INCREMENTAL TIME-STEP METHOD FOR PREDICTING LONG-TERM DEFORMATION OF A HPPC BRIDGE

Karthikeyan Jayakumar Ph.D., Lecturer, Department of Civil Engineering, National Institute of Technology, Trichy, India.

Akhil Upadhyay Ph.D., Associate Professor, Department of Civil Engineering, Indian Institute of Technology, Roorkee, India.

Navratan Mal Bhandari Ph.D., Professor, Department of Civil Engineering, Indian Institute of Technology, Roorkee, India.

ABSTRACT

The motive of this paper is to validate the proposed methodology [Incremental Time-step method] and the developed computation procedure in Excel work sheet for prediction of long-term deformation of a High Performance Prestressed Concrete (HPPC) bridge. Also an attempt has been made to predict the long-term behavior of the bridge in RM 2004 software using CEB 90 model designated as method ‘A’. The computation procedure has been carried out by using two alternative relaxation formulae namely (i). Eurocode-2 reduced relaxation designated as method ‘B’ and (ii). Low relaxation formula by PCI, PCI Committee report (1975) is designated as method ‘C’. Towards this predictions have been made for case study chosen from literature whose experimental observations based on results of actual monitoring of a bridge published in literature. In order to check the accuracy of the proposed method for predicting long-term deformation of HPPC bridges, there is a need to validate the model. In absence of availability of actual long-term monitoring data for any HPPC bridge in India, it has been felt that such a data available from any other country may be used. Accordingly, a bridge (I-section Girder Bridge) which had been constructed in Alabama, USA and had been monitored [Stallings et al. 2003, PCI Journal] for a continuous time period of 295 days for girder 1 and 2, 242 days for girder 3 and 4 and 234 days for girder 5, prior to the construction of the deck slab has been chosen. These published bridge monitoring results have been utilized for validation of the proposed analytical procedure. Reasonable match has been found with the measured and calculated results using proposed methods B and C. But, the proposed method has shown least percentage difference when compared to method C. Method A prediction has shown a higher percentage difference. This summarizes the conclusions of Stallings et al. “The current analytical techniques (by using incremental time-step method) can result in accurate prediction of long-term deformation and its losses for HPPC girders, by selection of appropriate material properties which are used at the time of girder production”.

INTRODUCTION

Prestressed concrete is ideally suited for the construction of short, medium- and long-span bridges. Ever since the development of prestressed concrete by Freyssinet in the early 1930s, the technique has found extensive application in the construction of long-span bridges. Prestressed concrete has been widely used throughout the world for simply supported, continuous, balanced cantilever and framed type bridge in the span ranging from 20 to 300 m.

The use of HPC in prestressed bridge girders can allow higher prestressing forces, which in turn permit longer practicable span lengths. The combination of longer span lengths and higher prestressing forces may lead to large calculated cambers. Overestimating camber during the design stage may unfairly discourage the use of high strength concrete and long spans ultimately nullifying the potential increase in efficiency. Camber is a function of time dependent effects in concrete. It is defined as the net upward deflection due to prestressing and the downward deflection due to self-weight and other imposed loads. Hence, eccentrically prestressed bridge girders usually exhibit net upward deflection due to this combined action both initially as well in the long-term with respect to time [1].

Description about the HPPC Bridge

The bridge consists of seven simple spans of five AASHTO BT-54 girders each. Plan and elevation views are shown in Figure 1. A cross-section of the bridge is shown in Figure 2. Geometric properties of a BT 54 cross-section are shown in Figure 3. The five interior spans have a span length of 34.21 m each between the centers of neoprene bearing pads. The two end spans have a length of 33.91 m each between the centers of bearings. Strain gauges were installed in all five girders of one interior span. This span is considered typical, and all calculations correspond to this span length. Girder design properties used in the analysis are summarized in Table 1. All girders are prestressed with forty-two 15.2 mm, ultimate strength of 1860 MPa for low-relaxation steel strands. The strand profile is shown in Figure 4. Twenty-eight of the strands in the bottom flange are straight, and ten of these are sheathed for 1.22 m at each end. Fourteen strands are draped with hold-down points located 3.05 m away from midspan. Mid span and end cross sections are shown in Figure 5.

(Figures 1 to 5 Courtesy: Stallings et al.2003.PCI Journal)

