economy of continuous construction is enhanced. This method is particularly economical where construction of superstructure using staging from the river bed is not feasible. Segments can be precast or cast-in-place. This type of bridge is suitable in the span range of 50 to 200 m, though a few bridges have been built beyond 200 m span.

The segmental cantilever method of construction of prestressed concrete bridges has several advantages. Centering and falsework are avoided, enabling construction of structures with tall piers and over deep valleys. The speed of construction is also increased. Quality and workmanship are enhanced due to the mechanization of repetitive tasks. By using precast segments, shrinkage effects can be avoided due to age of concrete at the time of erection and creep of concrete is less due to age at the time of initial loading. Further, precast segments lead to saving in construction time and protection from weather during concreting as precasting is done in a factory.

Time-dependent effects of creep and shrinkage of concrete and relaxation of prestressing steel affect the deformation of post-tensioned segmental bridges at each stage of construction and service life. These effects have to be considered at every stage to ensure effective control of the girder profile. In this chapter, the analysis of typical segmental bridge has been done for evaluating the long-term deformation due to time-dependent effects. Numerical studies have been performed to show the effect of various parameters on the long-term deformation. The effect of construction schedule on the bridge profile is also presented. The variation of bending moment diagram with time after the completion of construction of the bridge has also been presented. External prestressing as a solution for controlling long-term deformations and stresses has been discussed.

6.2. LONG-TERM DEFORMATION IN SEGMENTAL BRIDGES

During the erection of segments in case of segmental cantilever construction, the segments continue to deflect with time due to the time-dependent effects. When the two ends of the cantilever meet, there may be considerable difference in the ends to be joined. This difference can be reduced or eliminated by jacking or providing a predetermined camber. Therefore the estimation of time-dependent deformation is important to control the bridge profile. When structural form changes from determinate to indeterminate, the analysis becomes more involved. Time-dependent forces and stresses develop due to the restraint imposed on the deformation by indeterminacy.

6.3. ANALYSIS METHODOLOGY

The analysis of the long-term deformation has been performed using the LUSAS 14 software. At first the construction of a segmental bridge constructed by cantilever construction using multi-stage post-tensioning has been modeled and analysed. The geometry has been simplified to concentrate on the staged construction process and the definition of tendon properties. An internal span of the bridge is modeled with segments being placed from adjacent piers with a final closing segment to join the two constructions together. This example has been taken from LUSAS application examples manual.

Secondly, a three-span segmental bridge constructed by cantilever construction is modeled and analysed. The time-dependent deformations of a particular joint are studied. Further the effect of construction sequence on the difference between deflections of two meeting ends of the cantilever is evaluated. In this case, dead load and multi-stage post-tensioning are considered. Numerical studies are performed to show the effect of various parameters on the long-term deformation. The variation of bending moment diagram with time has also been presented.

6.4. ANALYSIS OF INTERNAL SPAN OF A SEGMENTAL BRIDGE

This example has been taken from LUSAS application examples manual. An internal span of the bridge is modeled with segments being placed from adjacent piers with a final closing segment to join the two cantilevers together as shown in Fig. 6.1.

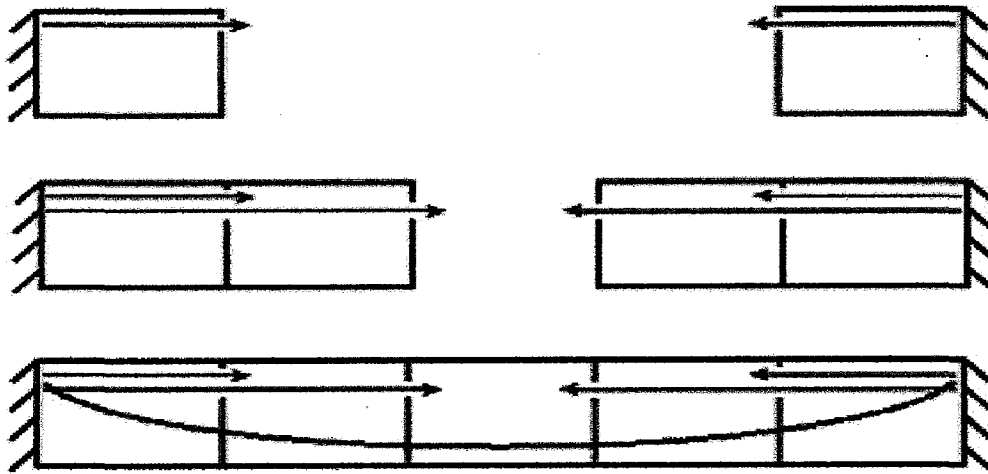


Fig. 6.1. Construction Stages of Simplified Segmental Bridge

Each stage of the construction analysis considers a 6m long section being added to the construction. The details of the modeling are as follows:

Area of cross-section: 5.36886 m²

Second moment of area: 4.63749 m⁴

Concrete grade: M45

Concrete density: 24 kN/m³

Material model for creep and shrinkage: CEB-FIP90

Diameter of cable: 47.87 mm

Modulus of elasticity of tendons: 2×10^5 N/mm²

Type of strand: Low relaxation

Prestressing force: 2000 kN in first and second stage cables and 4000 kN in the third stage cable

Friction coefficient: 0.2

Wobble coefficient: 0.0033 /m

Anchorage slip: 2 mm in first and second stages and 4 mm in third stage

The cables are stressed from one end in first and second stages and from both ends in third stage.

The structure is modeled using three-dimensional non-linear beam element. The age of the precast segment at the time of erection is taken as 14 days. Three construction stages are there and duration of each stage is 60 days.

Cable profile for first stage

X	Y	Z
0	1	0
6	1	0

Cable profile for second stage

X	Y	Z
0	1	0
12	1	0

Cable profile for third stage

X	Y	Z
0	1	0
0.978755	0.743539	0
1.96189	0.504416	0
2.9491	0.282704	0
3.94007	0.078474	0
4.93449	-0.10821	0
5.93206	-0.27729	0
6.93246	-0.42872	0
7.93538	-0.56244	0
8.94051	-0.67842	0
9.94753	-0.77662	0
10.9561	-0.857	0
11.966	-0.91955	0
12.9768	-0.96424	0
13.9882	-0.99106	0
15	-1	0
16.0118	-0.99106	0
17.0232	-0.96424	0
18.034	-0.91955	0
19.0439	-0.857	0
20.0525	-0.77662	0

21.0595	-0.67842	0
22.0646	-0.56244	0
23.0675	-0.42872	0
24.0679	-0.27729	0
25.0655	-0.10821	0
26.0599	0.078474	0
27.0509	0.282704	0
28.0381	0.504416	0
29.0212	0.743539	0
30	1	0

Analysis Results

The deformed shapes of the structure at various stages are shown in Fig. 6.2 to Fig. 6.6.