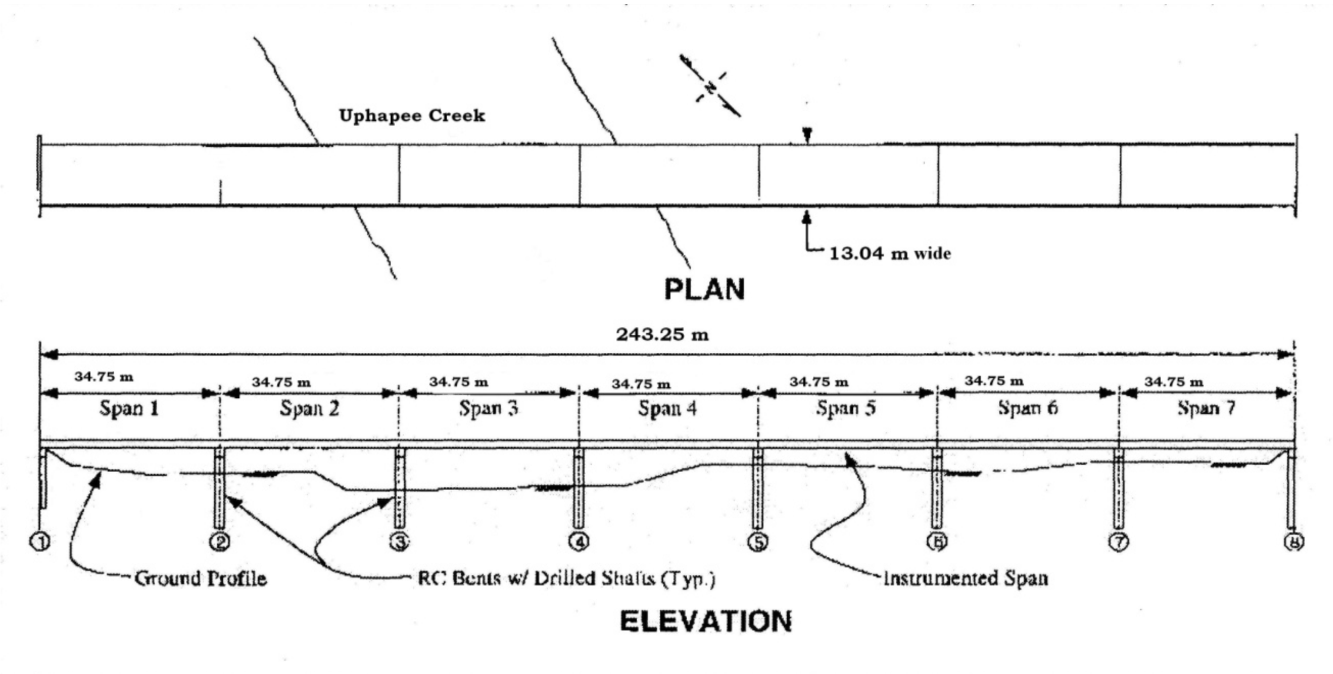

Figure 1 Plan and Elevation of the HPPC Bridge.

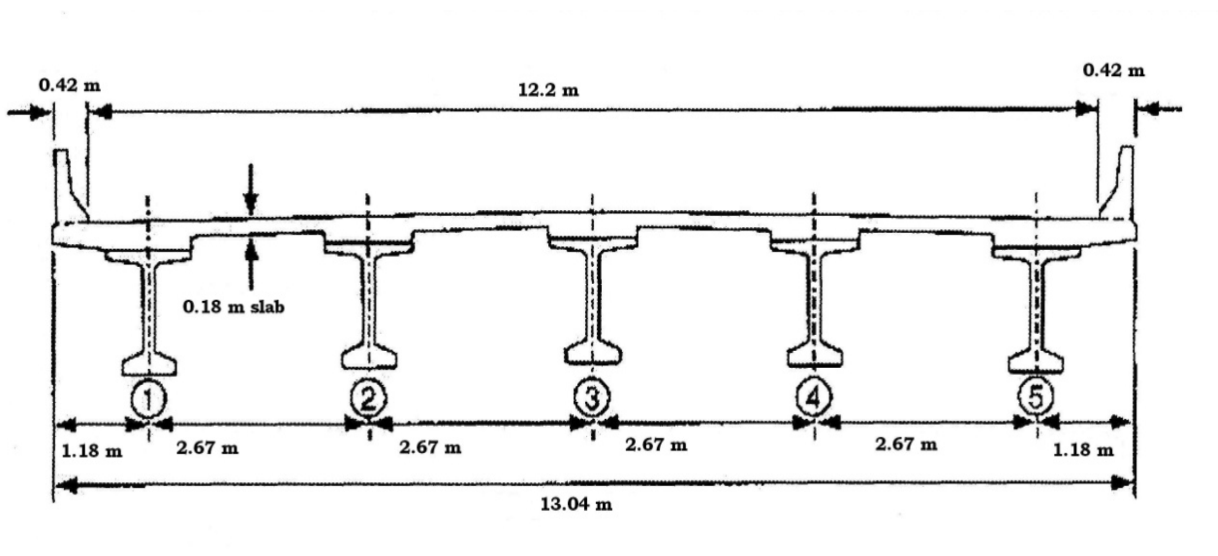

Figure 2. Cross-section of the HPPC Bridge.