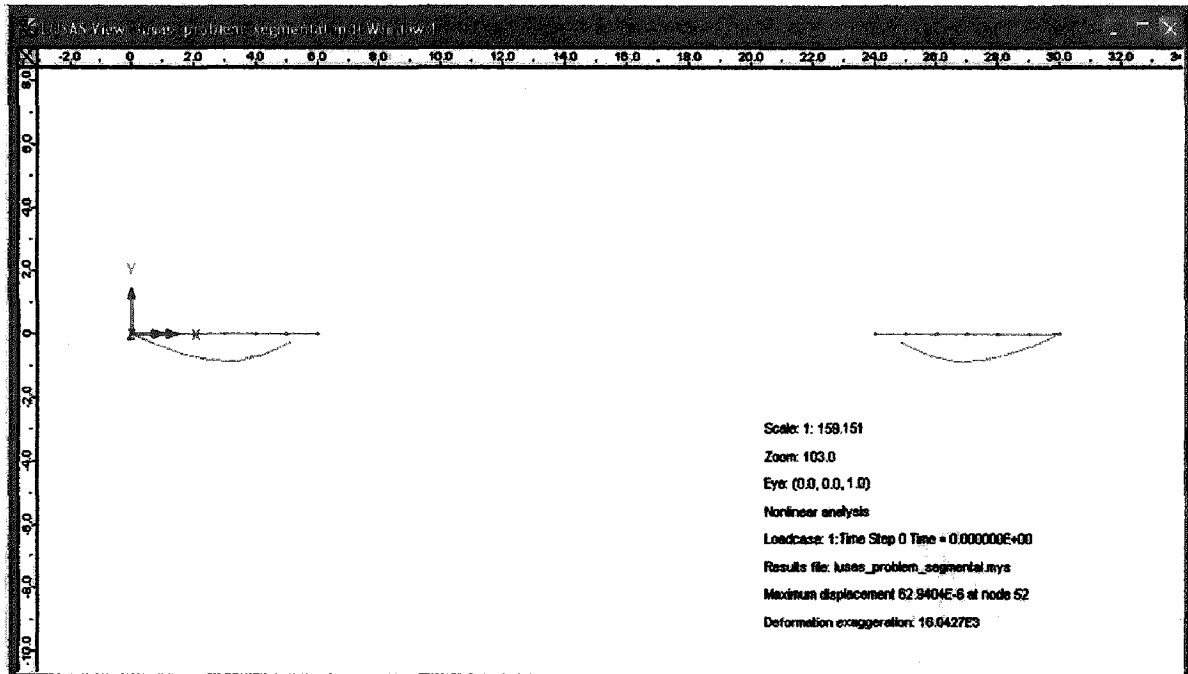


Fig. 6.2. Deformed Shape of the Bridge at 0 Day

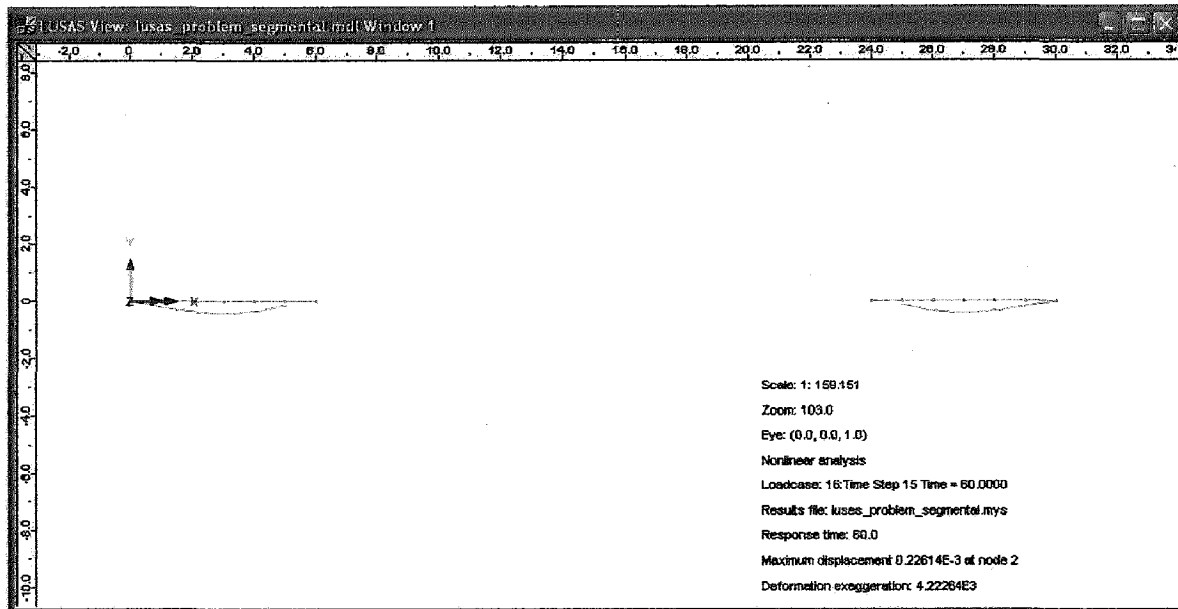


Fig. 6.3. Deformed Shape of the Bridge at 60 Days

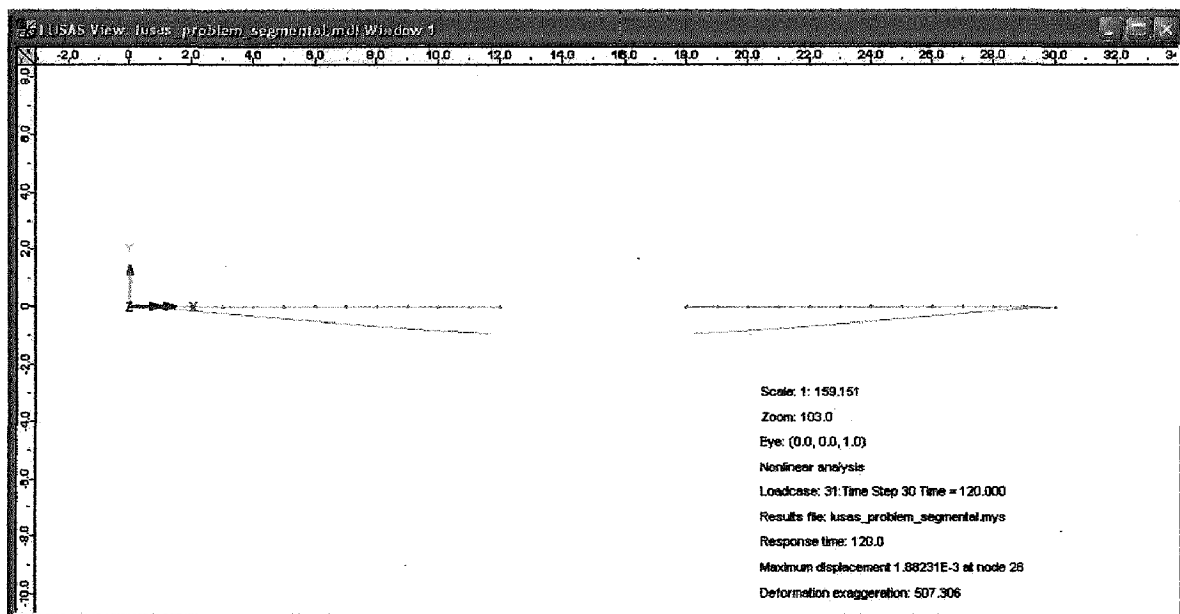


Fig. 6.4. Deformed Shape of the Bridge at 120 Days

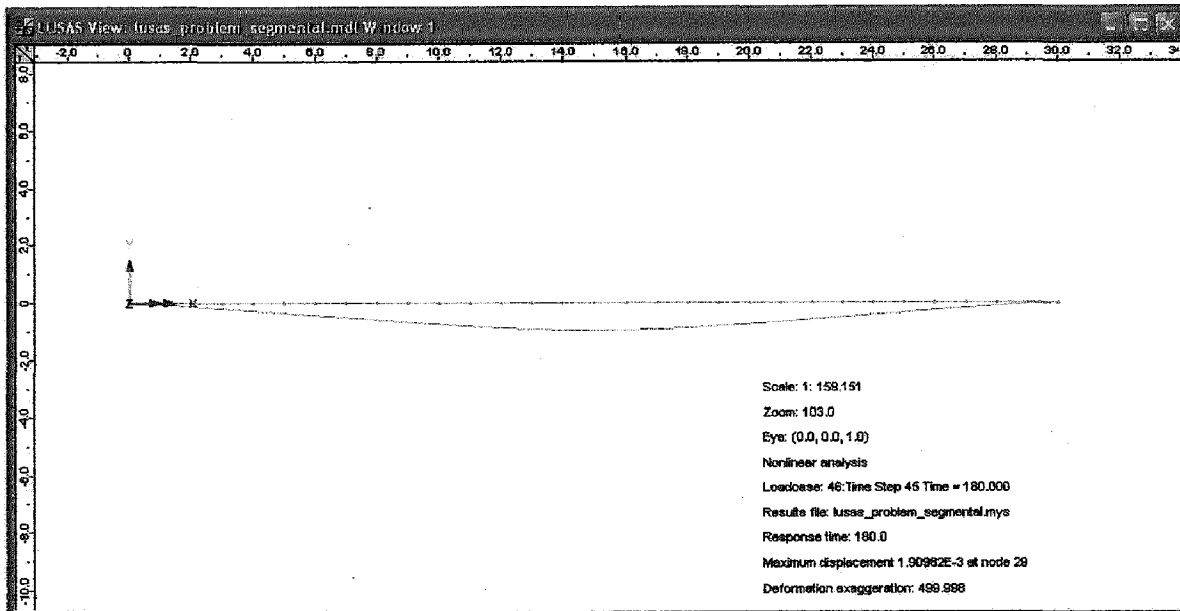


Fig. 6.5. Deformed Shape of the Bridge at 180 Days

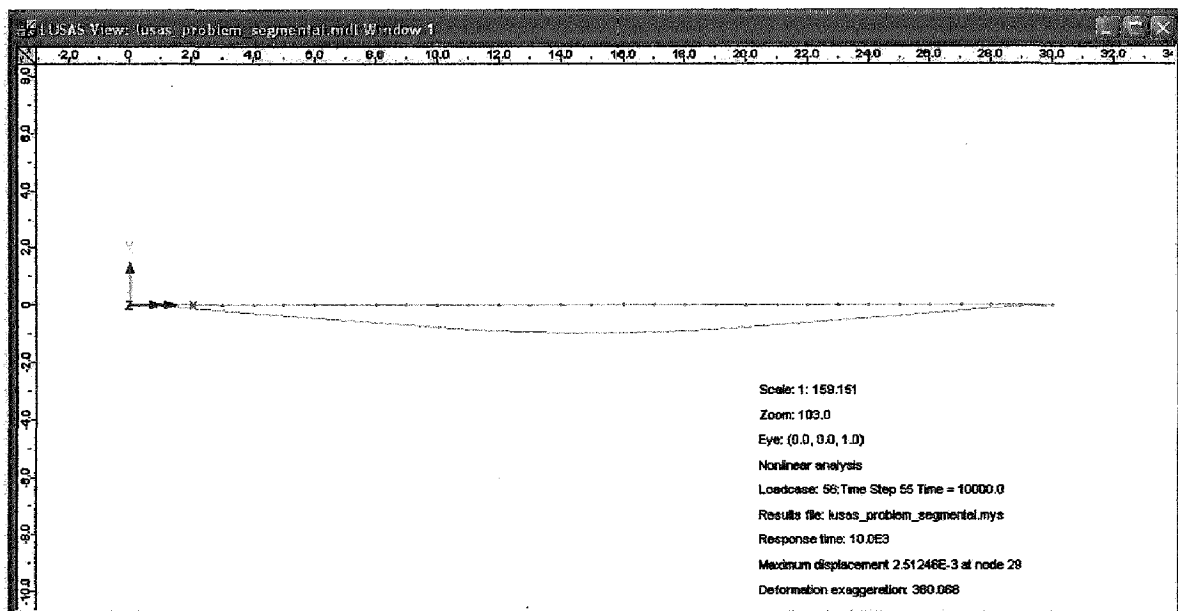


Fig. 6.6. Deformed Shape of the Bridge at 10000 Days

6.5. ANALYSIS OF SEGMENTAL BRIDGE

A three-span segmental bridge has been analyzed using LUSAS 14 software. The analysis includes different stages of bridge construction and service life. The cross-section of the bridge is box-section of constant depth which has been taken for simplicity. Each span consists of four segments in the cantilever. There are three closure segments in the side span and one closure segment in the middle span. Therefore, there are seven segments in each side-span and nine

Cable profile: Straight

The cable profile at various stages is shown in Figs. 6.8 to 6.10.

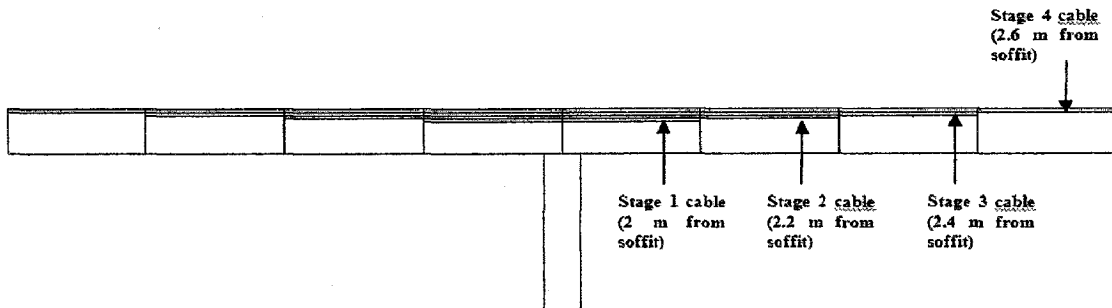


Fig. 6.8. Cable Profile in the Cantilever Stages

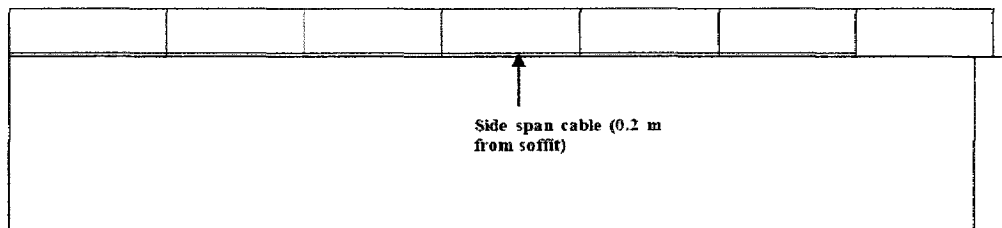


Fig. 6.9. Cable Profile in the Side Span

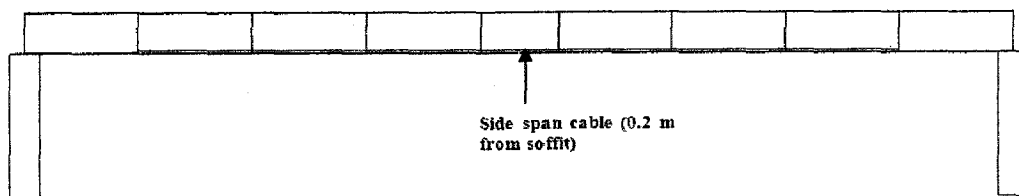


Fig. 6.10. Cable Profile in the Middle Span

The model of the bridge is meshed using three-dimensional thick non-linear beam elements. The age of the precast segment at the time of erection is taken as 28 days. There are seven construction stages. The time duration of each stage is 60 days. Therefore, the time of completion of bridge is 420 days.

6.5.1. Deflected Shape of Bridge at Various Stages of Construction

Deflected shapes of bridge at various at different stages of construction are shown from Fig. 6.11 to Fig. 6.17.