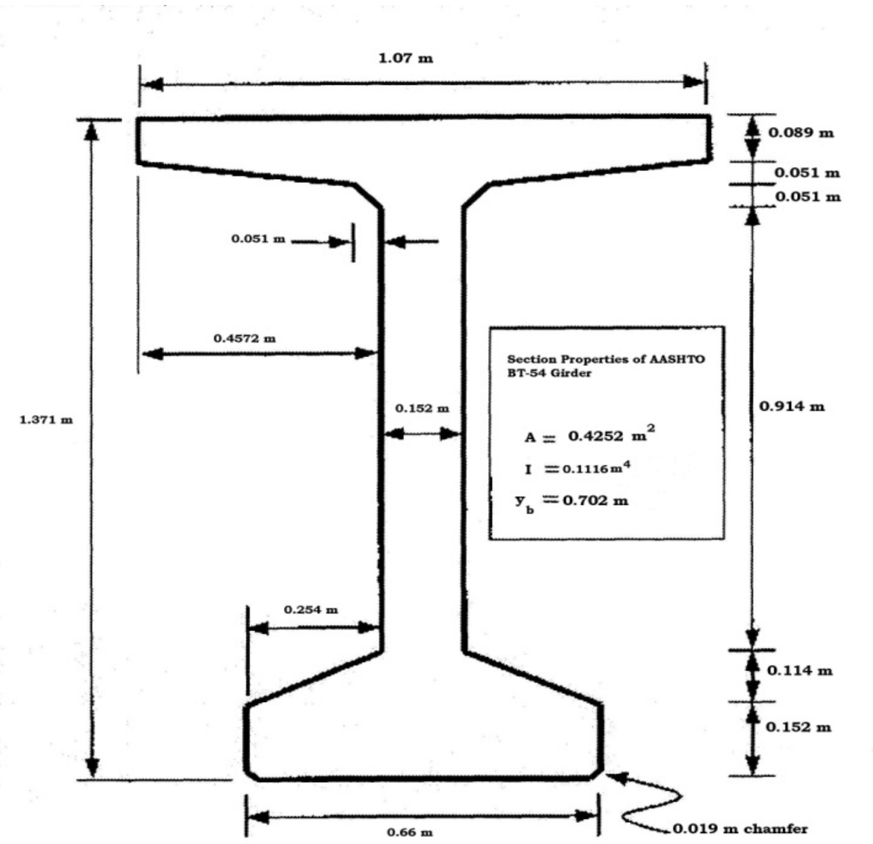

Figure 3. Cross-sectional properties of the AASHTO BT-54 girders.

Table 1. Design Parameters for HPPC girders.

Prestressing steel: 15.2 mm Low-relaxation strands	Number	42
	Strand Area	1.42 x 10^{-4} m^2 (0.559 $in^{2)}$
	Ultimate tensile strength, f_{pu}	1860 MPa (269.7 ksi)
	Modulus of elasticity, E_{ps}	189.6 GPa (27492 ksi)
	Jacking stress, f_{pj}	0.75 f_{pu}
Girder characteristics	Span length, L	34.21 m (112.23 ft)
	Hold-down location, a	14.06 m (46.13 ft)
Concrete properties	Release strength, f_{ci}’	55.16 MPa (7998.2 psi)
	28-day strength, f_c’	68.95 MPa (9997.75 psi)
	Unit weight	2399 kg/m^3 (149lb/ft^3)
	Modulus of elasticity, E_c	39.58 GPa (5739.1 ksi)

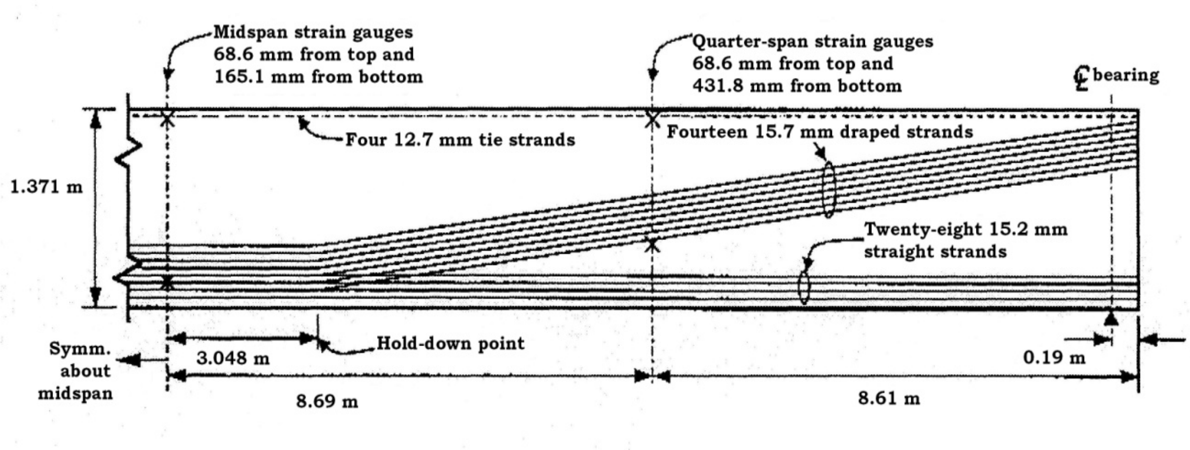

Figure 4. Cable profile for the BT-54 girder along with strain gauge locations.