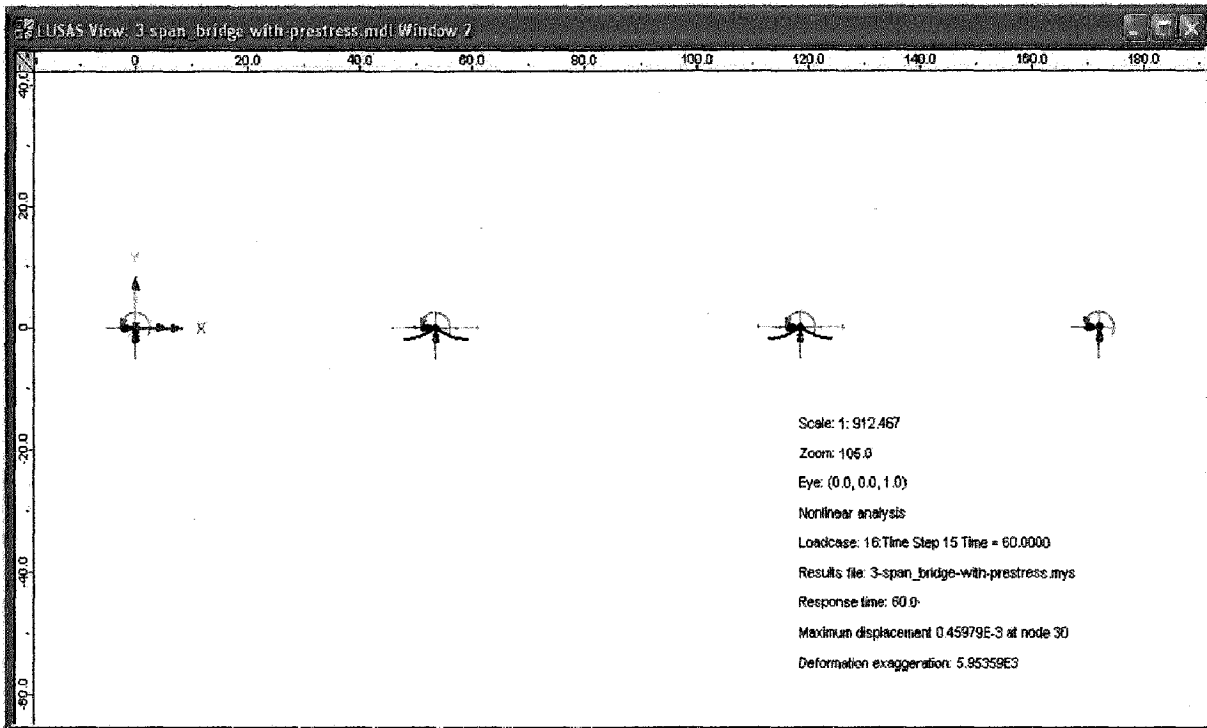


Fig. 6.11. Deflected Shape of Bridge at the End of Stage 1 (60 Days)

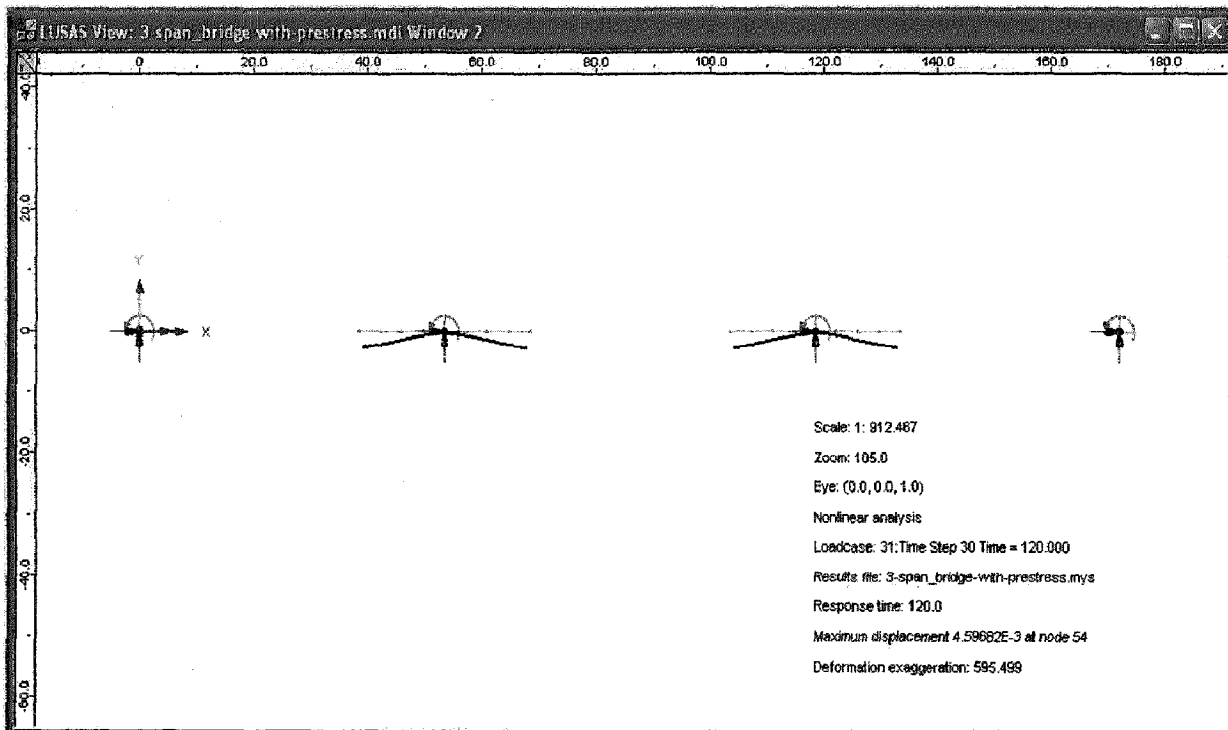


Fig. 6.12. Deflected Shape of Bridge at the End of Stage 2 (120 Days)

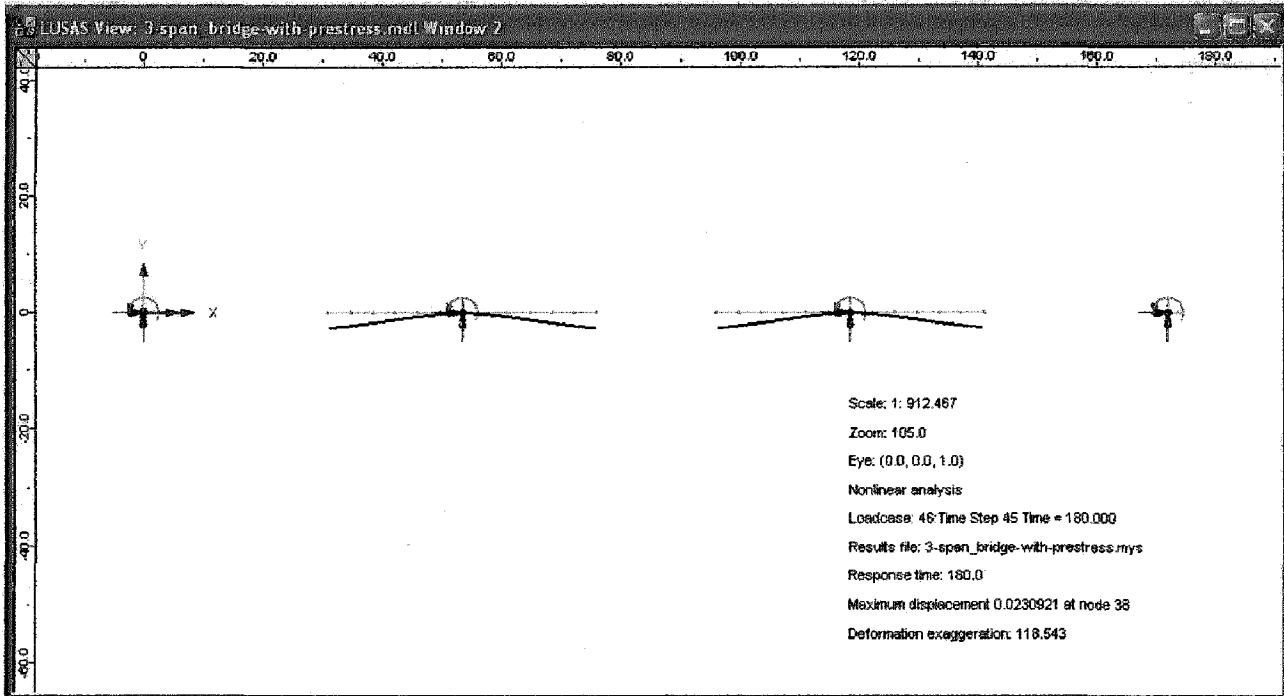


Fig. 6.13. Deflected Shape of Bridge at the End of Stage 3 (180 Days)

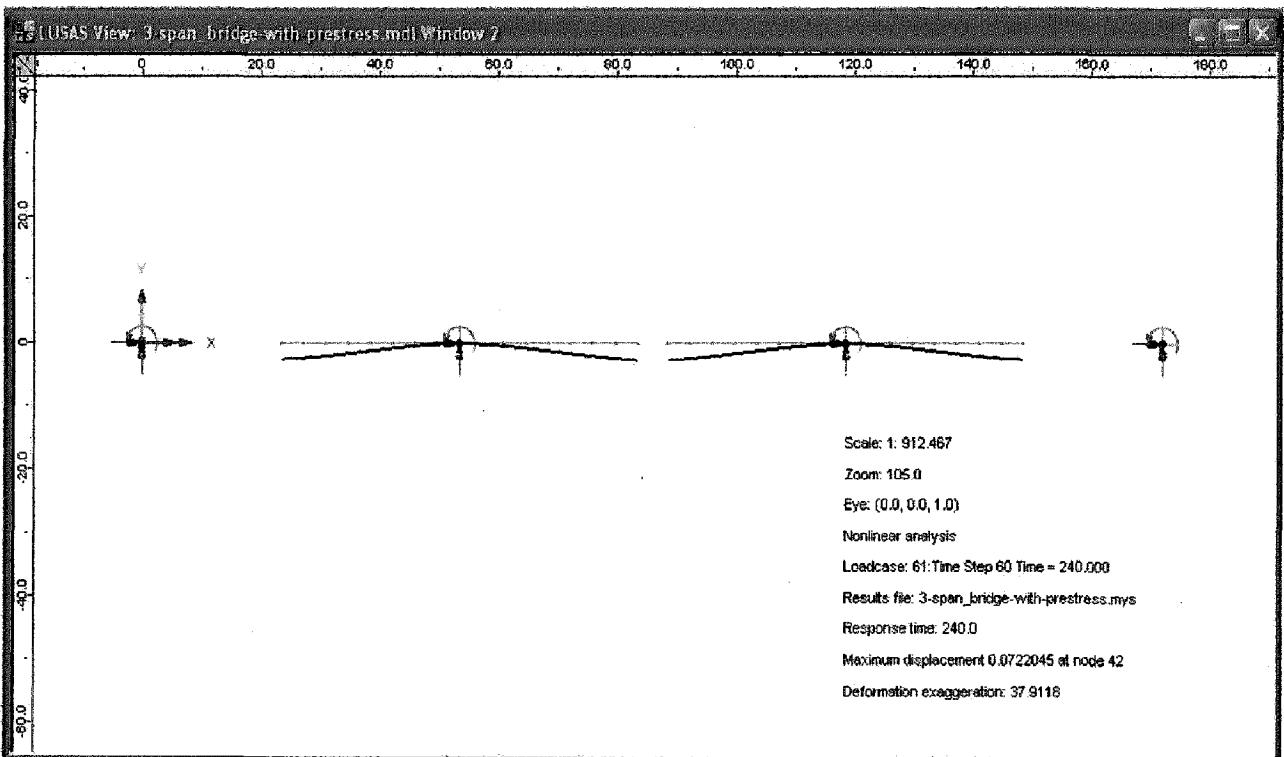


Fig. 6.14. Deflected Shape of Bridge at the End of Stage 4 (240 Days)

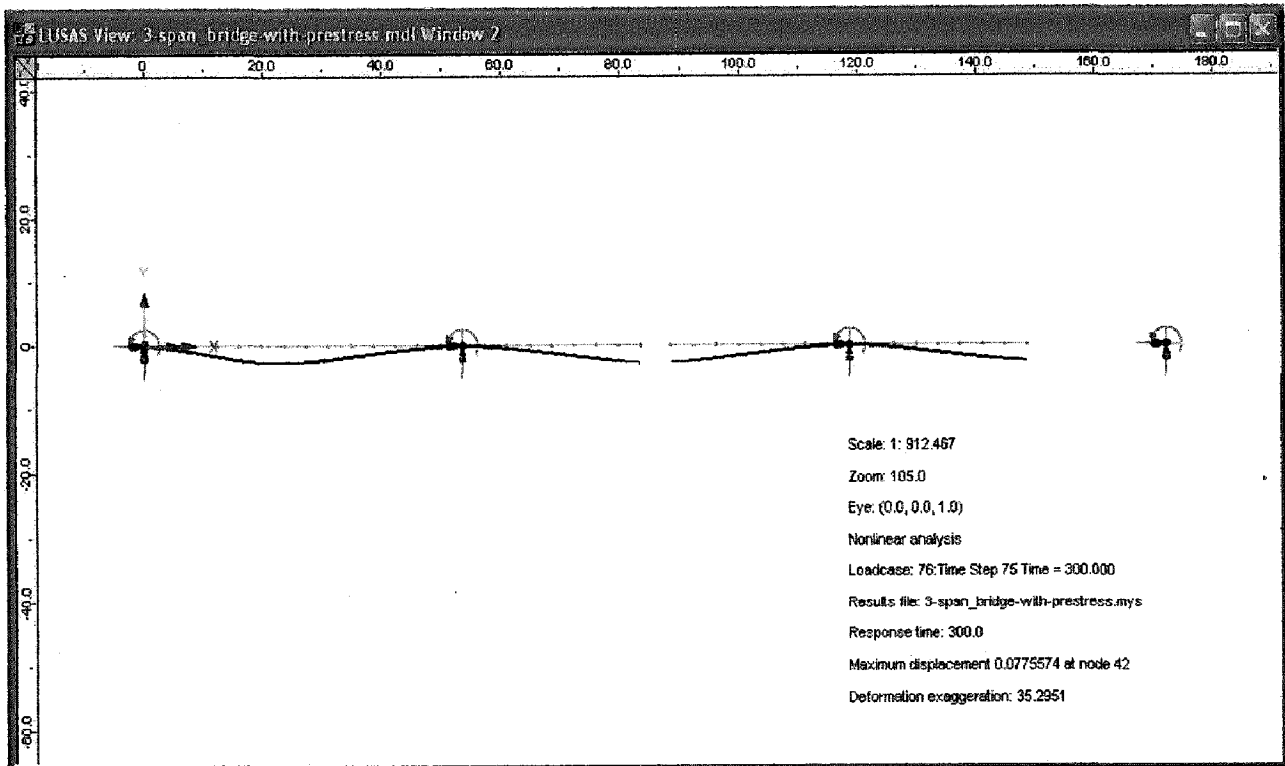


Fig. 6.15. Deflected Shape of Bridge at the End of Stage 5 (300 Days)

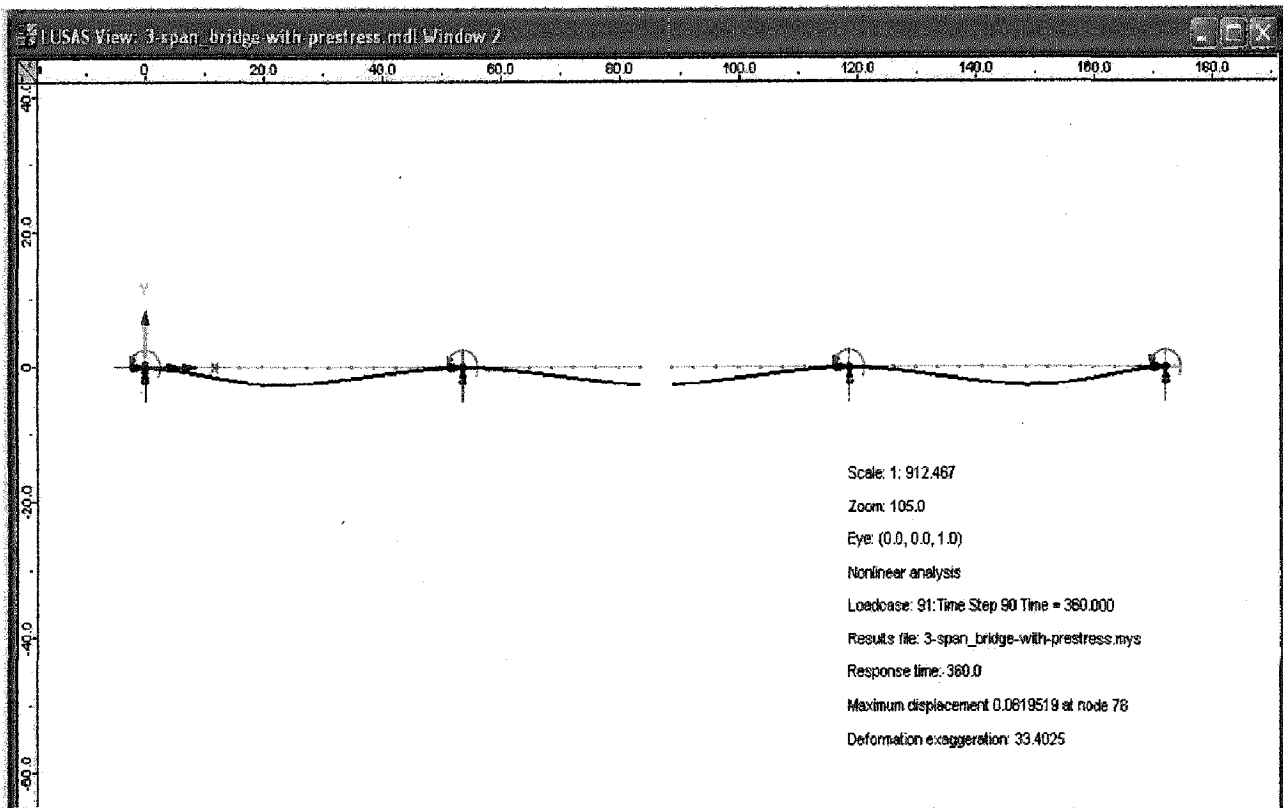


Fig. 6.16. Deflected Shape of Bridge at the End of Stage 6 (360 Days)

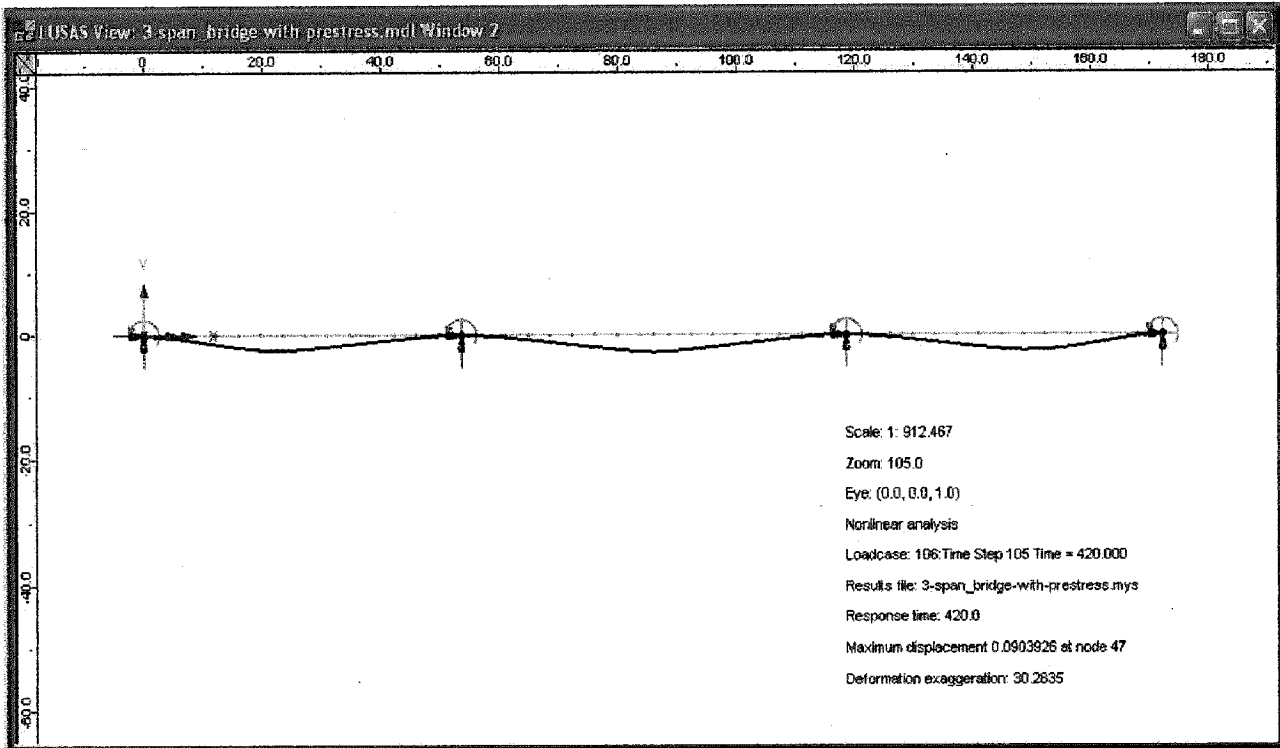


Fig. 6.17. Deflected Shape of Bridge at the End of Stage 7 (420 Days)

Table 6.1 gives the deflection at the end of cantilever segments at different stages of construction and at 10000 days.

Table 6.1. Deflection at the End of Cantilever Segments at Different Stages

Deflection (mm) at end of	Completion of erection of					Long-term deflection (10000 days)
	segment 1 (60 days)	segment 2 (120 days)	segment 3 (180 days)	segment 4 (240 days)	bridge (420 days)	
segment 1	0.3	2.2	5.4	10.2	12	14.6
segment 2	-	4.5	14.6	30.3	36.2	43.5
segment 3	-	-	23.1	53.1	64	75.7
segment 4	-	-	-	72.2	87.2	101.4

Table 6.1 gives the deflection at the end of cantilever segments at the time of completion of erection of different segments. From table 6.1, it can be observed that large portion of long-term deflection occurs during the construction stages than that after the completion of the bridge.

6.5.2. Effect of Construction Schedule

The effect of construction schedule on the deflected shape of the bridge is presented here. In the first case addition of segments from the two pier is done simultaneously while in the second case, after the completion of construction of cantilever from first pier and completion of first span only the construction from second pier is started. This can be better illustrated with the help of deformed shape of the bridge for the two cases. The two cases are presented here.

Case I: Addition of segments from two piers simultaneously

Fig. 6.18 shows the deflected shapes for this case at each construction stage before the addition of middle closure segment.