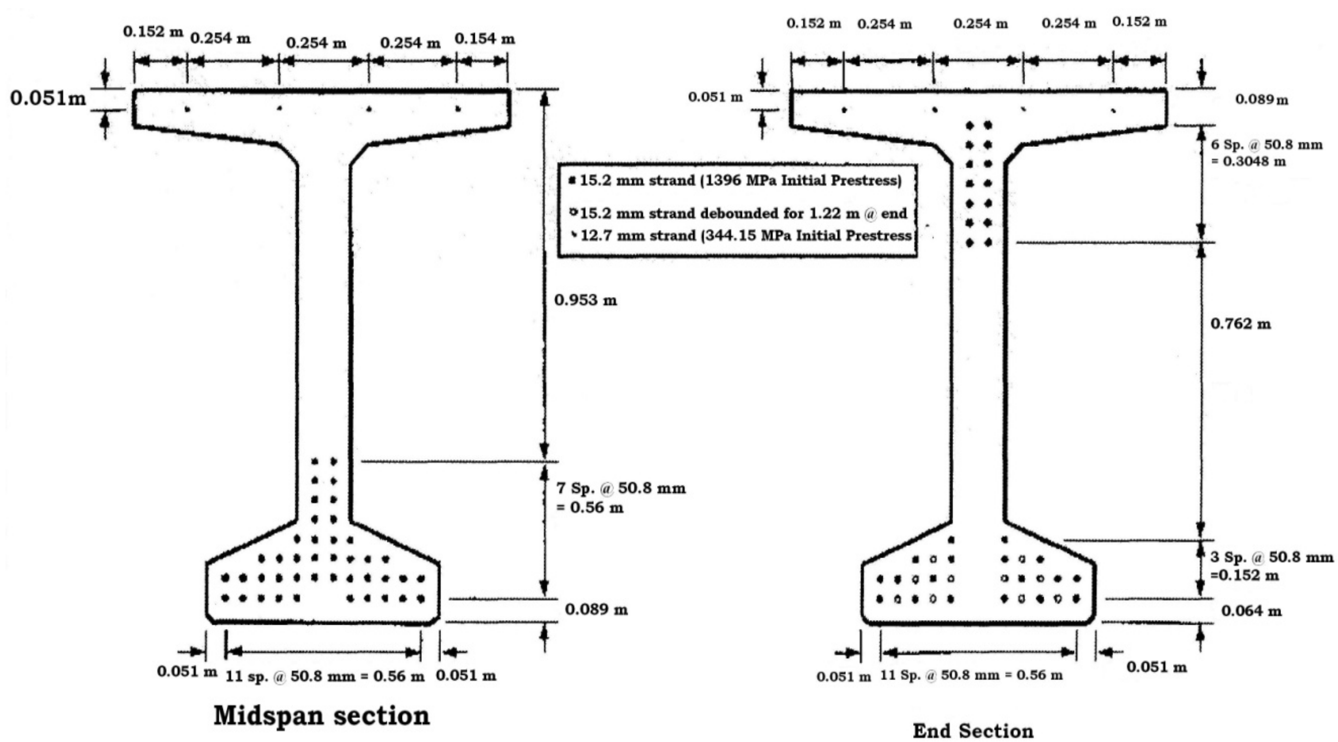

Figure 5. Midspan and end cross sections of HPPC girder showing the strands.

Measured results of the HPPC Bridge – A Review

Based on the above specified dimensions and design parameters, the I-section girder bridge is analysed, designed and constructed in Alabama- USA. The creep and shrinkage properties are determined using cylinders from single production of casting by using the same materials at the time of girder production. The specimens are match cured until the prestress was transferred to the girders. They are moved to the laboratory and the specimens are tested for creep and shrinkage. The measured creep coefficients and shrinkage strains for 295 days were found to be 1.22 and 460 x 10^{-6} respectively. The actual camber of a girder at any age is the algebraic sum of upward deflection due to prestressing and the downward deflection due to self-weight and other applied loads. Figure 6 shows the measured cambers of the five instrumented HPPC girders namely G1 to G5 for the specified days [Stallings et al.2003]. The measured camber results of the instrumented HPPC girders prior to application of any dead load other than the member self weight.

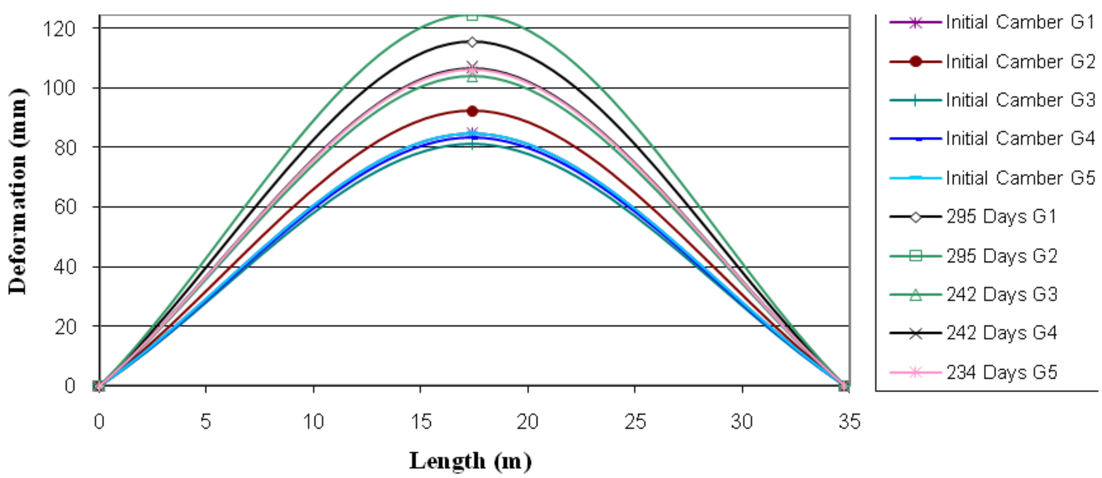

Figure 6. Measured Camber of Instrumented HPPC Girders (Stallings et al.2003)

Modelling of the HPPC bridge using RM 2004

The above bridge has been modeled as per the specified dimensions and design parameters by using RM 2004 bridge engineering software. The long-term deformation for the girders has been measured prior to the construction of deck slab. However, the bridge has been modeled the girders along with the deck slab only for aesthetic purpose, as shown in the (3-D view) Figure 8. The design parameters required for the deck slab portion in the bridge model have been specified as zero and the analysis has been carried out only for the girders. The long-term analysis is performed in RM 2004 software using CEB 90 model prediction designated as method - A. The modeled bridge girders have been analyzed for the dead load, prestressing and its load combination. Figures 7 and 8 show the arrangement of strands and 3D view of the bridge model. Figure 9 shows the long-term deformation of the bridge at 295 days for girders G1 and G2. For the other girders, the values have been tabulated in Table 2 and 3.

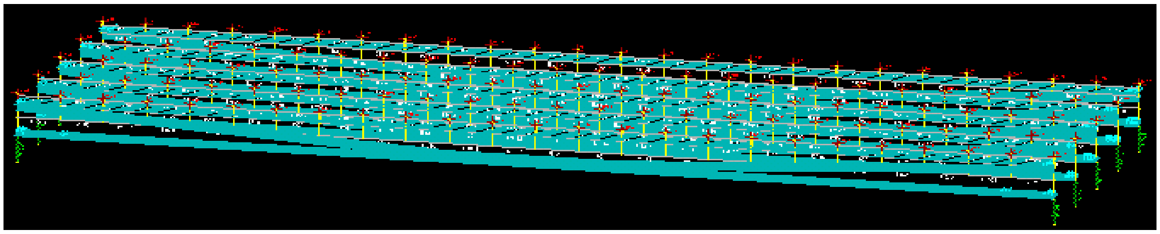

Figure 7. Strands of the modeled HPPC (Alabama) bridge

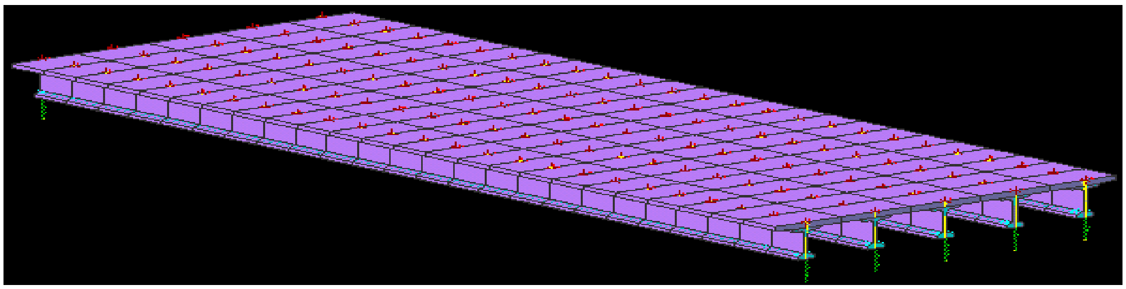

Figure 8. 3D view of the modeled HPPC (Alabama) bridge

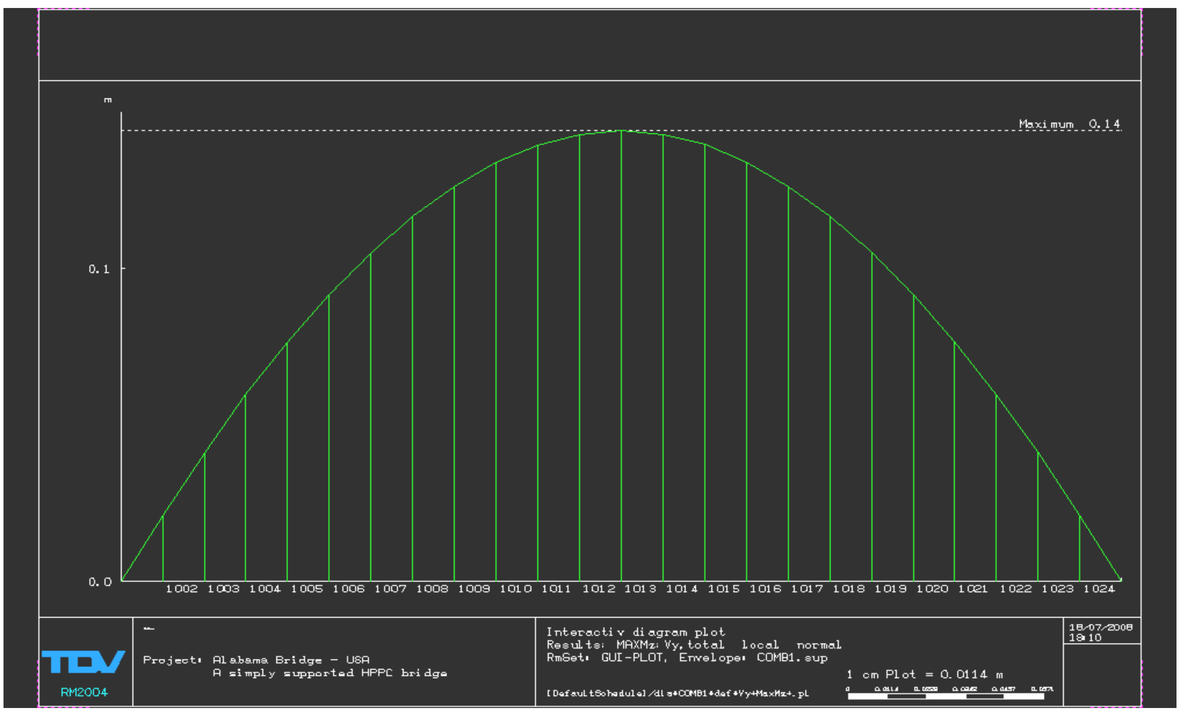

Figure 9 Long-term deformation [295 Days] of a simply supported HPPC bridge [Method – A]