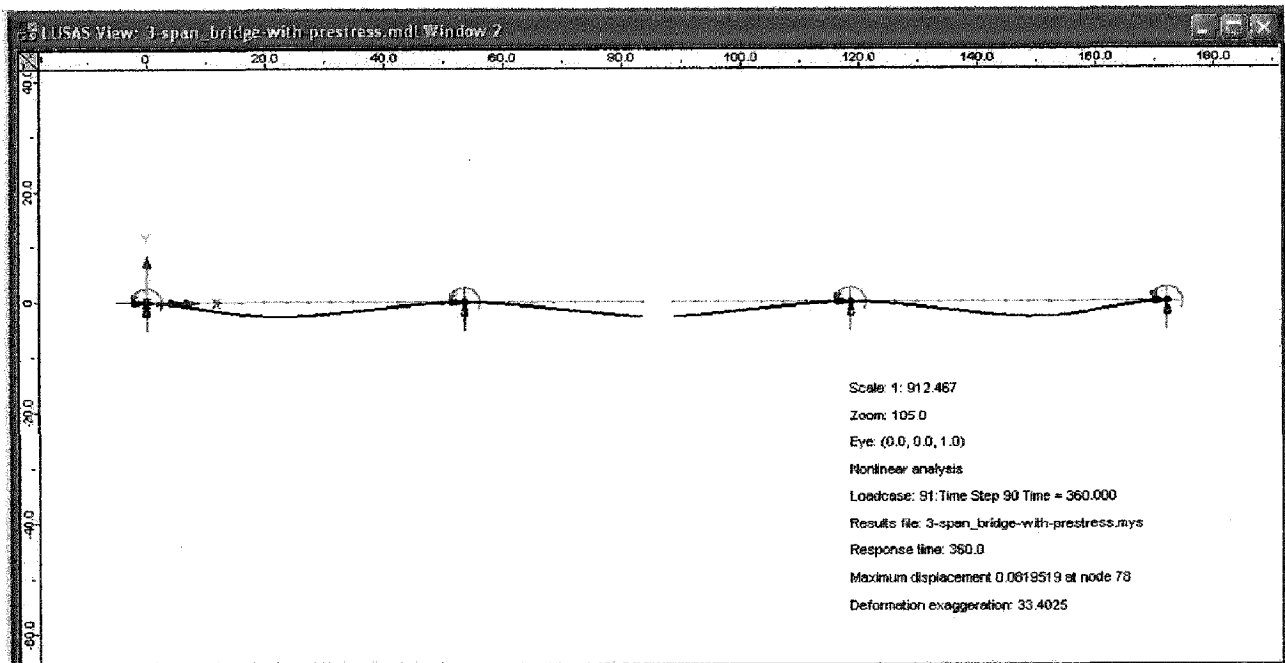


Fig. 6.18. Deflected Shape before the Addition of Closure Segment (Case I)

Case II: Addition of segments from first span and completion of first span first

Fig. 6.19 shows the deflected shapes for this case before the addition of middle closure segment.

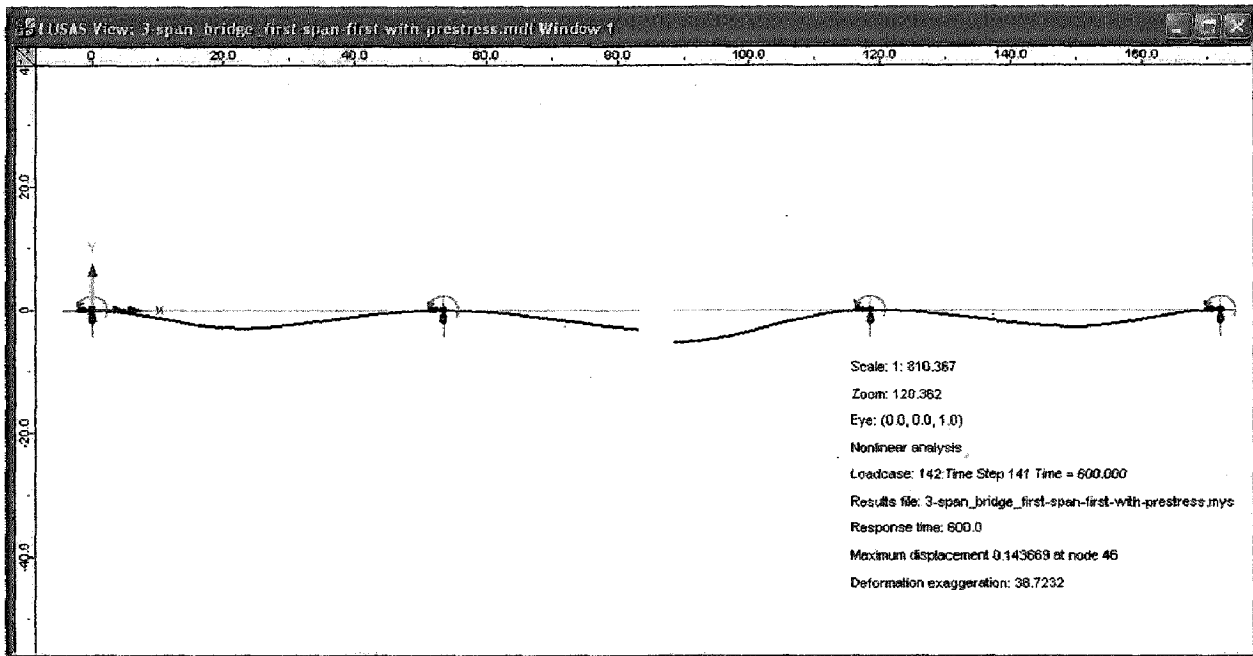


Fig. 6.19. Deflected Shape before the Addition of Closure Segment (Case II)

From Fig. 6.18 and Fig. 6.19, it is found that the difference in vertical deflection of the meeting ends of cantilever in the middle span is more in case II while it is zero in case I. From the analysis, the difference in vertical deflection of the two meeting ends is found to be 57.6 mm in case II.

6.5.3. Numerical Studies

Numerical studies are performed to show the variation of deflection with time and effect of relative humidity and age of precast segment at the time of erection on long-term deformation. The long-term deformation taken is the vertical deflection at the end of cantilever segments at 10000 days. Results are presented in the form of plots.

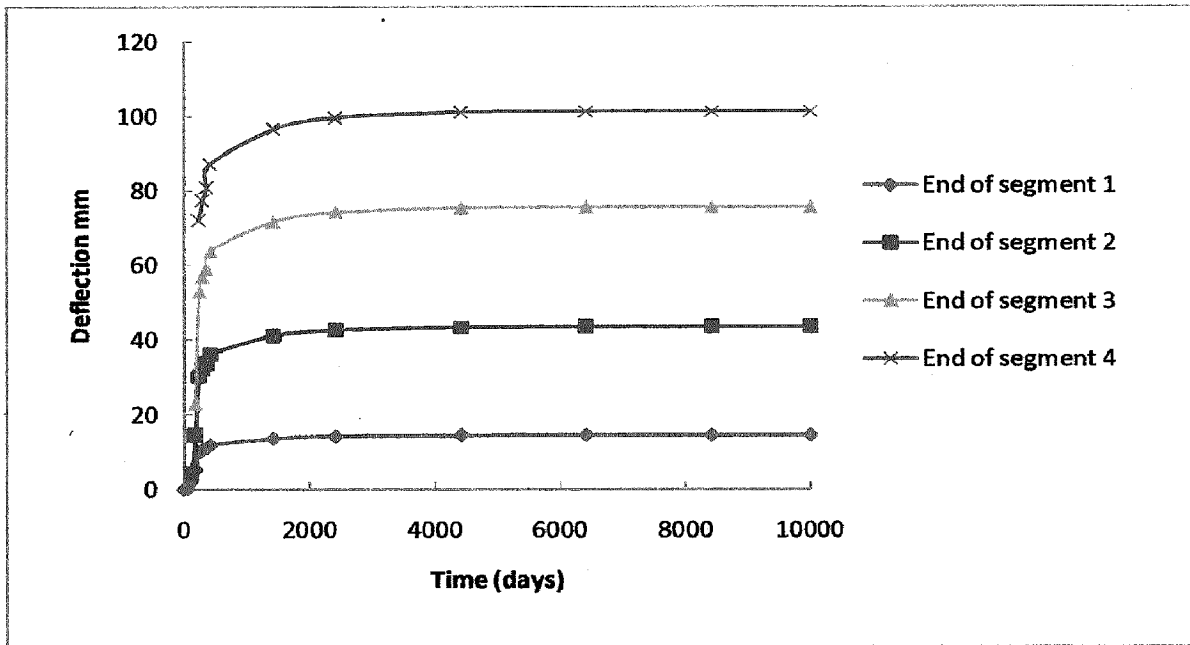


Fig. 6.20. Deflection at the End of Cantilever Segments Vs Time

Fig. 6.20 shows the variation of vertical deflection at the end of cantilever segments with time. It is observed that deflection increases with time. The rate of increase is more in the beginning while the deflection stabilizes in the end. For the end of segment 1, long-term deflection at 10000 days is found to be about 73 times of the instantaneous deflection at the start ($t=0$) and 1.22 times the deflection at the time of completion of bridge ($t=420$ days).

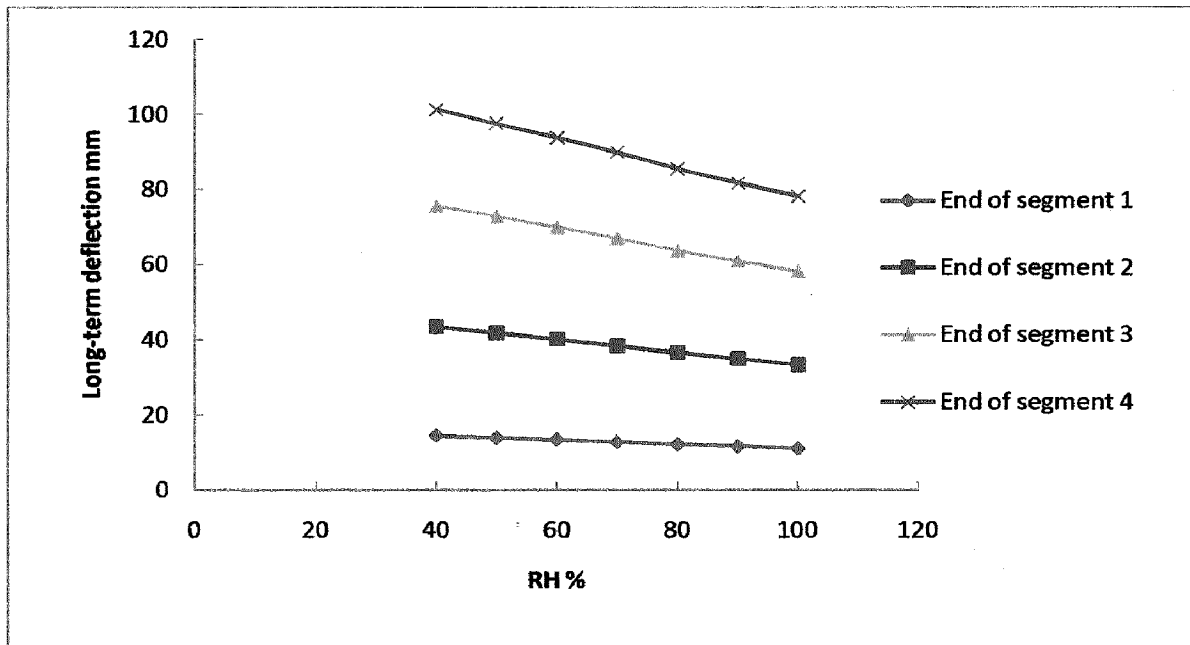


Fig. 6.21. Long-Term Deflection Vs Relative Humidity

From Fig. 6.21, it is observed that long-term deflection decreases as relative humidity increases from 40 % to 100 %. The percentage decrease in long-term deflection at the end of cantilever segments is given in Table 6.2.