Table 2 Method A prediction versus measured results

Girder	Age (Days)	Long-term Deformation at Mid-span mm (in)		Difference mm (in)	Percentage difference
		Measured (Stallings et al.)	Method A		
G1	1 295	84.84 115.57	74.43 142.15	-10.41 +26.58	-12.27 +23
G2	1 295	92.2 124.46	74.43 142.15	-17.77 +19.66	-19.27 +14.21
G3	1 242	81.03 103.89	74.43 140.54	-6.6 +36.65	-8.15 +35.28
G4	1 242	83.31 106.68	74.43 140.54	-8.88 +33.86	-10.66 +31.74
G5	1 234	84.84 105.92	74.43 140.26	-10.41 +34.34	-12.27 +32.42

Table 3 Method A prediction versus calculated results

Girder	Age (Days)	Long-term Deformation at Mid-span mm (in)		Difference mm (in)	Percentage difference
		Calculated (Stallings et al.)	Method A		
G1	1 295	76.71 109.474	74.43 142.15	-2.28 +32.676	-2.97 +29.85
G2	1 295	76.71 109.474	74.43 142.15	-2.28 +32.676	-2.97 +29.85
G3	1 242	76.71 107.696	74.43 140.54	-2.28 +32.844	-2.97 +30.5
G4	1 242	76.71 107.696	74.43 140.54	-2.28 +32.844	-2.97 +30.5
G5	1 234	76.71 107.492	74.43 140.26	-2.28 +32.768	-2.97 +30.48

Comparison of long-term deformation with the measured

The long-term deformation of the HPPC bridge is analyzed using the incremental time-step procedure. The above analysis is carried out by using two alternative relaxation formulae namely; (i) Eurocode-2 reduced relaxation (Hendy and smith, 2007) designated as method 'B', and (ii) Magura et al. relaxation (Magura et al. 1964) designated as method 'C'. The proposed procedure is divided into 11 discrete time steps for estimating the long-term deformation starting from 1, 3, 7, 21, 28, 65, 85, 125, 155, 212 and 270 days for G1 and G2 girders, 1, 3, 7, 21, 28, 65, 85, 125, 155, 212 and 242 days for G3 and G4 girders, 1, 3, 7, 21, 28, 65, 85, 125, 155, 212 and 234 days for G5 girder respectively using MS Excel. The experimental creep coefficients and shrinkage strains measured by Stallings et al. are used in these methods. The net curvature obtained from time-step method gives the net-long-term moment due to prestressing, using simple bending theory. The long-term deformation for the particular time interval can be calculated by finite difference method. Initial camber at 1 day [sum of deflections due to dead load, initial prestressing (i.e. immediately after prestressing) and other imposed loads] is also calculated. Table 4 shows the comparison of initial camber and long-term deformation for the HPPC girder using the proposed method B versus measured results of Stallings et al. Table 5 shows the comparison of initial camber and long-term deformation for the HPPC girder using the proposed method B versus calculated results of Stallings et al. Similarly, Tables 6 and 7 shows the comparison of initial camber and long-term deformation for the HPPC girder using method C. Figures 10 and 11 shows the comparison of initial camber and long-term deformations using different methods respectively.

Table 4 Method B prediction versus measured results

Girder	Age (Days)	Deformation at Mid-span mm (in)		Difference mm (in)	Percentage difference
		Measured (Stallings et al.)	Method B		
G1	1 295	84.84 (3.34) 115.57 (4.55)	74.43 (2.93) 110.95(4.37)	-10.41 -4.62	-12.27 -4.00
G2	1 295	92.2 (3.63) 124.46 (4.9)	74.43 (2.93) 110.95(4.37)	-17.77 -13.51	-19.27 -10.85
G3	1 242	81.03 (3.19) 103.89 (4.09)	74.43 (2.93) 107.36(4.23)	-6.6 +3.47	-8.15 +3.34
G4	1 242	83.31 (3.28) 106.68 (4.2)	74.43 (2.93) 107.36(4.23)	-8.88 +0.68	-10.66 +0.64
G5	1 234	84.84 (3.34) 105.92 (4.17)	74.43 (2.93) 106.813(4.21)	-10.41 +0.893	-12.27 +0.843

Table 5 Method B prediction versus calculated results

Girder	Age (Days)	Deformation at Mid-span mm (in)		Difference mm (in)	Percentage difference
		Calculated (Stallings et al.)	Method B		
G1	1 295	76.71 (3.02) 109.474 (4.31)	74.43 (2.93) 110.95(4.37)	-2.28 +1.476	-2.97 +1.35
G2	1 295	76.71 (3.02) 109.474 (4.31)	74.43 (2.93) 110.95(4.37)	-2.28 +1.476	-2.97 +1.35
G3	1 242	76.71 (3.02) 107.696 (4.24)	74.43 (2.93) 107.36(4.23)	-2.28 -0.336	-2.97 -0.312
G4	1 242	76.71 (3.02) 107.696 (4.24)	74.43 (2.93) 107.36(4.23)	-2.28 -0.336	-2.97 -0.312
G5	1 234	76.71 (3.02) 107.492 (4.23)	74.43 (2.93) 106.813(4.21)	-2.28 -0.679	-2.97 -0.632