Table 6.2. Effect of Relative Humidity on Long-Term Deflection

End of segment	Percentage decrease in long-term deflection at 10000 days as relative humidity increases from 40 % to 100 %
1	23.7
2	23.4
3	23.1
4	22.8

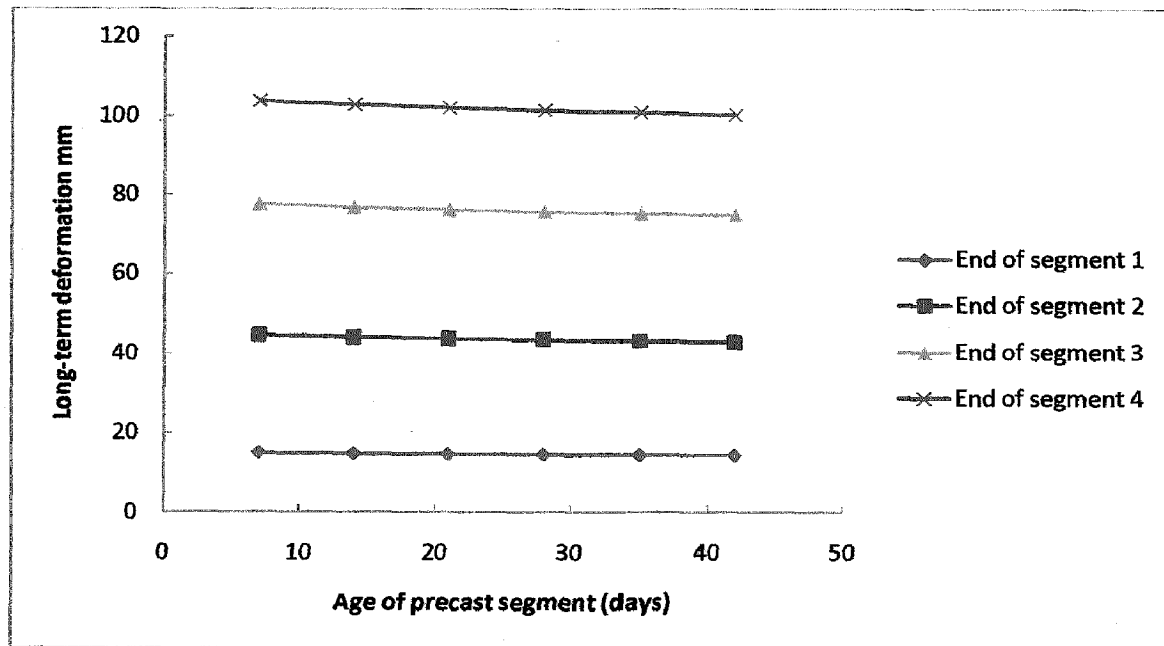


Fig. 6.22. Long-Term Deflection Vs Age of Precast Segment

From Fig. 6.22, it is observed that long-term deflection decreases as the age of precast segment at the time of erection increases from 7 days to 42 days. The percentage decrease in long-term deflection at 10000 days at the end of different segments is given in table 6.3.

Table 6.3. Effect of Age of Precast Segment at the Time of Erection on Long-Term Deflection

End of segment	Percentage decrease in long-term deflection at 10000 days as the age of precast segment increases from 7 days to 42 days
1	4
2	3.7
3	3.6
4	3.4

6.5.4. Variation of Bending Moment in the Completed Bridge with Time

Fig. 6.23 shows the bending moment diagram of the bridge at two times, i.e., at 420 days (time of completion of bridge) and at 10000 days.

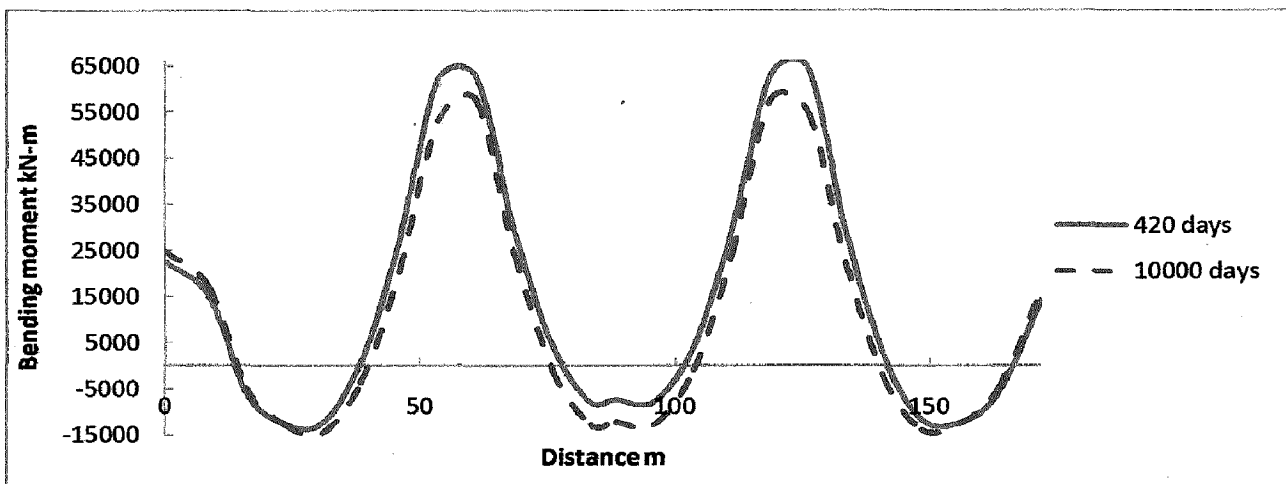


Fig. 6.23. Bending Moment Diagram of Completed Bridge

From Fig. 6.23, it can be observed that bending moment at the intermediate supports decreases with time while at the mid-span, it increases with time after the completion of construction of bridge.

6.6. EXTERNAL PRESTRESSING AS A SOLUTION FOR CONTROLLING DEFORMATION AND STRESSES

In case of indeterminate bridges constructed by segmental cantilever construction, option for external prestressing should be kept open to control deflection at any stage of construction and service life. In this section, the concept of external prestressing is discussed briefly

6.6.1. Introduction to External Prestressing

External prestressing refers to the method of prestressing where the prestressing tendons are placed on the outside of a structural member. The prestressing force is transmitted to a structural member through anchorage and deviators. The tendons are straight between points of attachments. Though tendons and concrete elements behave as different components of the overall structure, they act in unison by virtue of the connection at anchorage and deviators, thereby contributing to overall strength. Success in external post-tensioning will depend on the design of anchorage and the method of transfer of tendon forces to the concrete at the anchorage location. External prestressing is an attractive method in rehabilitation and strengthening operations because:

- It adds little weight to the original structure
- Its application poses little disturbance to users
- It allows the monitoring, re-stressing and replacement of tendons.

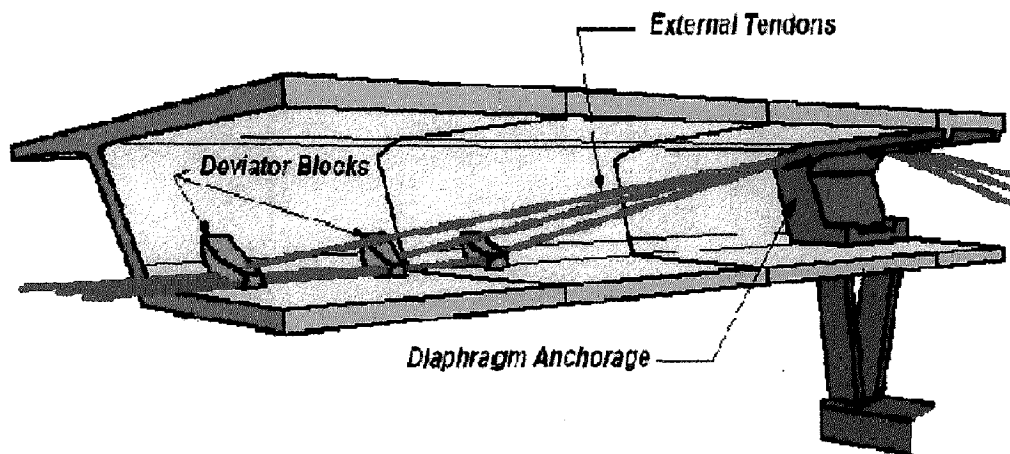


Fig. 6.24. Typical layout of an externally post-tensioned box girder bridge

6.6.2. Internal Vs External Prestressing

In case of internal prestressing, the compatibility of strains between the concrete and the tendon is a basic design assumption in calculations for stress and strains.

On the other hand, if tendons are un-bonded, as in case of external prestressing, the analysis of a section based on strain compatibility is in general inaccurate. Rather, the compatibility should apply to the displacements of concrete and tendons at the anchorages and also at deviators that do not allow slippage.

6.7. SUMMARY

In this chapter, the significance of time-dependent deformation calculation at various stages of construction and service life of bridges constructed using segmental cantilever construction has been presented. The effect of construction schedule on the deflected profile of the bridge has been illustrated. Further, numerical studies have been performed to evaluate the variation of deflection with time and the effect of various parameters like relative humidity and age of precast girder at the time of erection. Variation of bending moment with time after the completion of bridge has been presented. External prestressing as a solution for controlling long-term deformations and stresses has been discussed.

DISCUSSION OF RESULTS AND CONCLUSION

7.1. GENERAL

The objective and scope of present work has been outlined in chapter 1. Based on the objective and scope, studies were carried out to predict the long-term behaviour of determinate and indeterminate post-tensioned bridge structures. In chapter 4, long-term deformation and prestress loss of simply-supported PSC bridge girder have been predicted and numerical studies have been done to show the effects of various parameters. In chapter 5, time-dependent restraint moment at the intermediate support of a two-span precast, prestressed concrete bridge girders made continuous using continuity diaphragm has been predicted. Results of the numerical studies have been presented for this case. In chapter 6, long-term deformation of segmental bridge constructed by cantilever construction and results of numerical studies have been presented. In this chapter, results obtained from different studies have been discussed. Conclusions and scope of future research have also been presented.

7.2. LONG-TERM DEFORMATION OF DETERMINATE BRIDGE STRUCTURES

The results of the analysis of simply-supported bridge girder for long-term deformation and prestress loss, given in chapter 4, are presented here.

7.2.1. Effect of Material Model

- CEB-FIP90 model predicts less prestress loss and more deflection compared to ACI 209R-92 model. The difference between prestress losses predicted by two models is more in the beginning and less in the end.
- Long-term deflection due to self-weight at 10000 days becomes 3.63 times the initial deflection using CEB-FIP90 model while it becomes 3.03 times the initial deflection using ACI 209R-92 model.
- Long-term deflection due to prestress at 10000 days becomes 3.09 times the initial deflection using CEB-FIP90 model while it becomes 2.5 times the initial deflection using ACI 209R-92 model.
- Total long-term deflection due to self-weight and prestress at 10000 days becomes 2.16 times the initial deflection for CEB-FIP90 model while it becomes 1.58 times the initial deflection for ACI 209R-92 model.

7.2.2. Effect of Relative Humidity

As the ambient relative humidity increases from 40 % to 100 %, following effects are observed:

- Long-term prestress loss at 10000 days decreases by about 48 % using CEB-FIP90 model and by 62.4 % for ACI 209R-92 model.
- Long-term deflection due to self-weight at 10000 days decreases by 30.4 % using CEB-FIP90 model and by 27 % using ACI 209R-92 model.
- Deflection due to prestress decreases by 27 % using CEB-FIP90 model and by 19 % for ACI model.

7.2.3. Effect of Temperature

The effect of temperature on long-term deformation and prestress loss has been studied using only CEB-FIP90 model. As the temperature increases from 0°C to 50°C, the following effects are observed:

- Long-term prestress loss becomes about 2 times.
- Long-term deflection due to self-weight becomes 1.86 times.
- Long-term deflection due to prestress becomes 1.72 times.
- Total long-term deflection due to self-weight becomes 1.38 times.

7.2.4. Discussion of Results

- Deflection due to self-weight increases with time in the downward direction because creep coefficient increases with time. Deflection due to prestress also increases with time in the upward direction but not to the same extent as that due to self-weight because of reduction in prestress force due to losses of prestress.
- With an increase in the relative humidity, creep of concrete decreases. Therefore, this results in a reduction in long-term deflection due to self-weight and prestress.
- As relative humidity increases, long-term prestress loss reduces due to reduction in the creep and shrinkage of concrete.
- As temperature increases, creep and shrinkage of concrete decreases. Therefore, this results in an increase in long-term prestress loss and deflection due to self-weight and prestress.

7.3. LONG-TERM BEHAVIOUR OF PRECAST, PRESTRESSED CONCRETE BRIDGE GIRDERS MADE CONTINUOUS

The results of the time-dependent restraint moment at the intermediate support of a two-span bridge girder made continuous using cast-in-place diaphragm, given in chapter 5, are presented here:

7.3.1. Effect of Analysis Approach

- Two methods for the analysis of restraint moment has been used namely PCA method and Trost's approach.
- Restraint moment increases with time for both the methods.
- The rate of increase of restraint moment with time is more in the beginning and less in the end.
- PCA method predicts more restraint moment than that predicted by Trost's approach using both CEB-FIP90 and ACI 209R-92 model.

7.3.2. Effect of Material model

- Restraint moment increases with time for both CEB-FIP90 and ACI 209R-92 models.
- Restraint moment predicted by CEB-FIP90 model is more than that predicted by ACI 209R-92 model using both the analysis methods.

7.3.3. Effect of Age of Girder at the time of Establishment of Continuity

The effect of age of girder at the time of establishment of continuity on the long-term restraint moment at 10000 days has been studied using CEB-FIP90 model.

- As the age of precast, prestressed girder at the time of establishment of continuity increases from 30 days to 300 days, the restraint moment decreases.
- The decrease in restraint moment is 27.5 % using Trost's approach and 26.4 % using PCA method. The material model used is CEB-FIP90.

7.3.4. Effect of Relative Humidity

As the relative humidity increases from 40 % to 100 %, following effects are observed in the long-term restraint moment at 10000 days:

- Using CEB-FIP90 model, restraint moment decreases by 23 % for Trost's approach and by 23.7 % for PCA method.

- Using ACI 209R-92 model, restraint moment decreases by 28.7 % for Trost's approach and by 30.4 % for PCA method.

7.3.5. Effect of Temperature

The effect of temperature on the long-term restraint moment at 10000 days has been studied using CEB-FIP90 model. As the temperature increases from 0°C to 50°C, the following effects are observed:

- Restraint moment increases by 36 % for Trost's approach and 32.2 % for PCA method.

7.3.6. Discussion of Results

- PCA method gives a conservative estimate of the restraint moment at the intermediate support of a precast, prestressed girder made continuous using cast-in-place diaphragm. Also, using a particular method, the restraint moment is more when CEB-FIP90 model is used.
- As the age of the girder at the time of establishment of continuity increases, restraint moment decreases because the rotation at the support up to the time of establishment of continuity increases as the girder remains simply-supported for more time. This results in a decrease in the rotation which is restrained from the time of establishment of continuity to a particular time.
- The restraint moment decreases with an increase in relative humidity because creep of concrete decreases which results in a reduction in the creep rotation which is restrained.
- The restraint moment increases with an increase in temperature because creep of concrete increases which results in an increase in the creep rotation which is restrained.

7.4. LONG-TERM DEFORMATION OF SEGMENTAL BRIDGES

The results of the analysis of a three-span segmental bridge using LUSAS 14 software, given in chapter 6, are presented here. For modeling the behaviour of creep and shrinkage of concrete, only CEB-FIP90 model has been used. Deflections due to dead load and prestress have been considered.

7.4.1. Variation of Deflection with Time

- Long-term deflection at the end of first segment at 10000 days is found to be about 73 times of the instantaneous deflection at the start ($t=0$) and 1.22 times the deflection at the time of completion of bridge ($t=420$ days).

7.4.2. Effect of Construction Schedule

- Two cases of construction sequence have been considered. In case I, the addition of precast segments from both the piers is done simultaneously while in the in case II, cantilever from first pier and first span are completed first. It is found that the difference in vertical deflection of the meeting ends of cantilever in the middle span is more in case II while it is zero in case I.
- From the analysis, the difference in vertical deflection of the two meeting ends is found to be 57.6 mm in case II.

7.4.3 Effect of Relative Humidity

- As the relative humidity increases from 40 % to 100 %, long-term deflection at 10000 days decreases. The quantitative information is given in table 6.2 in chapter 6.

7.4.4. Effect of Age of Precast Segment at the Time of Erection

- As the age of precast segment at the time of erection increases from increases from 7 days to 42 days, long-term deflection at 10000 days decreases. The quantitative information is given in table 6.3 in chapter 6.

7.4.5. Variation of Bending Moment in the Completed Bridge with Time

- Bending moment at the intermediate supports decreases with time while at the mid-span, it increases with time after the completion of construction of bridge.

7.4.6. Discussion of Results

- Long-term deflection at the end of first segment at 10000 days is about 73 times the initial deflection while it is only 1.22 times the deflection at the time of completion of bridge. This is because during the construction stages, increase in load due to the addition of segments and post-tensioning coupled with the time-dependent effects results in a large increase in deflection. But, after the completion of bridge, structural

form changes to indeterminate which results in restraint in the time-dependent deflection. Therefore, the deflection does not increase to the same extent as in case of simply-supported bridge structures which are determinate.

- With an increase in relative humidity, long-term deflection decreases due to reduction in the creep of concrete.
- Long-term deflection at 10000 days decreases with an increase in the age of precast segment at the time of erection because creep and shrinkage of concrete reduces as the age of concrete at the time of application of loading increases.
- The construction sequence is very important in the case of segmental cantilever construction of bridge to prevent misalignment of the segments. There should be provision for external post-tensioning so that the profile of the bridge girder can be adjusted by stressing the external tendons.
- Due to restraint imposed on the deformation due to creep and shrinkage of concrete due to indeterminacy, bending moment changes with time even after the completion of construction of bridge.

7.5. CONCLUSIONS

- Time-dependent effects of creep, shrinkage and relaxation affect the long-term deformation and prestress loss significantly. Prediction of these quantities depends on material model as well as analysis approach. Therefore, it is very important to select a suitable material model for a particular environment.
- Environmental parameters such as relative humidity and temperature have a significant impact on the long-term deformation and prestress losses. Therefore, these parameters should be kept in mind while designing a prestressed concrete bridge.
- Due to the restraint in the deformation in case of indeterminate bridge structures, time-dependent indeterminate forces and moments develop. Restraint moments develops at intermediate support of the precast, prestressed concrete girder made continuous using cast-in-place diaphragm due to the restraint caused by the indeterminacy on the time-dependent deformations. The prediction of the time-dependent restraint moment is very important to design proper reinforcement for the continuity diaphragm.
- The accurate prediction of time-dependent deformation is of extreme importance in case of indeterminate bridge structures like continuous and rigid frame bridges which

are constructed by segmental cantilever construction. The excessive long-term deflection during the construction stages can result in misalignment of the meeting ends of the cantilever.

From the studies conducted and results obtained, it can be concluded that time-dependent effects of creep, shrinkage and relaxation affects the behaviour of both determinate and indeterminate bridge structures. In case of determinate bridge structures, the deformations due to the time-dependent effects are unrestrained, thereby no additional stresses are introduced. But, in case of indeterminate bridge structures, restraint imposed on the time-dependent deformations due to indeterminacy results in additional stresses which should be accounted for in the design. In case of bridges constructed by segmental cantilever construction, time-dependent deformations during the construction stages can result in misalignment of the joining ends. In such situation where structural form changes from determinate during construction to indeterminate during service, both the problems of deformations and stresses become significant.

7.6. SCOPE FOR FUTURE RESEARCH

The following can be the scope of the future research work:

- Prediction of long-term deformation of composite prestressed concrete bridge considering multi-stage prestressing.
- Assessment of the restraint moment due to the effects of both creep and differential shrinkage of a precast, prestressed bridge girder made continuous using cast-in-place deck and diaphragm.
- Development of computer program for the prediction of long-term deformation of indeterminate post-tensioned bridges like continuous and rigid frame bridges constructed by segmental cantilever construction at different stages of construction and service life.

REFERENCES

- Aalami, B. O. (1998), "Time-dependent Analysis of Post-tensioned Concrete Structures", *Progress in Structural Engineering and Materials*, 1 (4), 384-391.
- ACI Committee 209. (1992), "Prediction of Creep, Shrinkage, and Temperature Effects in Concrete Structures", (ACI 209R-92). *American Concrete Institute*, Farmington Hills, MI, 249-260.
- Al-Manaseer, A. and Lam, J. (2005), "Statistical Evaluation of Shrinkage and Creep Models", *ACI Materials journal*, 102(3), 170-176.
- Bazant, Z. P., and Baweja, S., "Creep and Shrinkage Prediction Model for Analysis and Design of Concrete Structures: Model B3", Structural Engineering Report 94-10/603c, Northwestern University, (1994); Published as draft RILEM recommendation in Materials and Structures (RILEM Paris), 28 (1995), 357-365, 415-430, 488-495.
- Bazant, Z. P. (2001), "Prediction of Concrete Creep and Shrinkage: Past, Present and future", *Nuclear Engineering and Design*, 203, 27-38.
- Bazant, Z. P., Yu, Q., Li, G. H., Klein, G. J. and Kristek, V. (2010), "Excessive Deflections of Record Span Prestressed Box Girder", *Concrete International*, 32(6), 44-52.
- Carreira, D. J., Daye, M. and Greening, L. R. (2000), "Simplified Method to Account for Effects of Creep and Shrinkage in Structural Analysis", *ACI Special Publication*, 94, 237-260.
- CEB-FIP Model Code. (1990), Design Code, *Comité Euro-international du Béton*.
- Gardner, N. J. and Lockman, M. J. (2001), "Design Provisions for Drying Shrinkage and Creep for Normal Strength Concrete", *ACI Materials Journal*, 98(2), 159-167.
- Goel, R., Kumar, R. and Paul, D. K. (2007), "Comparative Study of Various Creep and Shrinkage Prediction Models of Concrete", *Journal of Materials in Civil Engineering. ASCE*, 249-260.
- Huo, X. S., Al-Omaishi, N. and Tadros, M. K. (2001), "Creep, Shrinkage and Modulus of Elasticity of High Performance concrete", *ACI Materials Journal*, 440-448.
- Karthikeyan, J., Upadhyay, A. and Bhandari, N. M. (2008), "Artificial Neural Network for Predicting Creep and Shrinkage of High Performance Concrete", *Journal of Advanced Concrete Technology*, 6(1), 135-142.
- Lopez, M., Kahn, L. F. and Kurtis, K. E. (2004), "Creep and Shrinkage of High-Performance Lightweight Concrete", *ACI Materials Journal*, 101(5), 391-399.
- Nawy, E. G. (2003), *Prestressed Concrete: A Fundamental Approach*, 4th edition. Prentice- Hall, Upper Saddle River, NJ.