Table 6 Method C prediction versus measured results

Girder	Age (Days)	Deformation at Mid-span mm (in)		Difference mm (in)	Percentage difference
		Measured (Stallings et al.)	Method C		
G1	1 295	84.84 (3.34) 115.57 (4.55)	74.43(2.93) 107.32(4.2)	-10.41 -8.252	-12.27 -7.14
G2	1 295	92.2 (3.63) 124.46(4.9)	74.43 (2.93) 107.32 (4.2)	-17.77 -17.142	-19.27 -13.77
G3	1 242	81.03 (3.19) 103.89 (4.09)	74.43 (2.93) 106.61 (4.2)	-6.6 +2.722	-8.15 +2.62
G4	1 242	83.31 (3.28) 106.68 (4.2)	74.43(2.93) 106.61(4.2)	-8.88 -0.068	-10.66 +0.064
G5	1 234	84.84 (3.34) 105.92 (4.17)	74.43(2.93) 106.24(4.18)	-10.41 +0.318	-12.27 +0.3

Table 7 Method C prediction versus calculated results

Girder	Age (Days)	Deformation at Mid-span mm (in)		Difference mm (in)	Percentage difference
		Calculated (Stallings et al.)	Method C		
G1	1 295	76.71(3.02) 109.474(4.31)	74.43(2.93) 107.32(4.2)	-2.28 -2.156	-2.97 -1.97
G2	1 295	76.71(3.02) 109.474(4.31)	74.43 (2.93) 107.32 (4.2)	-2.28 -2.156	-2.97 -1.97
G3	1 242	76.71(3.02) 107.696(4.32)	74.43 (2.93) 106.61 (4.2)	-2.28 -1.084	-2.97 -1.01
G4	1 242	76.71(3.02) 107.696(4.32)	74.43(2.93) 106.61(4.2)	-2.28 -1.084	-2.97 -1.01
G5	1 234	76.71(3.02) 107.492(4.32)	74.43(2.93) 106.24(4.18)	-2.28 -1.254	-2.97 -1.17