- PCI Committee on Prestress Losses. (1975), "Recommendations for Estimating. Prestress Losses", *PCI Journal*, 20(4), 43-75.
- Russel, H. G. and Larson, S. C. (1989), "Thirteen years of deformations in water tower place", *ACI Structural Journal*, 86(2), 182-191.
- Stallings, J. M., Barnes R. W. and Eskildsen, S. (2003), "Camber and Prestress Losses in Alabama HPC Bridge Girders", *PCI Journal*, 48(5), 90-104.
- Tadros, M. K., Fawzy, F. and Hanna, K. E. (2011), "Precast, Prestressed Girder Camber Variability", *PCI Journal*, 56(1), 135-154.
- Youakim, S. A., Ghali, A., Hida, S. E. and Karbhari, V. M. (2007), "Prediction of Long-Term Prestress Losses", *PCI Journal* 52(2), 116-130.
- Tadros, M. K., Al-Omashi, N., Seguirant, S. J., and Gallt, J. G., "NCHRP Report 496", Washington D.C., 2003.
- Martin D. L. (1977), "A Rational Method for Estimating Camber and Deflection of Precast Prestressed Members", *PCI Journal*, January-February, 100-108.
- PCI Committee on Prestress Losses. (1975), "Recommendations for Estimating. Prestress Losses", *PCI Journal*, 20(4), 43-75.
- Mattock, A. H., "Precast-Prestressed Concrete Bridges, 5. Creep and Shrinkage Studies", *Journal of PCA Research and Development Laboratories*, V. 3, No. 2, May 1961, pp. 32-65. (Also reprinted as PCA Bulletin D46.)
- Freyermuth, C. L., "Design of Continuous Highway Bridges with Precast, Prestressed Concrete Girders", V. 14, N0. 2, April 1969, pp. 14-39. (Also reprinted as PCA Engineering Bulletin, No. EB014.01E, August 1969.)
- Oesterle, R. G., Glikin, J. D., and Larson, S. C., "Design of Precast Prestressed Bridge Girders Made continuous", NCHRP Report 322, *Transportation Research Board, National Research Council*, Washington, DC, November, 1989, 97 pp.
- Peterman, R. J., and Ramirez, J. A., "Restraint Moment in Bridges with Full-Span Prestressed Concrete Form Panels", *PCI Journal*, V. 43, No. 1, January-February 1998, pp. 54-73.
- Mirmiran, A., Kulkarni, S., Castrodale, R., Miller, R., and Hastak, M., "Non-linear Continuity Analysis of Precast, Prestressed Concrete Girders with Cast-in-Place Decks and Diaphragms", *PCI Journal*, V. 46, No. 5, September-October 2001, pp. 60-80.
- McDonagh, M. D., and Hinkley, K. B., "Resolving Restraints nad Designing for Continuity in Precast Prestressed Concrete Girder Bridges", *PCI Journal*, July-August 2003, pp. 2-17.
- Ma, Z., Huo, X., Tadros, M. K., Baishya, M., "Restraint Moments in Precast/Prestressed Concrete Continuous Bridges", *PCI Journal*, November-December 1998, pp. 40-48.

- Newhouse, C. D. (2005). "Design and Behavior of Prestressed, Precast Girders Made Continuous – An Analytical and Experimental Study." Ph.D. Dissertation, Virginia Tech.
- Koch, S., Prestressed concrete PCBT Girders Made Continuous and Composite with a Cast-in-Place Deck And Diaphragm. Virginia Polytechnic Institute and State University.
- Tadros, M. K., Ghali, A., Dilger, W. H., "Long term stresses and deformation of segmental bridges", *PCI Journal*, July-August 1979, pp. 66-87.
- Guo-jing, H., Yuan-yuan, L. I., Zhong-quan, Z., and Liang-liang, D.. "Effect of creep on pre-camber of rigid frame bridges", *J. Cent. South Univ. Technol.* (2008) 15(s1): 337-341.
- Herbert, J. T., "Computer Analysis of Deflections and Stresses in Stage Constructed Concrete Bridges", *PCI Journal*, May-June 1990, pp. 52-63.
- Shushkewich, W. K., "Time-Dependent Analysis of Segmental Bridges", *Computers & Structures*, Vol. 23, No. 1, 1986, pp. 95-118.
- Malm, R., Sundquist, H., "Time-Dependent Analyses of Segmentally Constructed Balanced Cantilever Bridges", *Engineering Structures*, 32, 2010, pp. 1038-1045.
- IRC:18-2000, Design Criteria for Prestressed Concrete Road Bridges (Post-tensioned Concrete).
- IS:1343,-1980, Code of Practice for Prestressed Concrete.

BIBLIOGRAPHY

Nawy, E. G. (2003), *Prestressed Concrete: A Fundamental Approach*, 4th edition. Prentice- Hall, Upper Saddle River, NJ.

A. H. Nilson. (1987), *Design of Prestressed Concrete*, Second edition, John Wiley & Sons.

Rajagopalan, N. (2005), *Prestressed Concrete*, Second Edition, Narosa Publishing House, India

Rajagopalan, N. (2009), *Bridge Superstructure*, Narosa Publishing House, India

Victor, D. J., *Essentials of Bridge Engineering*, Sixth edition, Oxford and IBH Publishing Co. Pvt. Ltd., New Delhi, India.

Krishna Raju, N., *Prestressed Concrete Bridges*, CBS Publishers and Distributors, India.

Applications Examples Manual (Bridge, Civil and Structural), LUSAS 14.

LIST OF TECHNICAL PAPERS COMMUNICATED

Farooqui, M. A. E., and Upadhyay, A., "Effect of Material Models on Long-Term Deformation of PSC Bridge Girder", *Structural Engineering and Mechanics*, Submitted on 2nd May 2011.

Farooqui, M. A. E., and Upadhyay, A., "Prediction of Long-Term Behaviour of PSC Bridge Girders", *National Seminar*, NBRDC, Hyderabad, Submitted on 26th May 2011.

LONG-TERM PRESTRESS LOSS CALCULATION USING IRC:18-2000

Input Data

The details of the post-tensioned bridge girder are as follows:

Girder Details:

- Span: 30 m
- Overall depth: 1.8 m
- Area of cross-section at mid-span, A_c : 0.77 m²
- Area of cross-section at support, A_s : 1.075 m²
- Second moment of area of cross-section at mid-span, I_{zc} : 0.2991 m⁴
- Second moment of area of cross-section at support, I_{zs} : 0.3319 m⁴
- Distance of CG of cross-section from top at mid-span, y_{tc} : 0.726 m
- Distance of CG of cross-section from top at support, y_{ts} : 0.774 m
- Distance of CG of cross-section from bottom at mid-span, y_{bc} : 1.074 m
- Distance of CG of cross-section from bottom at support, y_{bs} : 1.026 m
- Perimeter of the cross-section exposed to atmosphere at mid-span: 3.6 m
- Perimeter of the cross-section exposed to atmosphere at support: 3.6 m

Cable Details:

- Cable profile: Triangular
- Cable height from soffit at mid-span: 0.2 m
- Cable height from soffit at support: 0.870 m
- Cable hold-down point from support, a : 15 m
- Area of prestressing steel, A_{ps} : 6351 mm²

Material Properties:

- Concrete density: 24 kN/m³
- Characteristics strength of concrete, f_{ck} : 50 MPa
- Cement type: Normal hardening
- Ultimate tensile strength of prestressing steel, UTS: 1500 MPa
- Modulus of elasticity of prestressing steel, E_{ps} : 1.95x10⁵ MPa
- Type of strand: Low Relaxation

Loading Details:

- Initial prestress applied, f_{pi} : 0.75*UTS

- Concrete age at the time of prestress loading, t_0 : 7 days

Material model:

- Creep and shrinkage model: CEB-FIP90

Environmental Parameters:

- Relative Humidity, RH: 40 %
- Temperature, T: 20°C

Calculation of Prestress Loss According to the Provisions of IRC:18-2000

Initial prestress applied = $0.75 \times \text{UTS} = 0.75 \times 1500 = 1125 \text{ MPa}$

Assuming a short-term loss, i.e. prestress loss due to friction, anchorage slip and elastic shortening, of 8 percent,

$$\begin{aligned} \text{Average prestress after short-term loss} &= 0.92 \times 0.75 \times \text{UTS} \\ &= 0.92 \times 0.75 \times 1500 \\ &= 1035 \text{ MPa} \end{aligned}$$

$$\begin{aligned} \text{Force in cables, } P &= \text{Average prestress} \times \text{Area of cables} \\ &= 1035 \times 6351/1000 \\ &= 6573.28 \text{ kN} \end{aligned}$$

Eccentricity of cables at mid-span, $e_c = 0.874 \text{ m}$

Eccentricity of cables at support, $e_s = 0.156 \text{ m}$

Stress at the level of prestressing steel due to prestress, f_{CGS} :

At mid-span

$$\begin{aligned} f_{CGSC} &= P/A_c + P \cdot e_c \cdot e_c / I_{zc} = 6573.28/0.77 + 6573.28 \times 0.874 \times 0.874 / 0.2991 \\ &= 25324.33 \text{ kN/m}^2 \end{aligned}$$

At support

$$\begin{aligned} f_{CGSS} &= P/A_s + P \cdot e_s \cdot e_s / I_{zs} = 6573.28/1.075 + 6573.28 \times 0.156 \times 0.156 / 0.3319 \\ &= 6596.65 \text{ kN/m}^2 \end{aligned}$$

$$\begin{aligned} \text{Average stress due to prestress at CGS, } f_{CGS} &= (f_{CGSC} + f_{CGSS})/2 \\ &= (25324.33 + 6596.65)/2 \\ &= 15960.49 \text{ kN/m}^2 \approx 15.96 \text{ MPa} \end{aligned}$$

Prestress loss due to creep

Maturity of concrete at the time of transfer of prestress = 70 % (Assuming concrete attains 70 % of characteristics strength in 7 days)

From table 2, IRC:18-2000, Creep strain per 10 MPa = 6.1×10^{-4}

$$\text{Creep strain, } \epsilon_{CR} = 6.1 \times 10^{-4} / 10 \times 15.96 = 9.74 \times 10^{-4}$$

$$\begin{aligned}
\text{Prestress loss due to creep} &= \epsilon_{CR} * E_{ps} \\
&= 9.74 \times 10^{-4} \times 1.95 \times 10^5 \\
&= 189.93 \text{ MPa}
\end{aligned}$$

Prestress loss due to shrinkage

Age of concrete at the time of stressing = 7 days

From table 3, IRC:18-2000,

Strain due to residual shrinkage, $\epsilon_{SH} = 3.5 \times 10^{-4}$

$$\begin{aligned}
\text{Prestress loss due to shrinkage} &= \epsilon_{SH} * E_{ps} \\
&= 3.5 \times 10^{-4} \times 1.95 \times 10^5 \\
&= 68.25 \text{ MPa}
\end{aligned}$$

Prestress loss due to relaxation

Initial stress in cables = $0.92 * 0.75 * UTS = 0.69 * UTS = 0.69 \times 1500 = 1035 \text{ MPa}$

From table 4A, IRC:18-2000, for low relaxation steel,

Relaxation loss at 1000 hours $\approx 2.5 \%$

Long-term relaxation loss percent = $3 \times 2.5 \% = 6.75 \%$

Long-term relaxation loss = $6.75/100 \times 1035 = 69.86 \text{ MPa}$

$$\begin{aligned}
\text{Total long-term prestress loss} &= 189.93 + 68.25 + 69.86 \\
&= 328.04 \text{ MPa}
\end{aligned}$$

$$\begin{aligned}
\text{Percent long-term prestress loss} &= 328.04/1125 \times 100 \\
&= 29.16 \%
\end{aligned}$$

Comparison of Long-Term Prestress Loss Calculated Using IRC:18-2000 with that Calculated by Incremental Time-Step Method Using CEB-FIP90 Model

Long-term prestress loss has been presented using the following approaches:

- a) Incremental time-step method using CEB-FIP90 model for creep and shrinkage of concrete and relaxation formulae for prestressing steel.
 Predicted long-term prestress loss = 26.12 %
- b) Provisions of IRC:18-2000.
 Predicted long-term prestress loss = 29.16 %

Predicted loss by using IRC:18-2000 is about 12 % more than that predicted by incremental time-step method (approach a). So, IRC:18-2000 gives a conservative estimate of long-term loss.