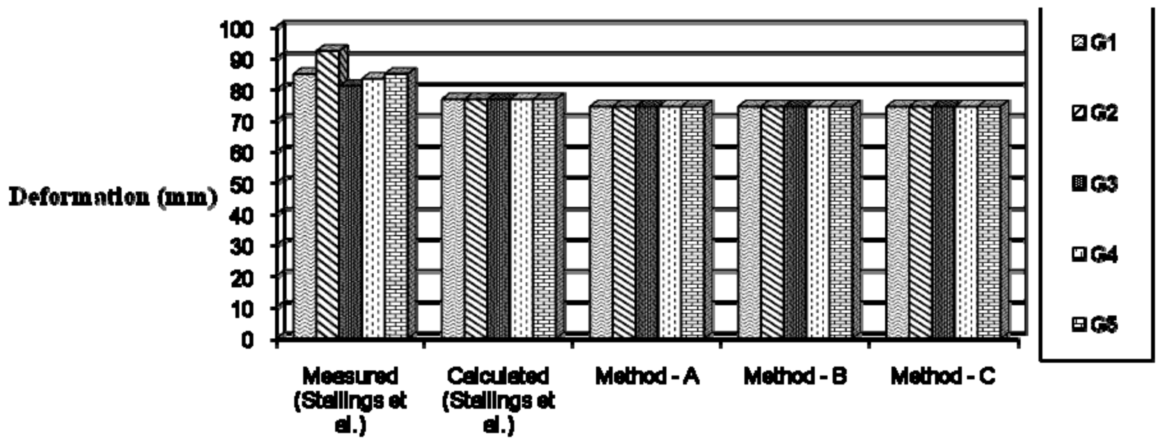

Figure 10 Comparison of the Initial camber using different methods

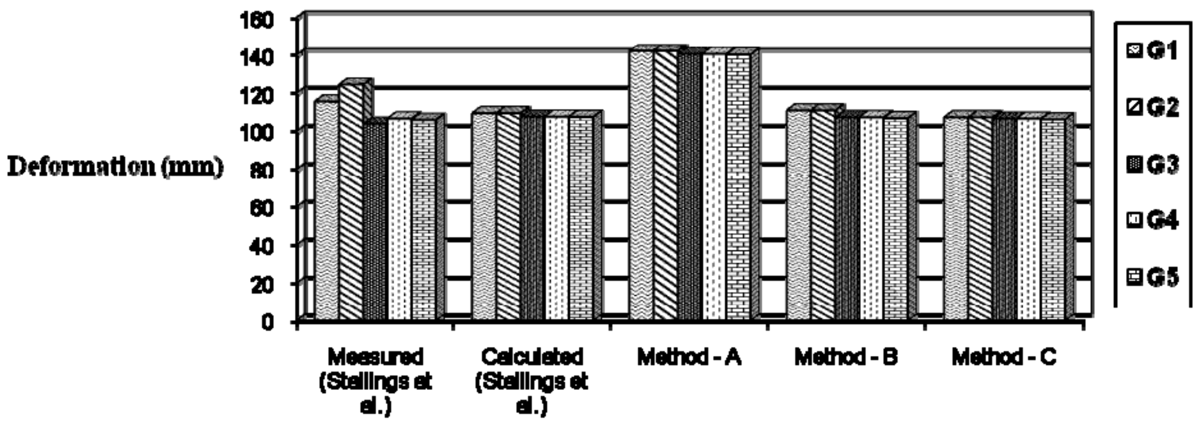

Figure 11 Comparison of long-term deformations using different methods

Discussion of results

Table 2 show the method A prediction versus measured results of Stallings et al. It has been inferred that the percentage difference between the method A and measured results for the initial camber and long-term ranging from -8.15 to -19.27 and +14.21 to +35.28 respectively. Table 3 shows the percentage difference between the method A and calculated results of Stallings et al. for the initial camber , it has been observed as -2.97 and the long-term deformation ranging from +29.85 to +30.5. Table 4 shows the percentage difference between the method B and measured results of Stallings et al. for the initial camber ranging from -8.15 to -19.27 and the long-term deformation ranging from -10.85 to +3.34. Table 5 shows the percentage difference between the method B and calculated results of Stallings et al. for the initial camber as -2.97 and the long-term deformation ranging from -0.632 to +1.35. Table 6 shows the percentage difference between the method C and measured results of Stallings et al. for the initial camber ranging from -8.15 to -19.27 and the long-term deformation ranging from -13.77 to +2.62. Table 7 shows the percentage difference between the method C and calculated results of Stallings et al. for the initial camber as -2.97 and the long-term deformation ranging from -1.97 to -1.01. The initial and final deformations using different methods have been shown in Figures 10 and 11 in the form of bar charts.

The long-term deformation predicted by RM 2004 using CEB 90 model (Method A prediction) have shown a higher percentage difference greater than +30, in both the measured and calculated results of Stallings et al. This is due to the creep coefficients and shrinkage strains predicted by CEB 90 model, which was higher than the measured creep and shrinkage results of Stallings et al.

The percentage difference between the proposed methods B calculated value of Stallings et al. have ranged from -0.632 to +1.35. Similarly, for method C, it ranged from -1.97 to -1.01. The percentage difference between the computed values of long-term deformation by the proposed methods B and the measured values of Stallings et al. have been ranging from -10.85 to +3.34. Similarly, for method C, the difference between the predicted values and Stallings et al. computed values have been ranging from -13.77 to +2.62.

Reasonable match has been found with the measured and calculated results using proposed methods B and C. But, the proposed method B has shown least percentage difference when compared with method C. According to Hendy and Smith (2007), relaxation losses are sensitive to variations in stress levels over time and can therefore be reduced by taking account of other time-dependent losses occurs within the structure at the same time (such as creep and shrinkage), while Magura et al. relaxation formula does not consider the same. Thus, the reason stated for the less percentage difference in method B.

This has been ensured that the validation results for long-term deformation using incremental time-step method showed a good agreement with the measured results. Hence, It has been demonstrated that the incremental time-step method is the most appropriate and rational method for predicting the long-term deformation for HPPC bridges.

CONCLUSIONS

Using the developed ANN model and incremental time-step method, an application tool for the calculation of long-term deformations in HPPC bridges is developed. For the validation of this tool, experimental long-term deformation observations reported by Stallings et al.2003 for HPPC girders are used. In these calculations, the creep coefficient and shrinkage strains reported by Stallings et al. 2003 are incorporated for the sake of validation with the following observations.

Case	Range in difference (%)
Present approach (Eurocode-2 reduced relaxation formula) Vs Measured deformations (Stallings)	-10.85 to +3.34
Present approach (Magura relaxation formula) Vs Measured deformations (Stallings)	-13.77 to +2.62
Present approach (Eurocode-2 reduced relaxation formula) Vs Analytical (Calculated) deformations (Stallings)	-0.632 to +1.35
Present approach (Magura relaxation formula) Vs Analytical (Calculated) deformations (Stallings)	-1.97 to -1.01
RM 2004 using CEB 90 model Vs Measured deformations (Stallings)	+15.8 to +38.7
RM 2004 using CEB 90 model Vs Analytical (Calculated) deformations (Stallings)	+31.65 to +34.08

Reasonable match with experimental and analytical results validates the present approach for calculation of long-term deformations in HPPC bridges.

REFERENCES

1. Stallings, J. M., Barnes R. W. and Eskildsen. S. “Camber and prestress loss in Alabama HPC bridge girders.” *PCI Journal*, V 48 No.5, Sept-Oct 2003, 90-104.
2. Szilard, R. (2004). “Theories and applications of plate analysis – classical, numerical and engineering methods.” Prentice Hall, New Jersey, USA.
3. Magura, D. D., Sozen, M. S. and Siess, C. P. “A study of Stress Relaxation in Prestressing Reinforcements.” *PCI Journal*, 9(2), 1964, 13-57.
4. IRC: 18 (2000), “Design Criteria for Prestressed concrete road bridges (Post-tensioned concrete).” 3rd revision, The Indian Road Congress, New Delhi, India.
5. Hendy, C. R. and Smith, D. A. (2007). “Designers guide to EN 1992-2 Eurocode 2: Design of concrete structures-part 2: Concrete bridges.” Thomas Telford ltd., London, UK.